CONSTRUCTED WETLANDS AS AN APPROPRIATE TREATMENT OF LANDFILL LEACHATE

by

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ABSTRACT

One of the main environmental problems associated with the disposal of waste on land is the release of liquid emissions from the site. This wastewater, known as leachate, is a product of the biodecomposition of the waste and filtrates from the landfill once the moisture saturation of the fill has been reached. The chemical composition of leachate is variable over time and between sites. Regardless of these variables, the main pollutants of concern in the leachate are ammonia and organics, both of which can cause environmental degradation in relatively low concentrations. Worldwide and in South Africa, leachate has either been directly released into the environment or into the local domestic sewage system. As more has been learned about the human and environmental health risks associated with these disposal methods, there has been a new focus in waste management toward treating the leachate at the source as part of the broader focus of sustainable landfilling. One of the treatment options being used is constructed wetlands (CW) due to the physical and chemical transformation mechanisms in these biological systems. This treatment process has been demonstrated to be effective as a final polishing treatment for leachate, and it is considered a technology appropriate in the South African context. Therefore the aim of dissertation is to ascertain the use of constructed wetlands as an appropriate treatment option for untreated methanogenic landfill leachate by determining the efficiency of ammonia and organic removal in a pilot-scale vegetated submerged bed (VSB) constructed wetland (CW) planted with *Phragmites australis*. During the 22-week treatability trial the VSB achieved an ammonia concentration removal efficiency of 91% and mass removal efficiency of 87%. Despite this substantial reduction of ammonia, the VSB was unable to achieve the required discharge standard. There were erratic fluctuations in both the treatment efficiencies for COD and BOD, and the results show no evidence of constant reduction of organics during the treatability trials. This is due to the refractory nature and the low biodegradability of the organics that remain in methanogenic leachate as suggested by the low BOD to COD ratio. Due to the low biodegradability of the organics, a biological treatment system, such as a VSB, will not be able to reliably meet the required discharge standards. Other passive treatment options or a combination of systems need to be explored in order to both satisfy legislative requirements and be appropriate in the South African context.

DEDICATION

This dissertation, like my life, is dedicated to the Glory and Love of God to whom I will be forever thankful for welcoming His prodigal daughter back home and for giving purpose to my life.

I was pushed back and about to fall,
but the Lord helped me.
The Lord is my strength and my song;
He has become my salvation

You are my God and I will give You thanks.
You are my God and I will exalt You.

-Psalm 118:13-14, 28

PREFACE

The experimental work described in this dissertation was carried out in the School of Life & Environmental Sciences, University of Natal, Durban, South Africa, from January 2003 to August 2003 under the supervision of Professor Gerry Garland and Dr. Cristina Trois.

These studies represent original work by the author and have not otherwise been submitted in any form for any degree or diploma to any tertiary institution. Where use has been made of the work of others it is duly acknowledge in the text.

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Chapter One

Introduction

1.1 Overview of Landfills and Leachate

For millions of years equilibrium existed between earth's life forms and the environment with waste products being readily assimilated as inputs into other processes. As civilizations developed and people began living in larger more stable groupings, this equilibrium began to falter as people generated domestic waste at an increased rate (Williams 1998). Modern society continues to exceed the assimilative capacity of the environment by creating items that result in large amounts of often-complex waste products (Peavy *et al* 1985). These waste products were historically disposed of in unmanaged waste sites, but more recently placed in highly engineered landfills (Qasim and Chiang 1994). Once at the site, the waste products are degraded via a number of biological and chemical processes. These suspended or soluble organic and inorganic products are flushed through the landfill by precipitation and/or groundwater entering the waste. The result is a highly complex wastewater known as landfill leachate.

Studies have shown that the quantity and quality of the leachate will change over time and will differ between landfills (Ehrig 1984; Christensen and Kjelden 1989). Despite these differences, in general the main pollutants in leachate are lesser biodegradable organics and ammonia (Vasel 2002). Organics found in the leachate are at various stages of microbial decomposition and are an environmental concern because their further decomposition in aquatic ecosystems results in a reduction in dissolved oxygen, which can create a toxic environment. Ammonia is a product of the decomposition of proteins in the landfill and cannot be further reduced in the landfill due to the anaerobic conditions present. The reason ammonia is of particular environmental concern is because it is a toxic compound in aquatic ecosystems at fairly low concentrations.

To protect the environment and human health, the design and management of landfills and the legislation governing landfills and leachate have been developed and modified to minimize the impact of leachate on the surrounding environment. These changes have transformed landfills into highly engineered waste storage facilities that attempt to protect the environment by encapsulating the waste and controlling the release of leachate. While this method is sufficient during the short-term, there are concerns regarding the lifespan of such controls and the long-term pollution potential of the leachate being released. This has resulted in conflict within the waste management industry and within the legal framework regarding how the landfills should be managed and regulated to attain the goal of being less polluting. The concern with isolating the waste from the environment is that it is not a long-term environmental solution because the potential hazards do not diminish with time even if the risk is low (Robinson 1995b). Because of this the sustainability debate becomes an issue of timescale. These entombment practices promote short-term environmental protection while other landfill management options focus on the desire to purge all the pollutants within a generation.

1.2 Leachate Treatment in South Africa

The way in which the leachate is disposed of is also an aspect of creating a less-polluting landfill. Leachate can either be released into the environment or sent into the local sewage system. It may be pretreated prior to disposal or discharged directly from the landfill. There are environmental and human health risks with all these options. When leachate is left untreated and disposed of by dilution into the ground or surface water, the ecosystem may be degraded depending on the quantity and quantity of the leachate. If leachate is transported to municipal sewage works, there is potential danger of methane explosions (Robinson 2001). There is also an expense of sending the leachate off-site to be treated and often this is not an option. In South Africa, most often the leachate is sent into a nearby sewer without any pretreatment or when that is not available it is released directly into the environment. Relying on the availability of a sewer system to treat leachate is a concern because new landfills may be sited in remote locations where there is no available sewer for discharge. In this case there will be a need for an appropriate onsite treatment in order to release the leachate into the natural environment.

The study site used in this research is located at the Bisasar Road Landfill in Durban, South Africa, which is the largest landfill in the Durban Metropolitan Area and releases an average 200,000 liters/day of leachate to the local sewage system. The Bisasar Road Landfill is considered an old landfill due to the methanogenic qualities of the leachate; of particular concern are its high ammonia (1200 mg/l) and organic (2600 mg/l as determined by COD) concentrations (Robinson *et al* 1997). The focus of the treatment therefore has been on the removal of nitrogen and organics. Studies have shown that ammonia is most efficiently removed from leachate through a combination of nitrification and denitrification treatment, such as a sequencing batch reactor (SBR) (Robinson 1995b; Strachan 1999). While typical performance efficiencies of such treatment processes may be sufficient to meet the legislative requirements for disposal into sewage treatment works, it does not meet the requirements for disposal into the local water systems. Therefore, there is a need for further treatment either by the sewage system or onsite treatment. Also the high cost of the carbon input needed for the denitrification of the leachate may limit the use of this treatment process.

The SBR was found to be successful in lowering the nitrogen concentration but did not remove a sufficient amount of organics. Therefore the landfill will still pay by volume for the treatment of the effluent sent to the sewer system. This substantial cost was one of the motivations for installing a polishing treatment to meet the South Africa General Standard requirements for the effluent could not be discharged otherwise (S.A. Government Gazette 1999). It was decided to add a final stage polishing treatment to remove this residual COD. Vegetative submerged beds (VSB) constructed wetlands (CW) were chosen because they have been shown to be successful in removing trace amounts of pollutants from low-strength raw leachates and treated effluents from biological treatment plants (Robinson 1993; Robinson *et al* 1997, 1998; Cossu *et al* 1997). The VSB were found to be easy to operate and inexpensive in comparison to other treatment options, but they could not reliably reduce the COD concentration to a level set by the General Standard (Trois *et al* 2002).

1.3 Long-term Solution to Landfill Leachate

Another issue with leachate is that it remains a potential pollutant for an unknown time after the landfill has been closed. An appropriate long-term solution to treat and to dispose of the leachate is needed. There are fluctuations in the quality of the effluent due to the changes in the landfilled wastes, so the treatment processes must be able to act as a buffer by ensuring that the peaks in the concentrations are moderated before release (Robinson *et al* 1993). For a CW to be considered an appropriate long-term treatment option, it must satisfy the criteria for appropriate technology: be affordable, be easy to operate, and be reliable under various conditions. CW are relatively low-cost in both initial infrastructure and operation and require minimum maintenance. Also they have been shown to create an anaerobic environment that would support denitrification and allow for the natural fluctuations in pollutant concentration. They would thus be an appropriate on-site polishing and buffering process. Besides using CW as a treatment option during the active life of the landfill, there is need to examine their efficiency during aftercare when the landfill reaches the methanogenic, stable phase. Vasel (2002) reviewed the treatment efficiencies and processes of using CW to treat leachate during the methanogenic and after care periods, and showed CW as an appropriate treatment option for landfill leachates during the immediate aftercare period (as long as the liner and collection system continue to function). It would require the landfill to have a barrier and collection system but the design itself could be seen as part of an engineered attenuation system to release the leachate back into the environment in an appropriate way.

1.4 Aim and Objectives

The aim of this research was to study the use of CW as a treatment option to protect aquatic ecosystems from being degraded by leachate being emitted from landfills. This research expanded on the 2001 study of the VSB as an appropriate treatment method for leachate in South Africa (Olufsen 2003). Instead of using it as a post-nitrification and denitrification option as in the 2001 study, the main objective was

- To ascertain the use of constructed wetlands as an appropriate treatment option for untreated methanogenic landfill leachate by determining the efficiency of ammonia and organic removal in a pilot-scale vegetated submerged bed (VSB) constructed wetland (CW) planted with *Phragmites australis*.

In order to better evaluate and to discuss treatment options, several of aspects surrounding the production and treatment of landfill leachate must be explored and the general processes within a landfill must be understood. To accomplish this the following objectives were undertaken:

- To assess landfill leachate production, generation and potential environmental impacts;
- To determine if constructed wetlands could be used as a sustainable or appropriate leachate treatment option as part of the legislative framework focused on pollution prevention and sustainability; and
- To explore functions and mechanisms of constructed wetlands in order to determine whether it is possible to use them as a sustainable or appropriate long-term treatment option for landfill leachate.

Chapter Two

Landfills and Leachate

2.1 Overview of Landfills and Leachate

2.1.1 Landfills in Waste Management

The generation of solid waste occurs in all modern societies (Qasim and Chiang 1994). The production of waste is a result of the resource cycle, which includes the transformation of inputs (materials and energy) to outputs (products, energy, and waste). In some cases those waste products can be used as an input to another process, but rarely are they reintroduced into the resource cycle (Powrie 2003). Whether or not this is truly a cyclical process is debatable, since the vast majority of the by-products created are not reusable in any manner (Hawken 1993). The result of this linear process is the creation of unusable by-products (i.e. waste), which then become a societal problem of waste management as communities contend with the best way of disposal (Peavy *et al* 1985).

The two most used routes for final disposal are currently incineration and landfilling. While incineration is a viable and encouraged option for some wastes such as medical and certain hazardous waste, residue from this will still require final disposal, often causing greater environmental impacts (Robinson 1995b). Most countries rely on landfills as the most common option for final disposal; this is true for developing countries, such as South Africa (Röhrs and Fourie 2002) as well as developed countries, such as the United States (US EPA website 2003).

Despite their commonality, there is growing opposition to new landfill site developments due to public dislikes and environmental pollution. Also there is concern about the loss of wasted resources in the use of space and disposed waste (Qasim and Chiang 1994). Because of these concerns, many countries have developed a waste management hierarchy which seeks to rank the most environmentally sound strategies for municipal solid waste (MSW). Source reduction (including reuse) is the most preferred method, followed by recycling and composting, and lastly, disposal in combustion facilities and landfills (UK Doe and Welsh Office 1995; US EPA 1989). While waste avoidance is a positive proactive goal that should be pursued by governments, there will always be waste as part of consumption. Since increased consumption is the driving force of the economies of countries, some type of final disposal option will be needed (Powrie 2003). For example in the Netherlands, despite their policy for waste reduction, all waste streams continue to increase (Mathlener 2001). As a country continues to grow and develop, its waste composition will change and increase. In order to handle this waste increase, the final disposal method should be the most environmentally sensitive given cost restrictions (Qasim and Chiang 1994; Powrie 2003).

The combination of a lack of viable and cost-efficient alternatives makes landfills the most feasible economic option (Mathlener 2001). They are also considered the Best Practical Environmental Option (BPEO) for the foreseeable future (Robinson 1996), mainly because even landfill alternatives (such as incineration) create an environmental impact and result in waste products. So despite public opinion regarding the impact of landfills, they will continue to be necessary in order to manage non-recyclable and non-combustible wastes. Environmental concerns at modern landfills have been minimized due to the use of more elaborate pollution control and monitoring devices (Qasim and Chiang 1994). These improvements in landfill design, operation and aftercare should continue as countries focus on waste minimization and recycling (Robinson 1995b). While these environmentally responsible waste management practices should be promoted, the overarching goal should be to rearrange our relationship from a linear system of creation and waste to a cycling one, thereby striving for sustainability by imitating the natural process of waste-equals-food principle that is found in nature (Hawken 1993).

2.1.2 Leachate

One of the environmental problems associated with landfills is the production and release of leachate. The Department of Water Affairs and Forestry (DWAF) of South Africa define landfill leachate as: "An aqueous solution with a high pollution potential, arising when water is permitted to percolate through decomposing waste. It contains final and intermediate products of decomposition, various solutes and waste residues. It may also contain carcinogens and/or pathogens" (SA DWAF 1998). Leachate is generated from the combination of the natural degradation of waste and the rainfall that has seeped through the waste. (Blakely 1992, Blight *et al* 1992; Qasim and Chiang 1994; Knox 2000).

As leachate is generated, it flows through the waste carrying an assortment of pollutants. This creates an effluent with more pollution potential than raw sewage or many industrial wastes (Qasim and Chiang 1994). It usually does not contain toxic substances, but some of the compounds could be at concentrations that are toxic to the receiving environment (Vasel

2002). Crawford and Smith (1985) list four main mechanisms by which contaminants are leached out of landfilled waste:

- Dissolution of inherently soluble material in the landfill (e.g. Na, Cl, SO_4, and some organics)
- Biodegradation of complex organic molecules. The simpler organic acids and alcohols tend to be more soluble than the original organics. Ntrogen is converted to soluble NH_4^+
- Chemical reduction (e.g. Fe^{+3} reduced to more soluble Fe^{+2})
- Washout of fine solid material will give rise to suspended solids and turbidity in the leachate

The first three have the greatest influence on the quality of the leachate produced (Andreottola and Cannas 1992) and are discussed in more detail in Section 2.4.

One of the main causes of the generation of leachate is the amount of moisture that enters the waste, so the goal of environmental legislation has been to limit that amount of moisture that can enter the waste and therefore limit leachate pollution. These legislative directives have caused the landfills to be designed, sited, operated and managed to encapsulate the waste. While this has assisted in providing a short-term solution to the uncontrolled release of leachate, it has created long-term environmental problems.

As with most polluting activities, it is vastly more effective to prevent pollution from leachate than it is to take remedial actions even if they are possible (Hammer and Hammer 2001). Since preventing pollution begins with an understanding of both the quantity and quality of landfill leachate (Qasim and Chiang 1994), these processes will be addressed in the following sections. Various options for waste management in the landfill and leachate treatment will also be included because they too are also critical to the protection of natural resources (Hammer and Hammer 2001).

2.2 Leachate Generation

The hydrology around and within the landfill is an important consideration when discussing leachate generation and subsequent collection and treatment. Leachate is the result of the net surplus of water in the landfill exceeding the moisture storage capacity of the soil and of the waste (Blakely 1992, Blight *et al* 1992; Qasim and Chiang 1994; Knox 2000). The production of leachate depends on the balance between the amount of moisture entering and leaving the landfill and the absorptive capacity of the waste. In general there is an initial delay in leachate generation until the absorptive capacity of the waste is saturated; once significant leachate production begins, the moisture content in the landfill remains fairly constant (Qasim and Chiang 1994).

There are several factors that are related to the water budget in a landfill. Inputs include precipitation, surface run-on, groundwater discharge, liquid in the waste, leachate recycling, and irrigation water. Common outputs are evaporation, transpiration, surface run-off, surface seepage, groundwater recharge, leachate abstraction, landfill gas removal, and waste formation. These components are illustrated in Figure 2.2.1.

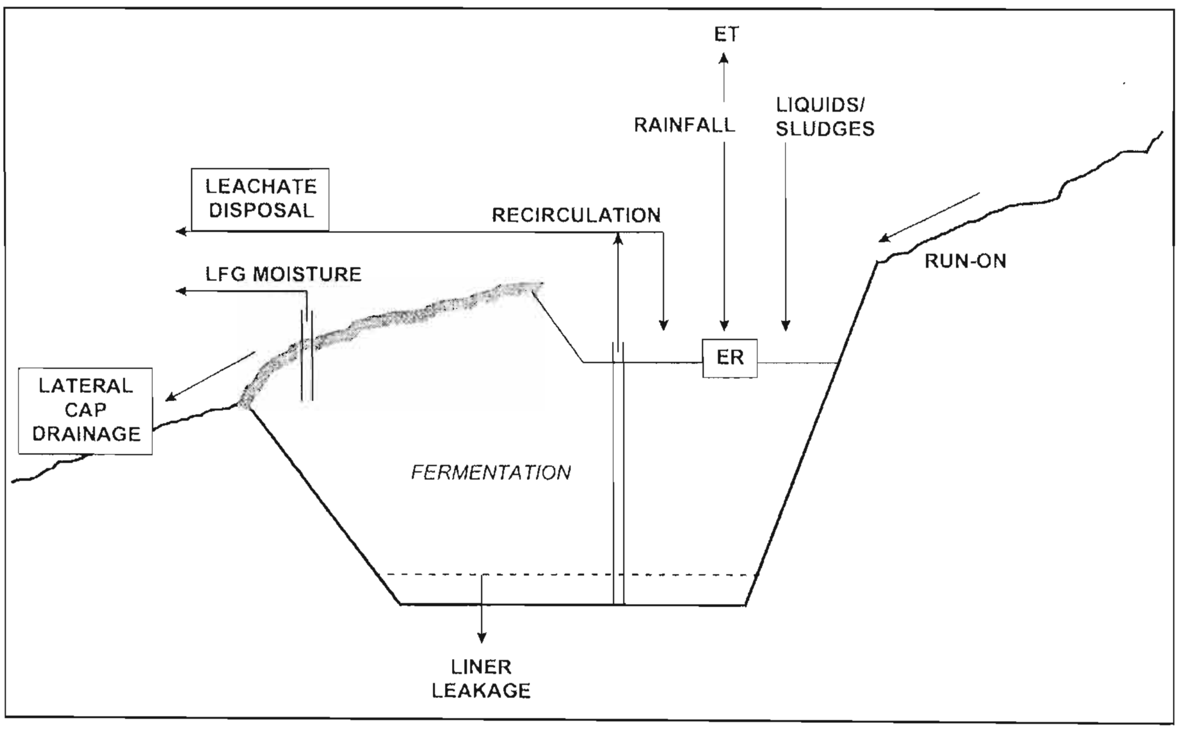

Figure 2.2.1: Schematic landfill cross-section, showing typical components of a water budget of a landfill (Knox 2000)

Most leachate can be attributed to local climatic conditions and the geology and hydrology at the landfill site. Depending on the type of landfill and operational practices, a small fraction of the leachate will be produced from the waste itself.

2.2.1 Climatic conditions

Precipitation has the greatest impact on leachate generation, but its impact can vary depending on the percentage of uncovered areas, the degree of impermeability of caps and liners, the compaction of the waste, which affects the hydraulic conductivity and the porosity of the waste (Qasim and Chiang 1994).

Evaporation is the other climatic factor that influences leachate production (Ball and Blight 1986; Blight *et al* 1987). For this reason leachate is not typically produced in arid regions. Research has shown that evaporation can decrease the moisture of the landfill to a depth of five meters (Fourie and Blight 1995). Theoretically leachate can only occur if precipitation exceeds evaporation long enough during the year for the field capacity of the waste to be surpassed. Field capacity is the amount of moisture that the waste body can hold under fully saturated conditions (Qasim and Chiang 1994). Considering the heterogeneous nature of waste and the varying permeability of the waste body, it is likely that leachate will be produced before the field capacity of the entire site has been exceeded (Canziani *et al* 1989; Blight *et al* 1992). In landfills where precipitation exceeds evaporation during the rainy seasons, leachate would be generated seasonally by infiltration and percolation of precipitation (Blight *et al* 1992). While in the arid areas of South Africa, where soil evaporation exceeds precipitation, leachate generation will be restricted to only extremely wet seasons (Ball 1984).

2.2.2 Geohydrology

Groundwater can enter the waste adding to the inflow of moisture into the waste (Couth 2000). This occurs in landfills that were placed in excavations that intersect the water table or in ones situated in wetlands. This is known as "wet tipping" or Class 3 landfilling in South Africa. Leachate would be generated due to the continuous contact between the wastes and the surrounding water (Ball 2002). This can be avoided by proper siting of the landfill in areas with suitable geology and hydraulic conditions.

2.2.3 Hydraulic Properties and Self-Generating Factors

Three factors within the waste are responsible for the quantity of leachate generated: the type of waste, the field capacity of the landfill and the moisture within the waste. The moisture in household waste aids the biochemical processes involved in the degradation of the wastes. The moisture is also released through compaction by machinary on the landfill and the pressure from the layers of waste (Qasim and Chiang 1994). Landfills can also generate leachate due to bad drainage control or co-disposal of liquid wastes (Ball 2002).

2.2.4 Water Balance and Leachate Generation

The net surplus of water needed to generate leachate can be predicted by estimating the quantities of inputs and outputs of moisture and then calculating the total water balance at a site. This information can then be used to design new landfills and new cells in existing landfills, to interpret and to evaluate leachate levels and flow rates (Knox 2000). In the case of South Africa, it determines the legislative requirements to which the landfill must adhere (Ball 2002). The previously described parameters involved in the generation of leachate are site and climate specific; so, theoretical values presented in literature do not always reflect the actual field values (Couth 2000; Griffith and Trois 2002). There is also difficulty and uncertainty involved with estimating some of these parameters. Despite the possibly inaccuracy of these calculations should still be evaluated due to the importance and costs of leachate management (Knox 2000).

2.3 The Effect of Waste Decomposition on Leachate

Before a type of treatment can be chosen, the variability and quality of the leachate must be evaluated and a basic understanding reached (Robinson 1995b; Robinson and Gronow 1995). One of the difficulties of establishing a long-term treatment process for landfill leachate is related to the fact that the nature and concentrations of pollutants change as the material in the landfill is physically, chemically and biologically degraded. Because of the biodegradation processes that occur, some researchers refer to the waste body as a bioreactor (Christensen and Kjeldsen 1989; Robinson 1995b).

The decomposition of wastes in a landfill has been the subject of numerous studies. Some have described the degradation in three phases: aerobic phase, anaerobic phase and anaerobic degradation (Qasim and Chiang 1994 and Tchobanoglous 1993). Others (Farquhar and Rovers 1973, Ehrig 1984 and Christensen and Kjeldsen 1989) have characterized leachate according to five distinct phases of biodegradation; they have delineated a transitional phase between initial acetogenic anaerobic phase and the methanogenic anaerobic phase, and also have added a final aerobic phase. The following is a brief description of the phases of biodecomposition of landfilled wastes:

- Phase 1 – Brief, initial aerobic degradation
- Phase 2 – Anaerobic conditions, acetogenic bacteria degradation
- Phase 3 – Anaerobic, transition phase
- Phase 4 – Anaerobic methanogenic degradation

- Phase 5 – Final aerobic diffusion

Figure 2.3.1 shows the theoretical changes in landfill gas and landfill leachate over time and is described in more detail within the explanation of the various phases. The top graph (a) is from Farquhar and Rovers (1973) and it demonstrates the changes in biogas through each phase. The middle graph (b) is from Ehrig (1984) and the bottom (c) is from Christensen and Kjeldsen (1989). Both graph (b) and (c) demonstrate the chemical changes in leachate during each phase. There are notable changes in the leachate characteristics during each one of the five phases; therefore all five phases will be discussed.

2.3.1 Aerobic Degradation Phase (Phase 1)

There is a brief initial decomposition of the waste via aerobic biological processes. It is a short phase of a few days due to the limited supply oxygen and the high BOD of the waste (Qasim and Chiang 1994). This leads to a rapid onset of an anaerobic environment (Crawford and Smith 1985, Robinson 1995a). During this phase there are several important biochemical reactions that occur:

- Proteins are degraded into amino acids and as a result of this typical aerobic process, in which carbon dioxide, water, nitrates and sulphates are formed.
- Carbohydrates are transformed into carbon dioxide and water.
- Fats are hydrolyzed first to fatty acids and glycerol then into simple catabolites through the formation of volatile fatty acids (VFA) and alkalis (Andreottola and Cannas 1992).
- Cellulose is initially degraded into glucose, which is consumed by bacteria and transformed into carbon dioxide and water.

Other notable observations during this phase:

- Temperature of the fill may rise to 70-90°C due to the exothermic nature of the biological reactions (Andreottola and Cannas 1992).
- Large quantities of hydrogen (up to 20% by volume) can be generated during this phase (Robinson 1995a).
- No substantial leachate generation will occur due to the brevity of the phase not allowing time for the absorptive characteristics of the waste to be exceeded (Andreottola and Cannas 1992).

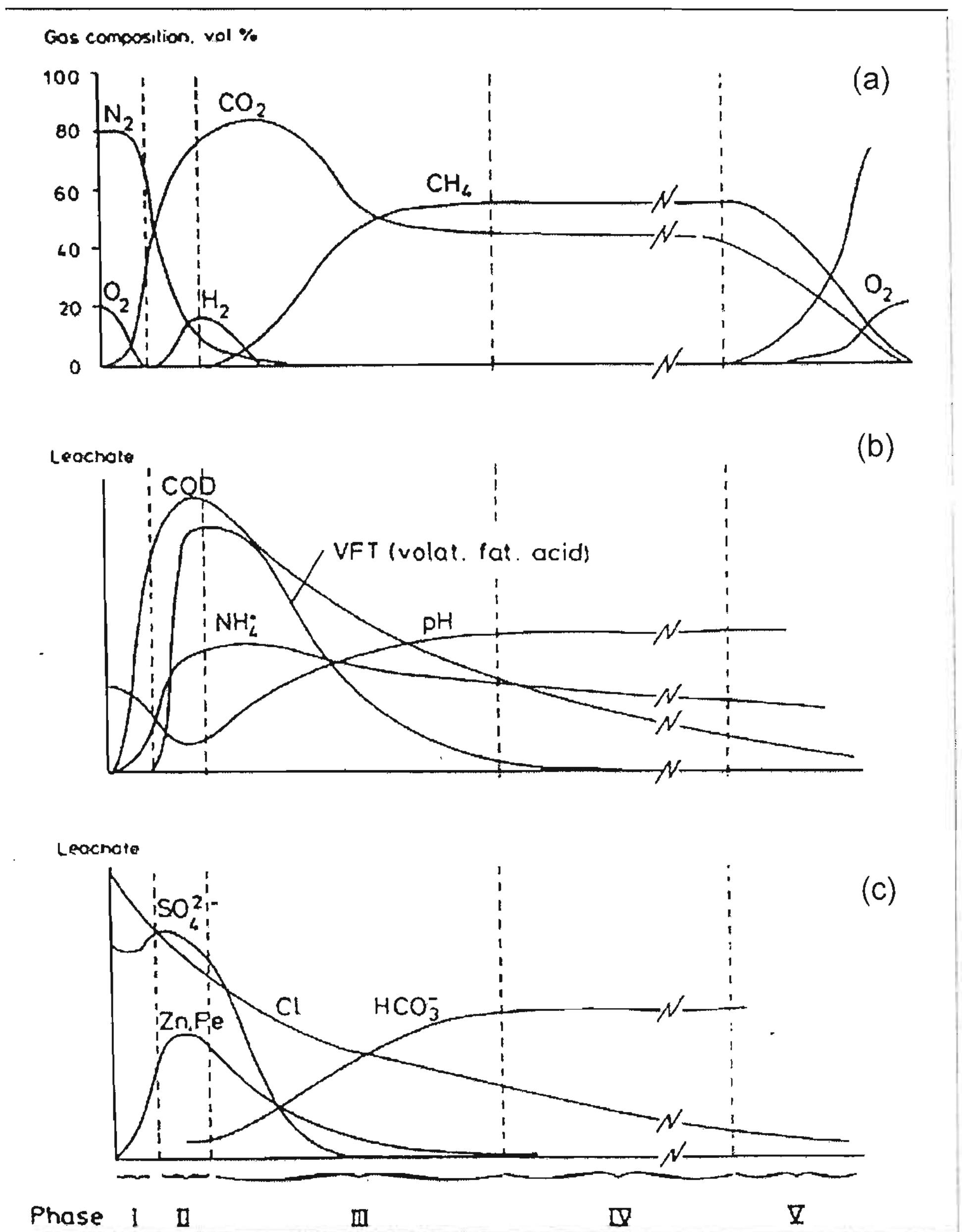

Figure 2.3.1: Changes in leachate and biogas over time through the five stages of decomposition. The top graph (a) is from Farquhar and Rovers (1973). The middle graph (b) is from Ehrig (1984) and the bottom (c) is from Christensen and Kjelden (1989). Compiled by Christensen *et al* (1992).

2.3.2 Acetogenic Phase (Phase II)

The first of the three anaerobic degradation phases is when acetogenic bacteria develop, which results in acid-fermentation. There are two basic processes in this phase of anaerobic decomposition. The first one involves the biodegradation of complex organic materials into simpler organics such as acetic acid (CH_3COOH), proponic acid (C_2H_5COOH) and pyruvic acid ($CH_3COCOOH$) or other simple organics and acids (Crawford and Smith 1985).

Leachate during this phase is referred to as a "young" leachate. As presented in Figure 2.3.1, the following is some of characteristics of the leachate during this phase:

- The pH decreases to around 4-5 because of the high production of volatile fatty acids such as acetic acid (Qasim and Chiang 1994) and the high partial pressure of CO_2.
- There are high concentrations of volatile acids and inorganic ions such as Cl^-, SO_4^{2-}, Ca^{2+}, Mg^{2+}, and Na^+.
- There is an increase in readily biodegradable soluble organics as seen by the high BOD_5:COD ratios (commonly 0.7 and greater) (Andreottola and Cannas 1994)
- Ammonia levels rise significantly to between 500-1000 mg/liter (Robinson 1989).
- There may be an increase in heavy metals and other constituents that become soluble in a reduced environment (Andreottola and Cannas 1992).

Typical gas emissions during this phase are characterized by the reduction of nitrogen and oxygen and an increase in both carbon dioxide and hydrogen (Farquhar and Rovers 1973). There are often only traces of methane released during this phase, but the most noticeable characteristic is the generation of sulphides, which produces a rather unpleasant odor (Qasim and Chiang 1994).

2.3.3 Anaerobic Transitory Phase (Phase III)

This phase is essentially the transitory phase between the acetogenic conditions in Phase II to the more stable methanogenic conditions found in Phase IV. The transition from acetogenic conditions into methanogenic conditions can be rapid (taking months to form) but typically the formation takes several years (Robinson 1995a). This phase is characterized by the formation of methanogenic bacteria. These bacteria consume soluble organic compounds (VFA) and convert them into carbon dioxide (CO_2) and methane gas (CH_4). The growth of these bacteria may be restricted if there is a high concentration of volatile acids produced in Phase II; these acids are toxic to the bacteria at concentrations of 6,000-16,000 mg/liter (Stegmann and Spendlin 1989). Bacterial concentrations will also be restricted by the low pH conditions formed during Phase II. Methanogenic bacteria can only tolerate a neutral pH conditions of between 6.6 to 7.3 (Qasim and Chiang 1994).

As the populations of methanogenic bacteria slowly increase, they will cause a steady increase in the methane concentration and subsequently a proportionate reduction in hydrogen, carbon dioxide and volatile fatty acids (Andreottola and Cannas 1992). The conversion of fatty acids causes an increase in alkalinity, which is reflected in an increase in pH values. This more alkaline environment results in a decrease in the calcium, iron, manganese and heavy metals in the leachate (Andreottola and Cannas 1992). Ammonia production reaches its peak during this phase but continues to be produced as long as the leachate is generated (Robinson 1995).

2.3.4 Methanogenic Phase (Phase IV)

During this phase a dynamic equilibrium is reached between acetogenic and methanogenic bacteria (Robinson 1995). This is reflected in the fairly stable production of methane (Farquhar and Rovers 1973) and the consistency of the leachate constituents (Christensen and Kjeldsen 1989). The methanogenic bacteria utilize the volatile acids that were the end products from the first stage of anaerobic decomposition. The volatile acids along with other organic matter are converted to methane and carbon dioxide (Qasim and Chiang 1994). As a result of the rise in pH to neutral levels, the methanogenic bacteria population increases resulting in more methane being produced. The following are some of the characteristics of the leachate during this phase:

- The pH becomes neutral or slightly alkaline.
- There is a decline in metal concentrations (Qasim and Chiang 1994).
- The concentrations of volatile acids decrease.
- Conductivity decreases, since inorganic materials are not as soluble at neutral pH.
- By this phase the majority of organics have been degraded resulting in a lower BOD to COD ratios.
- Ammonia concentrations remain constant (Andreottola and Cannas 1992).

Although this phase is the most biologically active phase, the leachate is often referred to in the literature as "stabilized": implying that the constituents in the leachate remain at fairly constant concentrations throughout this phase. This phase is of particular importance when examining the pollution potential of a landfill leachate because the phase lasts longer than the others (Robinson 1989). The methanogenic phase will continue until the organic substrate is depleted (Röhrs and Fourie 2002).

2.3.5 Final Aerobic Phase (Phase V)

Over time, the decomposable substrate in the landfill will begin to decrease, consequently reducing the bacteria populations. The decomposition will continue as long as there is some organic material that can be used as substrate for the bacteria (Qasim and Chiang 1994). This will cause the methane production to decrease to a level so low that air will diffuse from the atmosphere causing redox potentials to rise and thus not allowing further methane production (Christensen and Kjeldsen 1989). Aerobic areas may also form in the landfill due to the oxygenated water that will continue to infiltrate into the waste. As with the first aerobic phase, this second aerobic phase will appear only in the upper layer of the landfill. Leachate from this phase will continue to have relatively low BOD values and low BOD to COD ratios (Robinson 1995a). As the material in the landfill continues to be degraded, ammonia concentrations will remain high. Other inorganics such as iron, sodium, potassium, sulphates, and chloride may continue to be found in the leachate (Robinson 1995a; Robinson and Gronow 1995).

2.3.6 General Leachate Characteristics

The phase of degradation of the majority of the waste can be determined by examining the composition of the leachate and the production of methane. The differences between an acetogenic leachate and a methanogenic leachate can be seen in Table 2.3.1. In general Phase II (acetogenic phase) has a lower pH and a higher concentration of organic compounds and tends to have the highest pollutant concentrations (Lu *et al* 1984, Andreottola and Cannas 1992). Whereas Phases III & IV (methanogenic phase) have a more neutral pH and a lower concentration of organic compounds (lower BOD:COD ratio). However ammonia and chloride concentrations depend on dilution over time, not the phase of decay (Röhrs and Fourie 2002).

Table 2.3.1: General characteristics of acetogenic and methanogenic leachates. Modified from Couth (2000).

Determinand	**Acetogenic**		**Methanogenic**	
	Range	**Mean**	**Range**	**Mean**
pH value	5.1 - 7.8	6.7	6.8 - 8.2	7.5
BOD_5	2000 – 68,000	20,000	97 - 1770	375
BOD:COD	1.0 - 0.6	0.8	0.6 – 0.1	0.3
Ammonia	194 - 3610	900	283-2040	900
Chloride	659 - 4670	1800	570 - 4710	2000

Note: all figures in mg/liter except pH, which is unitless

It is important to note that although the phases of biodecomposition are fairly well understood, the steps and parameters which are responsible for the transition rate from an acetogenic to a methanogenic phase are not (Robinson 1995a). The phases described are theoretical and the potential pollutants will vary. Also biodegradation in the landfill may not be the same throughout the fill: different parts of the landfill and/or different cells within the landfill could be at different stages and thus the leachate emanating from the cells could still contain constitutes of both phases (e.g. could contain low pH but be producing methane) (Crawford and Smith 1985). The decomposition phases a landfill undergoes or could potentially undergo are fairly universally agreed upon, but there is inconclusive evidence regarding length of time required for each phase and for complete decomposition to occur (Robinson 1995b). The combination of the dearth of knowledge regarding the rate of the transitions between phases and the inherent variations of potential pollutants makes designing landfill leachate control systems difficult (Robinson 1995a; Robinson and Gronow 1995).

2.4 Factors that Affect Leachate Composition

As discussed in Section 2.3, the degradation phase the landfill is undergoing plays a crucial role in the amount and type of leachate produced. The types of waste allowed in the landfill and how the landfill is managed also effects the leachate composition. These practices directly influence the composition of the leachate and alter moisture and nutrient availability, which affect the time required for each phase.

2.4.1 Factors that effect degradation

Most literature associates certain types of leachates with the age of the landfill. This age could be from the time the waste was first placed in the landfill or from when leachate first appears. Equating the type of leachate with the age of the landfill is not entirely accurate for the composition of the leachate is dependant on the phase of degradation (the degree to which the waste has stabilized) rather than for how long it has been there. Since the phase of degradation is the main determining factor in leachate composition, the parameters that influence the degradation must be reviewed.

The rate at which these phases occur is highly dependent on several environmental factors such as:

- Oxygen availability: At the surface of the landfill (less than one meter) there will always be an aerobic zone as oxygen continues to diffuse into the landfill. The remainder of the landfill will be anaerobic which is essential for most of the degradation to occur, since certain microbes, such as the methanogenic bacteria, responsible for degrading waste, thrive only in an anaerobic environment (Christensen and Kjeldsen 1989).

- Moisture and leachate. As described in Section 2.2, climatic conditions of the landfill directly affect the amount of leachate being emitted for leachate will only occur when the moisture capacity of the landfill has been exceeded (Hammer and Hammer 2001). Irrigation of the landfill will have similar results (Blakey 1997). This increase in the moisture content limits the depth at which oxygen can diffuse and aids the interactions between microorganisms and the landfill environment (Christensen and Kjeldsen 1989).
- Water movement. Preferential water flows within the landfill will cause those areas that have contact with water to degrade more rapidly than the moisture deficient areas. This could leave sections of the landfill in varying phases of decomposition and restrict the spread of microorganisms and other constituents between waste microenvironments (Christensen and Kjeldsen 1989).

In general, the climatic conditions have the most influence in decomposition, since the leachate will be generated when precipitation exceeds evaporation. This can be observed in landfills in water sufficient areas. In a humid climate landfills can reach methanogenic conditions in three to nine months (Ham 1988; Bowers 1999), but in a dry climate a landfill may never have enough moisture to degrade the waste sufficiently to reach that condition (Ham 1988).

2.4.2 Landfill Management's Effect on Leachate Composition

The following landfill practices affect both the leachate composition directly and influence rate of decomposition of the waste:

- Degree of compaction. The depth at which oxygen can diffuse into the waste body is directly related to the degree of compaction of the waste (Qasim and Chiang 1994).
- Processed waste – Results from experiments studying the leachate from shredded processed refuse indicate that this leachate has a significantly higher concentration of pollutants than from a landfill with non-shredded waste. Baled refuse has shown the opposite results having a more diluted leachate and longer period of stabilization. These differences are only short-term, for eventually the cumulative mass of pollutant removed will be the same (Lu *et al* 1984).
- Depth of refuse – In general the deeper the fill, the greater concentration of constituents in the leachate (Qasim and Burchinal 1970a, 1970b), but these deeper

landfills require more water and more time to stabilize. This results in a longer polluting life of the wastes (Qasim and Chiang 1994)

- Codisposal with sewage sludge –The introduction of sewage sludge may accelerate leachate generation, biological stabilization (Pohland 1975) and methanogenic activity (Emcon Associates 1974). Sewage sludge, especially from industrial areas has high concentrations of heavy metals. This may cause a dramatic increase in heavy metals in the leachate, since landfill leachate generally contains only low concentrations of heavy metals (Robinson and Gronow 1998). However the only notable increases are in acids, BOD, nitrate, and enteric pathogens (Lu *et al* 1994).
- Codisposal with hazardous waste- Including hazardous waste in the landfill will not only effect the types of pollutants that will be contained in the leachate but also may have toxic effects to the bacteria in the landfill which could retard or inhibit the biological waste degradation processes. The investigations by Pohland *et al* (1990) suggest that that landfills possess a limited and finite capacity to attenuate both organic and inorganic hazardous waste constituents.
- Codisposal with sorbitive wastes – Sorbitive waste (such as incinerator ash, fly ash, kin dust, or limestone) fills voids in the waste reducing the amount of trapped oxygen and improving the absorptive capacity of the waste (Röhrs *et al* 2001). Research has shown that there is a reduction in the concentrations of many hazardous constituents in the leachate when sorbitive material has been disposed with municipal waste (Fuller 1978; Chen and Eichenberger 1981). The presence of ash has also been found to enhance degradation by increasing the waste pH (Shamrock 1998).
- Use of cover material – Type and depth of cover material will affect both the air diffusion into the fill and the rate of percolation (Qasim and Chiang 1994)
- Composition of solid waste. The percentage and type of organic waste in the fill has considerable influence on the degradation of the waste and therefore on the quality of the leachate (Andreottola and Cannas 1992; Robinson 1995a; Mathlener 2001)

2.5 Constituents in Leachate and Their Environmental and Health Impacts

As discussed in the previous sections, the quality and quantity of leachate will vary between landfills and over the lifetime of an individual landfill. Table 2.5.1 summarizes of the range of chemicals that are found in municipal solid waste (MSW) leachates. This section describes those chemical constituents that are of interest in this study and their associated human and environmental health effects.

Table 2.5.1: Chemical composition of MSW landfill leachates. A Summary of the results of sampling 80 landfill leachates. Compiled by Strachan (1999). Based on studies conducted by Ehrig (1989), Andreottola *et al* (1990), Robinson and Gronow (1995, 1998), and Durban Solid Waste (DSW) (1997/1998).

Determinand	Units	Range
COD (Chemical Oxygen Demand)	mg/liter	150-152,000
BOD_5 (Biological Oxygen Demand)	mg/liter	100 –0 90,000
pH -value	-	5.1 – 8.5
Alk. (alkalinity)	$mgCaCo_3$/liter	300 – 16,000
Hardness	$mgCaCo_3$/liter	500 – 8900
Cl (chloride)	mg/liter	30 – 5000
Conductivity	mS/liter	38 – 5200
NH_4 (ionized ammonia)	mg/liter	1 – 4110
N_{org} (organic nitrogen)	mg/liter	1 – 2000
TKN (N_{tot}) (Total Kjeldhal Nitrogen)	mg/liter	50 – 5000
NO_3 (nitrate)	mg/liter	0.1 – 50
NO_2 (nitrite)	mg/liter	0 – 25
Fatty acids (as carbon)	mgC/liter	1 – 22,500
P_{tot} (total phosphorus)	mg/liter	0.1 - 30
PO_4 (phosphate)	mg/liter	0.3 - 25
Ca (calcium)	mg/liter	10 – 6250
Mg (magnesium)	mg/liter	25 – 1150
Na (sodium)	mg/liter	50 – 4000
K (potassium)	mg/liter	10 – 3100
SO_4 (sulphate)	mg/liter	0 – 1600
Fe (iron)	mg/liter	0.4 - 2300
Zn (zinc)	mg/liter	0.03 - 170
Mn (manganese)	mg/liter	0.04 - 165
Cn (cyanide)	mg/liter	0.04 - 120
AoX (absorbable organic halogen)	ugCl/liter	320 - 3500
Phenol	mg/liter	0.04 –44
As (arsenic)	mg/liter	5 – 1600
Cd (cadmium)	mg/liter	0.5 – 140
Co (cobolt)	mg/liter	4 – 950
Ni (nickel)	mg/liter	20 – 2050
Pb (lead)	mg/liter	8 – 1900
Cr (chromium)	mg/liter	30 – 1600
Cu (copper)	mg/liter	4 – 1400
Hg (mercury)	mg/liter	0.1 – 50
Heavy metals *	mg/liter	0.15 - 2.80

Notes: a. *represents the sum of concentrations of chromium, nickel, copper, cadmium, lead, arsenic and mercury

b. The data provided refers to leachates sampled from municipal solid waste (MSW) type landfills that accept domestic household waste

2.5.1 Nitrogen

Nitrogen compounds found in leachate are typically ammonium ion (NH_4^+), free ammonia (NH_3), nitrite (NO_2^-), nitrate (NO_3^-), and organic nitrogen (N_{org}). This section describes nitrogen in the environment while the following sections detail how the constituents are released from the waste, their impact on the receiving environment, and the range of concentrations that are found in leachate.

In the environment, nitrogen can be found in several chemical forms. It is most commonly found as nitrogen gas (N_2) in the atmosphere, nitrate (NO_3^-) in soils and in water and organic nitrogen (N_{org}) in biota. Molecular nitrogen (N_2) is the most common species (78% by volume) in the atmosphere but due to its relatively low reactivity, it is metabolically unavailable directly to higher plants or animals (Howard 1998; Tortora *et al* 1989). Nitrogen gas must first be microbially converted into ammonia by the process of nitrogen fixation (Howard 1998). Once transformed into ammonia it may be used directly or further transformed.

Aerobic conditions must exist in order for ammonia to be oxidized into nitrite or nitrate. This process, known as nitrification, occurs when nitrifying bacteria (such as *Nitrosomonas* and *Nitrobacter*) use ammonia as the sole source of energy. The following is the reaction equation (Equation 2.1) for the oxidation of NH_4^+ (ammonium ion) to nitrite (NO_2^-) by the autrotophic bacterium *Nitrosomonas* (Johnson and Schroepfer 1964):

$$2NH_4^+ + 3O_2 \xrightarrow[\textit{Nitrosomonas}]{} 4H^+ + 2H_2O + 2NO_2^- \qquad (2.1)$$

Nitrite (NO_2^-) is chemically unstable and easily oxidized into nitrate (NO_3^-). The following is the reaction equation (equation 2.2) for the oxidation of nitrite (NO_2^-) to nitrate (NO_3^-) by the autrotophic bacterium *Nitrobacter* (Johnson and Schroepfer 1964):

$$2NO_2^- + O_2 \xrightarrow[\textit{Nitrobacter}]{} 2NO_3^- \qquad (2.2)$$

This process is critical to the removal of ammonia from leachate and will be discussed further in Section 4.5.2, which describes the removal of ammonia in constructed wetlands.

Another important microbial process relied on to remove nitrogen is denitrification, which involves the reduction of nitrate to nitrite, nitric oxide (NO), nitrous oxide (N_2O) or nitrogen gas (N_2). Denitrification requires an anaerobic environment, an external carbon source, and the mediation of denitrifying bacteria. The most important group of these denitrifiers are the *Pseudomona* species but other genera, including *Paracoccus*, *Thiobacillus* and *Bacillus* are also capable of reducing nitrates to nitrogen gas. The most commonly used external carbon source is methanol. When methanol is added, the denitrification reaction is

$$6NO_3^- + 5CH_3OH \rightarrow 3N_2 + 5\,CO_2 + 7H_2O + 6OH^- \qquad (2.3)$$

This is a fairly expensive process considering the quantities of methanol needed for the denitrification of the thousands of liters of leachate produced, (Peavy *et al* 1985).

2.5.2 Ammonia

Ammonia is formed from one of the microbial processes of organic waste decomposition. Proteins and fats in animal and vegetable matter are broken down into soluble sugars, long-chain fatty-acids, glycerol and amino acids (Tortora *et al* 1989). The amino groups associated with the amino acids are further transformed by bacteria into reduced forms of inorganic nitrogen: ammonium ion (NH_4^+ or ionized ammonia) and ammonia (NH_3 or unionized/free ammonia) depending on temperature and pH. Free ammonia (NH_3) is a gaseous chemical, whereas the ionized form (NH_4^+) remains soluble in water. The following equilibrium equation shows the relationship between the two forms of aqueous ammonia:

$$NH_3 + H_2O \leftrightarrow NH_4OH \leftrightarrow NH_4^+ + OH^- \qquad (2.4)$$

The relationship between the two is based significantly on the pH of the solution. Figure 2.5.2 demonstrates the relative percentages of ammonium ion (NH_4^+) and ammonia (NH_3) at varying pH values and temperatures.

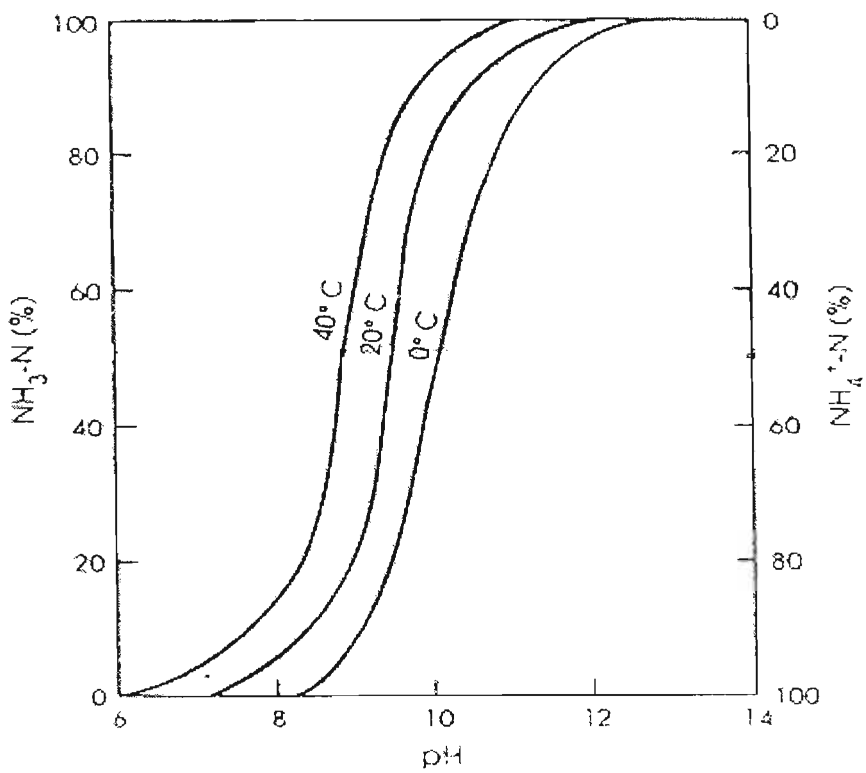

Figure 2.5.2: The effects of pH and water temperature on the fraction of total ammonia in the unionized and ionized forms (Kadlec and Knight 1996).

The graph shows that under acidic conditions (pH < 6) NH_4^+ ions only would exist, but as the solution becomes more basic, NH_3 becomes increasingly predominate. Temperature is also a factor. As with all gases, the higher the temperature in the solution the less gas dissolved. If the pH rises to 12 or higher, the solution solely contains ammonia (NH_3) as a dissolved gas (Reed *et al* 1995). Although considering that the average pH of a landfill is typically neutral, NH_4^+ ions will be the predominant species in the leachate.

Concentrations of ammonia in leachate can range from 1 to 3610 mg/liter (Andreottola and Cannas 1992; Robinson 1995a). This is of particular concern in leachate because ammonia cannot be further degraded in the landfill due to the anaerobic conditions (Novella *et al* 1998). Some nitrogen may be transformed in the leachate as nitrates/nitrites but no experimental evidence has demonstrated the loss of ammonia as nitrogen gas in landfills (Burton and Watson-Craik 1998). Therefore ammonia will remain in large concentrations in the leachate, which needs to be considered when considering the treatment of leachate.

Ammonia causes environmental problems because of its high oxygen demand and its toxicity to aquatic life (Crawford and Smith 1985). It is readily oxidized by chemolithoautotrophs, leading to a depletion of dissolved oxygen concentration (otherwise known as a nitrogenous oxygen demand, or NOD) to a level that causes stress on aquatic life. In natural waters this causes a lowering of the dissolved oxygen concentrations by +/- 4.6 grams O_2 per gram NH_4^+. This also affects the pH and in order for the pH to remain constant, 7.1 mg of alkalinity will be needed to neutralize the acid produced (Peavy *et al* 1985).

The unionized (free) ammonia form (NH_3) is considerably more toxic to organisms such as fish, so therefore considerablly more attention is usually given to the relative concentration of this particular contaminant. At concentrations ranging from 0.53 to 22.8 mg/liter, NH_3 is toxic to most aquatic species (especially fish). Fish rely on the concentration gradient of ammonia between the gills and the external water. This typically high concentration gradient allows fish to release ammonia through their gills. As the concentration of ammonia increases in the environment, the magnitude of the gradient decreases causing the fish to fail to release ammonia. When this occurs, the concentration of NH_3 in the blood rises, leading to stressful and eventually to lethal levels. This can affect the hatching and growth rates of fish, and cause changes in the structural development of tissues in the gills, liver, and kidneys. Although ammonia concentrations primarily and most directly affect fish, they can be devastating to an entire aquatic system. In general plants are more tolerant of ammonia than animals, and invertebrates are more tolerant than fish, but by upsetting the fish population the entire ecosystem can be affected. In humans, toxic concentrations of ammonia may cause loss of equilibrium, convulsions, coma, and even death (Reed *et al* 1995, Holmes 1996; Kadlec and Knight 1996, Hammer and Hammer 2001).

2.5.3 Nitrate/Nitrite

Nitrogen can also be released from landfills in the forms of nitrite (NO_2^-) or nitrate (NO_3^-), but these only make up a small percentage of the total nitrogen (TKN) in leachate: nitrite (NO_2^-) can range from 0 to 25 mg/liter and nitrate (NO_3^-) from 0.1 to 50 mg/liter (Andreottola and Cannas 1992; Qasim & Chiang 1994; and Robinson 1995a).

The main human health hazard associated with excessive consumption of nitrates in drinking water is infant methemoglobinemia, but occurrences of this disease are now rare (Hammer and Hammer 2001). The only other concern is the formation of nitrous acid from nitrite in acidic solution, which may react with secondary amines to produce nitrosamines. Nitrosamines are known to be carcinogens and toxic to mammals (Clesceri *et al* 1989).

The major environmental problem associated with nitrate (NO_3^-) occurs when excessive concentrations are released into surface waters. This over-enrichment of nutrients leads to eutrophication, which results in an oxygen depletion of the aquatic system due to excessive algae production. Other unpleasant effects of eutrophication are taste and odor problems (Peavy *et al* 1985).

2.5.4 Organic Nitrogen

Organic nitrogen (N_{org}) is nitrogen that is bound to carbon in biota. Specifically it is found in amino acids, which make up proteins, and it also occurs in the nucleotides of nucleic acids (Howard 1998). In leachates, the concentration of organic nitrogen can range from 1 to 2000 mg/liter (Andreottola and Cannas 1992). Organic nitrogen itself has no impact on the receiving environment, but as it is broken down by digestion into urea and ammonia (Howard 1998), it will impact the environment in the same manner as ammonia.

2.5.5 Organics

There are several classes of organic compounds that have been identified in landfill leachate. Lu *et al* (1984) classified them into three general categories:

- Fatty acids of low molecular weight;
- Humic acids: carbohydrate-like substances of high (Howard 1998) and intermediate molecular weight ; and
- Fulvic-like substances of intermediate molecular weight

Fatty acids can be simply characterized as being the biodegradable portion of the organics and the humic and fulvic acids, due to their nature of being complex mixtures of polymeric material with high molecular weight (above 300 Dalton), are considered the less or non-biodegradable organics. Humic and fulvic acids can be further divided by the conditions under which the components can be extracted (Howard 1998):

- Humic acids are compounds which are alkali soluble and are precipitated by acid, and
- Fulvic acids are lower molecular weight compounds which are soluble at all pH levels.

The biodegradable fraction of the organics is measured by the biological oxygen demand (BOD) while the non-biodegradable fraction is measured in terms of the difference between BOD and its chemical oxygen demand (COD), which is the sum of both the biodegradable and the non-biodegradable fractions (Crawford and Smith 1985). In leachate the BOD can range from 100 to 90,000 mg/liter and since COD includes all organics it is larger: ranging from 150 to 150,000 mg/liter (Andreottola and Cannas 1992).

As a landfill ages (i.e. goes through the phases of degradation) the proportions of organic components change. The readily degradable volatile fatty acids will be removed first and therefore, their concentration in the leachate will decrease. This will cause a continual increase in lesser degradable fulvic-like fractions (Lu *et al* 1984). For example in young landfills the BOD to COD ratios can reach values of 0.58, but by the time the methanogenic phase is reached, the ratio has been reduced to around 0.06 (Ehrig 1989). In addition, investigations by Andreottola and Cannas (1992) have shown that the hydroxyl aromatic compounds in the humic and fulvic-like fractions may decrease slightly but will constitute the largest portion of the total organic carbon found in older landfills.

This temporal change of the proportion of biodegradable organics found in leachate is directly related to the pollution potential of the leachate. The majority of the biodegradable organics will be removed from the waste body during the first three phases of degradation before methanogenic conditions stabilize. The readily biodegradable fatty acids are considered the sole polluting fraction of the organics. By their nature these less biodegradable humic and fulvic acids tend to have little or no impact on the receiving environment. These refractory organics decompose extremely slowly (Tchobanoglous *et al* 1979) thereby not lowering the dissolved oxygen concentration at a rate that will be detrimental to the aquatic environment (Rogers *et al* 1985). Considering this, there is debate surrounding the motivation for removing these substances from the leachate (Christensen *et al* 1998). Therefore the polluting potential of organics in leachate will be more of an issue in young landfills rather than a long-term problem. The main concern with the continual release of humic and fulvic acids is these acids add a residual yellowish-brown color to the leachate (Peavy *et al* 1985).

Unlike the unknown effects of the humic and fulvic acids on ecosystems, the effects of biodegradable organics on aquatic systems are fairly well known. Some of these organics may cause color, taste and odor problems, but the main environmental impact is the depletion of the oxygen level as a result of the action of microorganisms on these substances (Hammer and Hammer 2001; Peavy *et al* 1985). As these organics are released into an aquatic environment containing dissolved oxygen, the organics will be degraded in an aerobic metabolic processes that uses the dissolved oxygen as the terminal electron acceptor. Depending on the quantity of organics released and the amount of available oxygen in the water, anaerobic conditions can develop, which will cause detrimental effects on the ecosystem. Oxygen can be replaced by atmospheric reparation and algal photosynthesis, but the replacement rate is both slow (as in the case of atmospheric reparation) and inefficient (oxygen is a byproduct of photosynthesis, but when it is consumed it depletes the oxygen) (Peavy *et al* 1985). Also in the case of a release of organics into a river or other lotic system, the effects of the pollution may not be seen until further downstream as organics continue to be degraded, as seen in Figure 2.5.5 (Peavy *et al* 1985; Hammer and Hammer 2001).

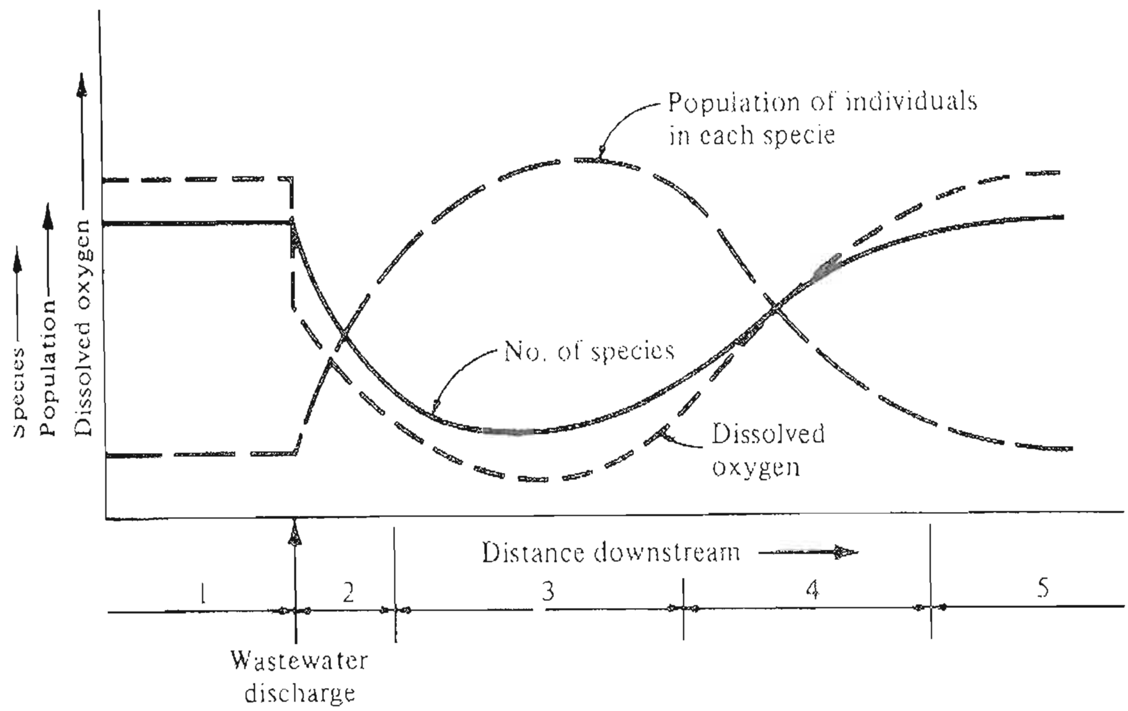

Figure 2.5.5: Ecological response curve, caused by polluted water discharge into a natural receiver (Peavy *et al* 1985).

2.5.6 Phosphorus.

Total phosphorus in landfill leachate ranges from 0.1 to 30 mg/liter (Andreottola and Cannas 1992; Robinson 1995a). Due to its low concentration in leachate, the concentration of phosphorus is typically not of concern, although the levels could be so low as to limit the biological systems being used to treat the leachate (Robinson 1995b).

Phosphorus, like nitrogen, is a vital macronutrient in aquatic ecosystems, and is transformed though a cycle of decomposition and photosynthesis. In aquatic environments the only form of phosphorus that is found is as one of the forms of phosphate (PO_4^{3-}). Phosphates can be found in solution as part of particulate matter, or in tissues of plants or animals (Peavy *et al* 1995). There are no known health risks associated with phosphorus except at incredibly high levels it may cause digestive problems (Howard 1998). The main environmental problem with phosphorus, like nitrogen, is that it can be a limiting macronutrient and thus excessive concentration can lead to eutrophication of surface waters (Reed *et al* 1995).

2.5.7 Alkalinity

There are several sources of alkalinity in leachate: it can come in the form of carbonates (CO_3^{2-}), bicarbonates (HCO_3^-), silicates ($HSiO^{3-}$), borates (H_2BO^{3-}), ammonia (NH_3), organic bases, sulfides (HS^-), and phosphates (PO_4^{3-}). The concentration of alkalinity in young (acetogenic) leachate ranges from 300 to 15,870 mg/liter as calcium carbonate ($CaCO_3$) (Andreottola and Cannas 1992, Robinson 1995a). Once the landfill stabilizes, the concentration decreases almost ten-fold to a range of 200-1000 (Qasim and Chiang 1994).

Alkalinity is important for fish and aquatic life because it protects or buffers against rapid pH changes (Hammer and Hammer 2001). Most organisms, especially aquatic life, function best in a pH range of 6.0 to 9.0. Alkalinity is a measure of how much acid can be added to a liquid without causing a large change in pH. It does not pose a risk to public health and has not yet been considered a cause of pollution (Cole 2001). For that reason the United States (US) Environmental Protection Agency (EPA) has not set a maximum level for alkalinity in wastewater discharge (Peavy *et al* 1995) and neither has it been included in the water quality standards that govern South African aquatic systems (SA Government Gazette 20526 1999).

2.5.8 pH

The concentration of hydrogen ions (the pH) of the leachate is dependant on what phase of degradation the landfill is undergoing. During the acetic phase (Phase II), the leachate will be more acidic: ranging from 4.5 to 7.5. While during the methanogenic phase (Phases III and IV), the leachate produced will be more basic: ranging from 7.5 to 9 (Ehrig 1989, Robinson 1995). Since the pH of water affects the solubility of many toxic and nutritive chemicals, this in turn alters the availability of these substances to aquatic organisms. As pH decreases, metals, cyanide and sulfides become more soluble and therefore more toxic (Andreottola and Cannas 1992; Cole 2001). Within a given pH range, an increase of one pH unit will increase the NH_3 concentration about 10-fold. The pH also effects which type of bacteria can exist in the landfill. Fermentative and acetogenic bacteria can exist in a wider range of pH concentrations than the methanogenic bacteria, which can only function within a neutral (6-8) pH environment (Christensen and Kjeldsen 1989).

2.5.9 Electric Conductivity

Electric conductivity of wastewater is a measure of quantity of ionized materials in a polluted water sample (Peavy *et al* 1985; Kadlec and Knight 1996). The typical range of electrical conductivity in landfill leachate is 1000 to 52000 mS/m (Ham 1988; Robinson 1995a). This measurement is often used to approximate the amount of total dissolved solids (TDS) in wastewater (Peavy *et al* 1985). Conductivity itself is not a human or aquatic health concern but it can highlight a change of the amount of dissolved ions in the water and thereby be an indicator of other water quality problems (Cole 2001).

2.6 Treatment and Management Options for Landfill Leachate

Onsite leachate treatment is now a well-researched and established technology (Robinson 1999), but several issues make choosing an appropriate treatment option difficult and highly site specific. Qasim and Chiang (1994) list several problems inherent with the treatment of landfill leachate:

- The source of leachate is dependant on hydrologic and climatic factors, which may change seasonally and annually.
- The composition of the leachate depends on the types of waste accepted at the landfill and the stage of decomposition of the waste.
- There cannot be direct technology transfers between landfill sites due to the variability between sites.
- The quality and quantity of leachate will fluctuate over both the short and long time intervals.

For these reasons the treatment system chosen must be flexible over the long-term as changes in technology, regulations, leachate characteristics and economics occur. This requires that the method chosen be able to treat the leachate as long as it is being released from the landfill, not just through the active use phase of the landfill. The treatment process chosen should also be appropriate to the proposed area in both financial and knowledge-based resources. (Robinson 1999).

While this dissertation is concerned with evaluating constructed wetlands as a treatment option, there are a wide variety of other treatment and filtration options available for removing contaminants from leachate. The choice of method depends on several technical factors: landfill design, leachate quantity and quality, degree of treatment needed and ultimate disposal methods of effluent and residues. Non-technical factors such as legal issues, regulatory constraints and public participation should also be considered (Qasim and Chiang 1994). Treatment options are based on biological processes, chemical processes, or a combination of both. They range in complexity and cost depending on the requirements that the effluent must meet. These processes are used to treat a variety of types of wastewaters, although most often domestic wastewater. This section briefly describes a few of the more successful methods used to treat landfill leachate. For the theory and design of these processes see Qasim and Chiang (1994), Hammer and Hammer (2001), or Tchobanoglous and Burton (1991). Extensive literature reviews on the effectiveness of the processes on the treatment of landfill leachate have been conducted by Chian and DeWalle (1977) and Kennan *et al* (1983, 1984).

2.6.1 Chemical treatment methods

Chemical treatment processes utilize the addition of chemicals to enhance the removal of contaminants. These processes are used in conjunction with either biological or physical treatment processes (e.g. precipitation requires settling). Common chemical treatment processes are coagulation and precipitation, ozonation, carbon absorption, and chemical oxidation (Qasim and Chiang 1994). Often prior to the main treatment process, the constituents in the leachate may require chemical transformation in order to be further degraded. Chemical (and physical) methods are often used to treat older leachates that have low BOD to COD ratios and a high percentage of refractory organics, which cannot be treated by using biological treatments (Fuellade and Lagier 2001).

2.6.1.1 Coagulation and Flocculation

Some of the simplest treatment methods involve adding chemicals to either destabilize colloidal particles (coagulation) or aggregate small, unsettleable particles (flocculation). These are relatively low-cost and effective processes for the removal of suspended particles. Precipitation and sedimentation are often used in conjunction with these processes. These are physicochemical processes that precipitate soluble substances (precipitation) and remove them from solution (settling) (Qasim and Chiang 1994). The process of lime precipitation has been reported to be the most effective in treating organics with a molecular weight larger than 50,000, which are common in medium-age fills (Chiang and Dewalle 1977). The main disadvantages are that the removal of BOD and COD by this method is relatively small (Pietrelli *et al* 2001) and large quantities of sludge are produced (Qasim and Chiang 1994).

A similar alternative to this process is electrocoagulation, which uses the principle of soluble anodes (iron or aluminum) to generate metallic ions. It successfully removes turbidity and organics, but for some parameter the results are comparable to coagulation/flocculation processes (Feuillade and Lagier 2001).

2.6.1.2 Ozonation

Ozone gas, 0_3, is formed when oxygen is passed through a high voltage electrical field. It is used to oxidize resistant organic materials (such as pesticides) into biodegradable substances, which then can be subsequently treated. This treatment method involves sending the ozone gas through the leachate and then through an ozone destructor before releasing it to the atmosphere. Ozonation has been shown to be successful in degrading the herbicides, mecoprop and isoproturon, into organic materials that could be further degraded (Robinson and Harris 2001).

2.6.1.3 Carbon Absorption

Carbon absorption is commonly used as a polishing treatment for removing residual dissolved organic matter from wastewater that has already undergone a biological process (Tchobanoglous and Burton 1991; Pietrelli *et al* 2001). Activated carbon can either be used in crushed granular form known as granular activated carbon (GAC) or in pulverized form known as powdered activated carbon (PAC). Both use the same absorption properties and mechanisms, but rely on considerably different application techniques (Peavy *et a*l 1985). Using the PAC process, the COD removals ranged between 34% and 85% and in column studies, the results were 59%-94% removal (Qasim and Chiang 1994). Carbon absorption is a successful treatment process, but the large quantities needed make this option unfeasible for a main treatment process for leachate. Also this option has been shown to be more effective at removing organics once the leachate is stabilized first by a biological treatment processes (Chian and DeWalle 1977). One advantage of this process is that once the material has been used in it can be reactivated by a thermal process that destroys the organics with only a small loss of total mass (Pietrelli *et al* 2001).

2.6.1.4 Chemical oxidation

Chemical oxidation is used to remove the non-biodegradable organic content (refractory compounds) in wastewater (Pietrelli *et al* 2001). This process involves adding oxidizing agents, such as hydrogen peroxide, permanganate, chlorine compounds to the leachate in order to remove sulphides, sulphite, formaldehyde, cyanide and phenolics (UK DoE 1995). This process converts complex chemical compounds into simple and more easily biodegradable compounds (Ehrig 1987). Sludge will be produced during chemical oxidation unless UV is used as a catalyst (Pietrelli *et al* 2001). This process has been shown to be highly effective but an expensive treatment option (Collivignarelli *et al* 1998).

2.6.2 Physical separations

Common physical treatments for landfill leachate are evaporation, air stripping, and filtration (Qasim and Chiang 1994). The physical processes of flocculation and settling were discussed along with coagulation in the chemical treatment in Section 2.6.1.1. The physical treatment processes are useful in separation methods but must be used in conjunction with other methods. These processes leave the filtered permeate free from the pollutants by concentrating the pollutants rather than treating them. Therefore the problem that remains with this option is how to dispose of the concentrate.

2.6.2.1 Filtration

Filtration processes include separation methods such as reverse osmosis and ultrafiltration. Filtration is the process by which a filtering medium separates out the colloidal particles in the wastewater. Ultrafiltration is a similar process but removes smaller particles. While filtration uses a microstrainers with 15 to 60 μm openings, ultrafiltration uses ones in the 0.0002-10 μm range. Reverse osmosis removes even smaller matter (Qasim and Chiang 1994). This process is best used for leachates with high inorganic loading and low flow rates. It removes dissolved solids, suspended solids, colloidal materials, ammonia and heavy metals. It can also be used to reduce the levels of COD and BOD. This is a separation process that involves forcing the liquid through a semi permeable membrane against natural osmotic pressure. Water passes through while contaminants of high molecular weight cannot (Hammer and Hammer 2000). Reverse osmosis has been found to be the most effective of the physical-chemical processes to remove COD (Qasim and Chiang 1994). Reverse osmosis has also been successful in removing dioxins from leachate from landfills with a high-incinerated ash content (Ushikoshi *et al* 2001). However such filtration processes do not treat any effluents because they only concentrate the contaminants, which still will require treatment or disposal (Peters 1999).

2.6.2.2 Evaporation

The natural process of evaporation can be used to concentrate waste and sludges (Qasim and Chiang 1994). Often this process involves lowering the pH of the leachate in order to convert the volatile ammonia into soluble ammonium salts and then evaporating the liquid. The main problem with this method is the reintroduction of contaminants back into the landfill when the final sludge is disposed (Robinson 1995a).

2.6.2.3 Air stripping

Air stripping of ammonia is one of the most economical means of nitrogen removal (Peavy *et al* 1985) and the most frequently used physical-chemical treatment options for landfill leachate (Robinson 1995a). The main advantage of this process is that it does not require as much area as other processes like biological treatments (Eden 2001). This process is completed in a specifically designed stripping tower where the pH of the leachate is raised to at minimum of 11 to transform the ammonia into its gaseous form (NH_3). Then large amounts of air are forced through the liquid to release the gaseous molecular ammonia into the atmosphere. The main problems with this method are:

- The environmental and social impact of the release of NH_3;
- The disposal of the sludge and effluent from the process; and
- The large amount of power needed to generate the air.

It has been suggested that this would be an appropriate method if there was available waste gas to power the process, such as landfill gas (Eden 2001).

This process has also been successfully used for stripping of methane gas prior to disposal in the sewage system (Robinson 2001). Leachate can contain 2-15 mg/liter CH_4 and concentrations as low as 1.4 mg/liter can cause methane explosions in pipelines transporting the leachate off-site, for example to the sewage treatment works. Methane stripping plants have had a 99% removal efficiency; reducing the methane concentration to 0.095 mg/liter (Robinson 1999).

2.6.3 Biological Treatment

The main goal of biological treatment processes is to reduce the biodegradable organics and nitrogen in order to minimize secondary treatment costs (Pietrelli *et al* 2001). This treatment method utilizes microorganisms to degrade the pollutants in the leachate into less toxic forms through the consumption of organic matter, nitrification and denitrification processes (Qasim and Chiang 1994). Biological treatment of wastewater can occur in anaerobic and aerobic conditions with the biological organisms or biomass that effect the treatment attached to synthetic or natural surfaces (attached growth systems) or with them suspended in the wastewater (non-attached growth systems). These organisms use the biodegradable organics and nutrients as a substrate for survival and growth. In general, biological treatment processes have been used successfully to treat landfill leachate (Qasim and Chiang 1994). These processes use less energy and chemicals in comparison to the other treatment alternatives, but the land requirements are much greater (Qasim and Chiang 1994). Biological processes also have an advantage because they tend to be less expensive than physical or chemical treatments and create only a limited volume of new biomass (Ehrig and Stegmann 1992).

2.6.3.1 Attached Growth -Aerobic Treatment

There are two common examples of attached growth treatments: trickling or percolating filters and rotating biological contactors. The trickling/percolating filter is also called a biological bed since the process involves the biological or chemical oxidation rather than a straining process (Hammer and Hammer 2001). For this treatment the leachate is first aerated by being sprayed into the air and then gravity forces the liquid through a bed of crushed rock or synthetic material that is covered in a bacterial slime. This system has been shown to completely nitrify low-strength leachates with ammonia concentrations ranging from 200-600 mg/liter and COD concentrations ranging from 850 to 1350mg/liter (Knox 1985). One of the main problems with this treatment is that clogging of the system can occur if the leachate has a high organic or inorganic concentration. The high organic load causes clogging through the buildup of bacterial slime and the high inorganic load causes a buildup of inorganic salts. In those cases, pretreatment of the leachate prior to this treatment may be necessary for the system to be successful (Robinson 1995a).

Rotating biological contractors (RBC) consist of rows of circular discs covered in bacterial slime that are attached to a shaft that rotates slowlys allowing a portion of each disk to be exposed to the air while the other is submersed in the leachate (Hammer and Hammer 2000). RBC have been shown to be successful for the treatment of landfill leachates (Knox 1992). The main disadvantage is that it may be impractical to implement an appropriately sized system in order to meet the demands of high and variable ammonia loads (Robinson 1987).

2.6.3.2 Non-Attached Growth Aerobic Treatment

Non-attached growth aerobic treatments are the most widely used and successful process to treat domestic wastewater. Unlike the attached growth systems, these systems rely on the natural flocking action of microbes as substrate. There are three main types of processes: aerated lagoon, sequence batch reactor (SBR) and activated sludge. In all three systems, a large population of appropriate microorganisms (known as activated sludge) is introduced into the wastewater and continually mixed and aerated in order to promote their growth. The distinguishing feature between the processes is that in the aerated lagoon, the excess activated sludge from the process is not recycled as it is in the activated sludge process, while the SBR process is completed in one basin (Qasim and Chiang 1994; Peavy *et al* 1985).

Aerated lagoon treatment systems have been used successfully in treating leachate in the UK since the early 1980s. These systems often achieve an efficiency rate high enough to be able to reliably discharge the effluent directly into very sensitive surface watercourses (Robinson 1999). SBR have been shown to be able to successfully achieve complete biological nitrification of leachates ranging in ammonia concentration of 400 mg/liter (Strachan 1999) to 2000 mg/liter (Robinson *et al* 1995).

2.6.3.3 Anaerobic biological treatment

This process is similar to the one that takes place in the anaerobic degradation phase within a landfill. Like that phase, anaerobic lagoons can be used to reduce high levels of COD and BOD (Ehrig and Stegmann 1992). In these processes microorganisms transform and stabilize complex organics and release carbon dioxide and other organic products. Like aerobic treatment, anaerobic treatment is also divided into suspended and attached growth systems.

The advantage to these methods is they are generally simple and low cost. Another benefit is the energy recovery from methane gas produced by the anaerobic treatment (Water Research Centre 1990). The main disadvantage with the use of this type of wastewater system is that it can be a redundant process once the landfill has reached anaerobic conditions. Also, treating the leachate in an anaerobic environment will not solve the problem of high ammonia concentrations since ammonia cannot be further degraded anaerobically (Ehrig and Stegmann 1992).

2.6.4 Management Options

Depending on the quantity and quality of the leachate, there can be several managerial approaches to treatment that do not rely on additional onsite treatment process.

2.6.4.1 Discharge to the sewer

The co-treatment of leachate and domestic sewage is a common practice worldwide. It is dependant on the pretreatment requirements at the local publically owned treatment works. The discharge of leachate should not cause direct problems with the sewage treatment works as long as the raw leachate comprises less than 0.5% of the total volume (Ahnert and Ehrig 1992). Currently this is the method of disposal used by some of the landfills in South Africa including the Bisasar Road leachate, which has a methane concentration of around 0.4% by volume (Strachan 1999). Another significant concern regarding methane in leachate is if the amount of dissolved methane in leachate reaches concentration of 1.4 mg/liter, it can cause an explosion in pipelines (Robinson 1999; 2001).

2.6.4.2 Recirculation back into the landfill

Leachate released from the landfill can be reintroduced to the landfill thereby using the landfill as an uncontrolled anaerobic treatment processes. By sending the leachate back into the landfill the bacterial processes can continue to reduce the organic component in the leachate. Research has shown that this method can substantially reduce the COD concentration in the leachate (Qasim and Chiang 1994). Other benefits of leachate recirculation are reducing the time for the landfill to reach stabilization and reduction in treatment costs. The main disadvantages are the high capital and maintenance costs (Qasim and Chiang 1994), and that the anaerobic process can do little to remove the high ammonia concentration in leachate (Barber and Maris 1993)

2.6.4.3 Land treatment via spray irrigation

This method uses the natural soil attenuation properties to remove pollutants. Since this method has the potential to cause groundwater and/or surface water pollution, the potential site must be thoroughly examined and local regulations considered (Qasim and Chiang 1994).

2.7 Landfill Management

When waste began to be collected and dumped on land, the first "storage facilities" were non-engineered "dumps". As information grew about the environmental risks and pollution from these sites, ways to separate the waste from the environment were developed. Engineered landfill sites were initiated as a solution, but this however left the waste in an undegraded, stagnate state that would by design remain entombed for as long as the liner and cover materials remained intact. (Robinson 1995; Knox 2000; Stentsøe and Houe 2001).

The potential polluting characteristics of a landfill will remain in the waste until it is leached out, so minimizing leachate formation does not lessen the risk of pollution (Knox 2000). The type of landfill does modify the rate of degradation and leachate formation, and thus the timescale in which the landfill will be a pollution risk. Because the formation and control of leachate is the key issue regarding the polluting potential of a landfill (SA DWAF 1998; Robinson 1998) the types of landfill designs and operations strategies need to be clearly understood in the context of pollution risk and leachate management.

The World Health Organization (WHO) describes four general types of landfills (WHO 1995) shown in Figure 2.7.1.

1. Uncontrolled dump: unengineered and unplanned design resulting in an uncontrolled release of landfill emissions.
2. Total containment: containment with barrier system but without leachate collection or treatment system
3. Containment with leachate control: containment with barrier system and leachate collection and treatment
4. Controlled release: capping layers, no or partial lining, situated on low permeability of natural soils. This is also known as a controlled attenuation landfill

The details of landfill hydrology with respect to leachate are described further in this section having been introduced in the leachate generation section (Section 2.2).

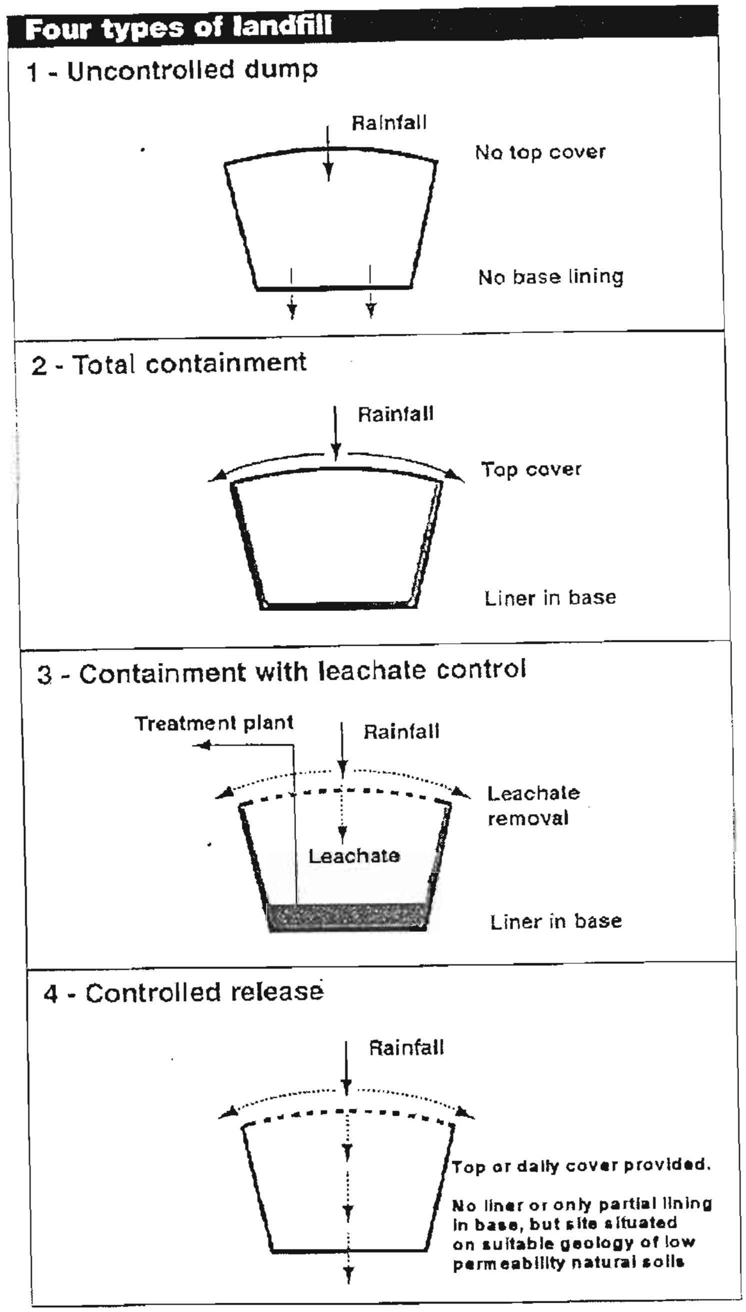

Figure 2.7.1 Illustration of the four general types of landfills (WHO 1995).

2.7.1 Uncontrolled dump

An uncontrolled dump scenario is when no precautions are taken at the site, and the contaminates are unrestrictedly emitted regardless of quantity or quality (Knox 2000). This is often the case when there are no legal waste management requirements. Prior to the concern about the effect of landfills on water quality and the resulting importance placed on landfill locations, many communities dumped their wastes in areas that were already viewed as a waste, such as marshlands, abandoned sand and gravel pits, old strip mines and limestone sinkholes. Unfortunately, having dumps in these areas placed the leachate in direct contact with the ground water, resulting in groundwater pollution (Hammer and Hammer 2001).

In order for this method to not result in pollution, it requires that the surrounding environment be able to purify the leachate as it migrates through the soil (Ball and Blight 1988). The rate at which the pollution from these activities reaches the groundwater is affected both physically by the porosity of the soil and hydraulically by the rate of water movement (Hammer and Hammer 2001). This natural attenuation process has been used as the primary leachate management mechanism in South Africa as well as in other countries (UK DoE 1987 and Robinson 1987; Strachan 1999). Attenuation is not considered an acceptable practice for waste management, but it may be considered an appropriate final stage once the waste is determined to have little pollution potential.

Even landfills that were not designed to contain leachate, may still produce a significant amount of leachate that will need to be subsequently managed. This is the case for several unlined landfills in South Africa. Drainage systems, treatment and/or disposal of leachate have had to be retrofitted to many sites (Strachan 1999). An example would be the Bisasar Road landfill used for this study.

2.7.2 Encapsulation

The goal of encapsulating a landfill is to prevent any infiltration of water into the landfill or percolation of water out of the landfill. This keeps the field capacity of the waste unsaturated and therefore no leachate is produced. Design and construction of such a landfill requires the use of liners, collection systems, and peripheral cut-off drains to be put in place. The lining used is typically a multi-lining system, using both natural mineral and clay layers with synthetic flexible membrane liners (geomembrane liners) (Qasim and Chiang 1994). While the barrier systems put in place are never completely effective, landfills have also been designed and sited in response to the demand of keeping the leachate separate from the environment (Conziani and Cossu 1989; Qasim and Chiang 1994). During operation areas in a landfill must be covered with a compacted soil material to prevent water entry, migration of odors and wind-scatter (Strachan 1999; Qasim and Chiang 1994). It is a more sophisticated and expensive method especially in comparison to the attenuation approach (Robinson *et al* 1992; Qasim and Chiang 1994).

The problem with these designs is the uncertainty in assigning a timeframe for how long the liner/drainage system should be required, and just as significant are the uncertainties in regards to the realistic lifetime of those systems (Stentsøe and Houe 2001). There have been published cases of landfill failure (e.g. Koerner and Soong 2000), liner failure (Peggs 1994 and draining system clogging (Rollin *et al* 1991; Farquhar 1989; Bruen *et al* 1993; Boswell and Bell 2000). This provides evidence that there are legitimate short- and long-term possibilities for pollution. Since this scheme is completely dependent on the containment system put in place, once that system fails, the leachate can begin to flow uncontrolled into the surrounding environment (Rohrs *et al* 2001).

2.7.3 Containment and collection

A containment and collection landfill is the most commonly occurring landfill design and operation practice (Knox 2000). A landfill designed for containment and collection of leachate will require similar barrier systems as an encapsulated landfill, but it is necessary to allow for, and in some cases encourage, leachate production. The leachate will then be collected and not allowed to be in contact with the surrounding environment. So, instead of uncontrolled releases to groundwater, it could then either be treated onsite or sent offsite for treatment.

Like encapsulated landfills, containment landfills also have the problem of the failure of the containment system and clogging of the drainage collection system. Obviously since these systems have been covered by tons of landfilled waste, it is nearly impossible to restore the integrity of either system once waste has been placed in the landfill (Strachan 1999).

2.7.4 Controlled attenuation

The difference between a controlled attenuation landifll and the previously mentioned uncontrolled attenuation landfill, is the degree of planning and engineering regarding the siting, design and operation of the landfill. The attenuation approach for managing landfill is achieved by controlling the quantity and/or quality of the leachate generated, and then allowing it to flow unrestricted into the ground and groundwater (Knox 2000). It also requires that the leachate be acceptable to the environment and/or that the natural permeable liners and layers be able to attenuate the pollutants before reaching the water supply.

This has been found to be an unreliable practice since leachate attenuation is extremely difficult to predict or quantify, especially when the volumes exceed predictions (Ball 2002). Other uncertainties with this landfill design include the long-term fate and impact of the pollutants in the leachate to the receiving environment (Heyer *et al* 1998), and the time-scale that the leachate will continue to be a pollutant (Christensen *et al* 1998). This may be a viable treatment option after the initial active treatment is completed. Stentsøe and Houe (2001) refer to this as the passive phase in the life of a landfill. Once the waste reaches a certain state, it can be left unattended. The leachate will then be released in an unrestricted manner such that it does not create an unacceptable impact to the environment.

2.7.5 Flushing bioreactor

While the predominant strategy used to manage landfills is the containment and collection approach, there are still decisions that must be made in regards to the operations of the site. This containment approach does take into consideration that leachate is produced, but the waste are still entombed and kept separate from the environment. These landfills are engineered to protect the environment by limiting the contact of water with the waste, but this causes the waste to degrade slowly in comparison to waste that is allowed to be flushed with water (Röhrs *et al* 2001). Liners and covers that keep the waste encapsulated will not last indefinitely, and eventually moisture will enter the fill and the highly polluting leachate will be allowed to enter the environment (Röhrs *et al* 2001).

In order to solve this problem it has been suggested that instead of considering the landfill as a storage compartment, it should be managed like an anaerobic reactor. This flushing-bioreactor approach attempts to accelerate the degradation of wastes and the release of leachate in order to more rapidly induce stable landfill conditions. It does this by focusing on enhancing the biochemical processes in the landfill so that the waste becomes inert and the leachate emissions become compatible with the surrounding environment (Burton and Watson-Craik 1998; Robinson 1995b, 1999). Once waste decomposition is accelerated, the end products can then be released back into the environment in an acceptable manner and at an acceptable rate (Robinson 1995b).

In order for this to be achievable, the wastes must reach a stable decomposed state and contaminants flushed from the waste body. Accelerating biological decomposition is a feasible option, but increasing the rate in which the landfill is flushed is not. Flushing the contaminants initially seems to be a matter of allowing water to pass through the site, but the mean hydraulic retention time may be as long as a century (Robinson 1995b)

The flushing-bioreactor approach is theoretically feasible, but there are several hindering problems with this process:

- The actual timescales needed for the landfill to become inert are unknown, but through modeling it has been projected to take 300 years under optimum conditions (Robinson

1995). According to another study (Röhrs *et al* 2001) it would take between 320 to 1090 years for certain leachate concentrations to reach an acceptable limit for the landfill to no longer need aftercare monitoring and treatment.

- There have been notable difficulties with the flushing process and leachate circulation (Robinson 1997).
- There will be preferential flow paths within the waste leaving some areas unaffected by the flushing process (Robinson 1995).
- Due to field capacity of the waste, some leachate will remain within the landfill (Knox 2000).

2.7.6 Landfill management and leachate treatment

Regardless of design, management, or operation methods chosen, leachate will be produced if the water balance of the landfill is positive; so as it builds-up on the base of the site, extraction and treatment systems will be required (Robinson *et al* 1992). The two basic leachate management philosophies have been the "dilute and disperse" approach and the "contain and concentrate" idea. The "dilute and disperse" is the method used in both controlled and uncontrolled attenuation landfills since it relies on the attenuation capabilities of the surrounding environment. As mentioned previously, pollution results when the capacity of the environment is exceeded (Ball 2002). "Contain and concentrate" is the idea used in encapsulated and contained landfills. It is the most common method used, but it is expensive and the life-span of the barrier systems is unknown (Knox 2000).

There does not seem to be consensus about the best type of landfill or which would be the most sustainable/environmental-friendly. In South Africa, attenuation landfills may prove to be the most appropriate and sustainable option (Strachan 1999). If the goal is to attempt to stabilize the waste as rapidly as possible, uncompacted, open dumps may be the best at achieving this bioreactor concept (Cossu 1997; Robinson 1995). It has been determined that neither approach would wholly satisfy the needs or requirements of leachate management in the South African context. While the more sophisticated "containment" method would prevent any leachate from polluting the surrounding environment, its expense may not always be appropriate, considering that many of the landfills are in arid regions that would not produce significant volumes of leachate. Also, even on sites that do produce significant volumes, the types of leachate (and resulting potential pollution) and the cost of containment and treatment must be balanced. One of the suggestions is to use a gradation of leachate management practices from using attenuation designs for low-risk leachates to the more sophisticated containment designs to manage high-risk leachates (Ball 2002).

Chapter Three

Legislative Framework

3.1 Introduction

Environmental legislation covering landfills and landfill emissions has been the result of increased knowledge about the health and environmental risks generated from this waste management practice. In most countries, each aspect of waste disposal has legislative guidelines from the siting of a new landfill through to closure (Crawford and Smith 1985). Landfills that generate leachate must also abide by the water quality legislation that governs its disposal. Since leachate is a result of waste decomposition and moisture entering the waste body, legislation governing engineering designs and management practices directly relates to the quantity and quality of leachate produced.

As people have become more environmentally aware, the role of environmental policy has been adjusted to reflect those changes. The most recently accepted concept has been sustainable development and sustainability; this idea is being focused on worldwide in a multitude of ways due to the myriad of definitions and philosophies by which it is described. In the context of the waste management industries, one concept being promoted is that of sustainable landfilling, which is also marred with a plethora of definitions and directives. Unfortunately in addition to the problem of attempting to encourage practices based on an indefinable concept, there is the conflict between legislation requirements and policy goals. Current legislation requires short-term environmental protection while government policies promote long-term (i.e. sustainable) environmental protection. As described in Section 3.6.1, sustainable landfilling can be approached in a number of ways depending on the perspective of the parties involved. One scheme to promote sustainable landfills has been the desire to use sustainable leachate treatment methods.

An objective in this research was to examine the use of constructed wetlands (CW) as a sustainable treatment method, but sustainability has little to do with the leachate treatment option chosen, as will be discussed further in this chapter and Chapter Four. All treatment options rely on the rate of degradation of wastes and the sustainability of the leachate barrier and collection systems. Instead of sustainability, what is important when discussing leachate treatment systems is their appropriateness in the given context. Section 4.6 describes the reasons and circumstances required for CW to be considered an appropriate technology for treating landfill leachate.

3.2 Connection of Regulations to Leachate Treatment

The focus of this dissertation is on the use of constructed wetlands as an appropriate treatment for landfill leachate. As part of that focus the generation of leachate must be understood. The climatic, biochemical, and management factors that affect leachate production were described in the previous chapter (Chapter 2). As shown in Figure 3.2.1, the production of leachate and the standards that apply to it are partly the result of national environmental policies, landfill legislation and water quality standards. They each have an influence in determining the quality and quantity of leachate that is produced and how it must be treated. These legal directives and requirements should support each other, but as will be discussed in Section 3.7, they often have conflicting aims.

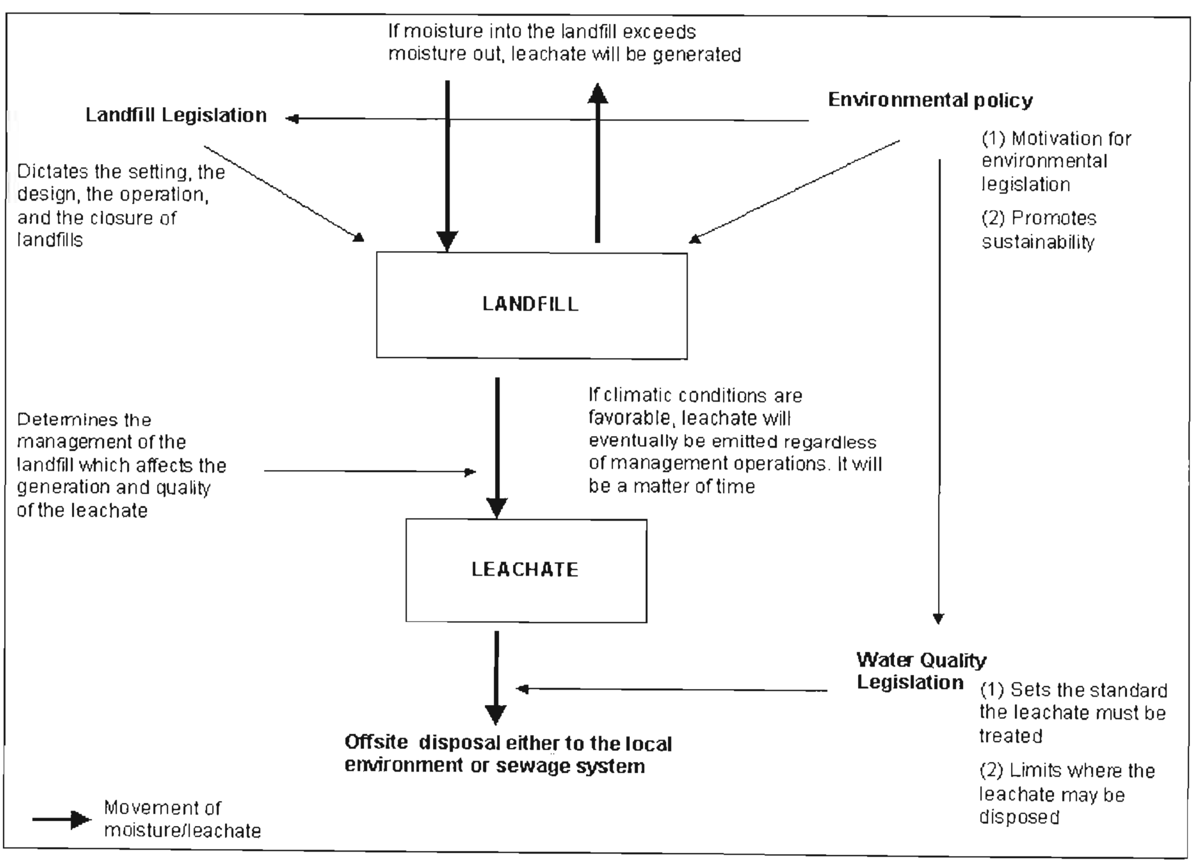

Figure 3.2.1: The connection of governmental legislation and policy to landfills, leachate and treatment

3.3 Reason for legislation

Legislation governing the disposal of waste was introduced, as early as 1875 in the United Kingdom (UK), to prevent the human health problems that had resulted from unmanaged waste. Since then most countries have implemented legislative requirements to protect the health of people and the environment (Williams 1998). This type of legislation defines limits and restrictions on industries in order to limit environmental pollution (Canziani and Cossu 1989). One of the reasons landfill legislation exists is to prevent pollution from contaminating the local aquatic systems, which are often used as a water supply, food resource, recreation and ecosystem habitat (Peavy *et al* 1985; Hammer and Hammer 2001). Along with waste management legislation, standards governing the disposal of wastewater effluents were also developed to protect this resource from indiscriminate use. To accomplish this, they stipulate the limitations on the quality and quantity of wastewater effluent that may be sent into the local water body or into the local sewerage system. These regulations vary between countries, but in general they all have the overall goal of protecting the aquatic environment and human health (Williams 1998; Hammer and Hammer 2001).

In regard to landfills, both the landfill operations and the by-products are regulated. Legislation regarding landfill operations covers issues such as site selection, design, construction, operation, and aftercare requirements. The overall goal is to prevent pollution of local ground and surface water. Preventing groundwater pollution is critical because after contamination, remedial actions are generally not technically or economically feasible and natural purification may or may not occur regardless of timescale (Hammer and Hammer 2001). The majority of landfills are designed to contain the leachate that is produced. So unlike the leachate that is diffused as a result of attenuation landfills (as described in Section 2.7), this concentrated leachate is a point source pollutant and considered an industrial effluent in most countries. It is regulated according to where it is released: it must meet wastewater effluent standards if released into a watercourse or local pretreatment standards if disposed into a sewage system (Hammer and Hammer 2001).

3.4 Landfill and Leachate Legislation in South Africa

3.4.1 Landfill Legislation

In South Africa, the Department of Water Affairs and Forestry (DWAF) has developed a series of regulatory requirements for the selection, investigation, design, permitting, preparation, operation, closures and monitoring of landfills. This series is known as the

Minimum Requirements for Waste Disposal by Landfill (SA DWAF 1998). The overall aim of the Minimum Requirements is: to ensure practical and affordable environmental protection. To accomplish this the document has several main objectives:

- To take pro-active steps to prevent the degradation of the environment;
- To improve the standards of waste disposal in South Africa;
- To provide guidelines for environmentally acceptable waste disposal by landfills of different types and sizes; and
- To provide a framework of minimum waste disposal standards for the general development of landfills.

The SA landfill legislation is similar to standards enforced in North America and Europe. For example in the United States (US) waste is regulated through the Resource Conservation and Recovery Act (RCRA). The US Environmental Protection Agency (EPA) is responsible for the programs and legislation under RCRA. RCRA's goals are to: protect the environment from the potential hazards of waste disposal, conserve energy and natural resources by recycling and recovery, reduce or eliminate waste, and clean up waste which may have spilled, leaked, or been improperly disposed of (US EPA 1989). The overall legislation gives similar criteria for municipal solid waste landfills in the US as in SA. Both have standards regarding:

- Location restrictions - ensuring that landfills are built in suitable geological areas
- Operating practices - such as compacting and covering waste frequently to reduce odor; control litter, insects, and rodents; and protect public health.
- Groundwater monitoring - requiring testing groundwater to determine whether waste materials have escaped from the landfill and then implementing corrective action controls
- Closure and post-closure care - including covering landfills and providing long-term care of closed landfills.

One of the main differences in legislation between South Africa and the United States is in regard to the containment requirements. In the US all landfills are required to have liners that are geomembrane or plastic sheets reinforced with two feet of clay on the bottom and sides of landfills (US EPA 1984). In South Africa landfills are classified by their waste type, size of operation and potential for significant leachate generation. Only those storing hazardous waste or have a potential significant leachate generation require liners and leachate management (SA DWAF 1998). As described in Chapter Two, the potential quantity of leachate generated is determined by landfill hydrology and management. Landfills that accept general waste (such as Bisasar Road landfill - the study site selected for this project) will require liners and leachate management depending on the landfill's water balance. If rainfall exceeds evaporation, it is considered a water surplus area, and as such significant leachate is expected to be produced. A liner is then required with a maximum outflow of 300 mm/year. This classification (known as a B+ landfill) dictates the permit regulations and operational strategies of an existing landfill and the liner/barrier system requirements for the development of a new landfill (Ball 2002).

There are concerns about the appropriateness of this current legislative requirement. The definition used in South Africa for significant leachate generation is only based on the amount of precipitation exceeding evaporation. The issue is that this does not take into account local geohydrological conditions or the quality of the leachate in determining what constitutes leachate that could impact adversely on the environment (Blight and Novella 2000). Instead of this arbitrary justification of significant leachate, Blight and Novella (2000) suggest that it would be more rational to determine the quality and quantity of leachate that can be absorbed by the surrounding environment without it being adversely impacted. This would save unnecessary costs involved with complying with unnecessarily stringent requirements.

3.4.2 Water Quality and Effluent Legislation

If an industry wants to dispose of its effluent into a local water body, that effluent must meet certain legislated standards. The original standards covering effluent in SA were set in the General and Special Effluent Standards (SA Government Gazette 1984). They have since been superseded by the National Water Act (SA Act No. 36 1998). Under the National Water Act, water standards were developed based on the receiving water qualities. These standards, described in Government Gazette No.20526 (SA DWAF 1999), allow for a set quantity and quality of effluent to be released depending on the status given to the receiving aquatic system. Table 3.3.1 shows the South African discharge limit values applicable to discharge of wastewater into a water resource (SA Government Gazette 1999).

The SA system was based on the determination of requirements in the United States (Hammer and Hammer 2001). In the US all industrial, municipal, and other facilities that discharge their effluents to surface waters are regulated according to the National Pollutant Discharge Elimination System (NPDES) permit program. NPDES is based on individual discharges and takes into consideration the stream or water body where the release is occurring. All surface waters are classified by their most beneficial use and that determines what the most appropriate physical, chemical and biological water quality standards should be for that type of aquatic system. The overall goal is not to further pollute the aquatic system and to improve the water quality (Nathanson 2000).

Table 3.4.1: Discharge limit values applicable to discharge of wastewater into a water resource (SA Government Gazette 1999). The listing of which water resource are governed by the special limit are found in Appendix A.

Parameter	General limit	Special limit
Fecal Coliforms (per 100 ml)	1000	0
COD (mg/l)	75*	30*
pH	5.5 to 9.5	5.5 to 7.5
Ammonia (mg/l)	3	2
Nitrate/Nitrite (mg/l)	15	1.5
Free Chlorine (mg/l)	0.25	0
Suspended Solids (mg/l)	25	10
Electrical Conductivity (mS/m)	70 mS/m above intake to a max. of 150 mS/m	50 mS/m above background receiving water to a max. of 100 mS/m
Ortho-Phosphate (mg/l)	10	1 (median) and 2.5 (max.)
Fluoride (mg/l)	1	1
Soap, oil or grease (mg/l)	2.5	0
Dissolved Arsenic (mg/l)	0.02	0.01
Dissolved Cadmium (mg/l)	0.005	0.001
Dissolved Chromium (mg/l)	0.05	0.02
Dissolved Copper (mg/l)	0.01	0.002
Dissolved Cyanide (mg/l)	0.02	0.01
Dissolved Iron (mg/l)	0.3	0.3
Dissolved Lead (mg/l)	0.01	0.006
Dissolved Manganese (mg/l)	0.1	0.1
Dissolved Selenium (mg/l)	0.02	0.02
Dissolved Zinc (mg/l)	0.1	0.04
Mercury and its compounds (mg/l)	0.005	0.001
Boron (mg/l)	1	0.5

Note: *After the removal of algae

In South Africa leachate treatment is only required when the leachate needs to meet required effluent standards (Strachan 1999: SA DWAF 1998). Since the leachate from Bisasar Road is not released into the environment but transported to the local sewage system, the leachate must comply with the conditions listed in the Durban Metro Sewage

Bylaws (DMA 2002). The charges and tariffs for leachate disposal are based on total volume and amounts (mg/liter) of COD and settleable solids (DMA 2002).

3.5 Environmental Policy

The South African environmental policy requires that there be a national effort to promote achieving sustainable development (SA DEAT 1996). In the United States, the national charter for the protection of the environment is the National Environmental Policy Act of 1969 (NEPA) (42 U.S.C. 4321-4347). It establishes policy, sets goals, and provides means for carrying out the policy to protect the environment in a sustainable way. Since the current guiding directives and principles used in waste management policy and water resource protection derive from both legislative requirements and the idea of sustainability as promoted by the national government's policies (Williams 1998), the concept of sustainability and sustainable development should be reviewed.

3.6 Sustainable Development and Sustainability

Since the publication of the World Commission on Environment and Development's (the Bruntland Commission's) report *Our Common Future* (WCED 1987), sustainable development has become the guiding principle at all levels of policy making from the international to the local (Gibbs *et al* 1998). In fact it is a political virtue that ranks as high as democracy, justice and liberty (O'Riordan 1993). The most publicized definition of sustainable development is the one stated in The Bruntland Commission's report (WCED 1987): "development that meets the needs of the present generation without compromising the ability of future generations to meet their own needs". What sustainability and sustainable development actually implies and requires has been the subject of an on-going debate since the concept first became a worldwide focus at the 1992 United Nation's Earth Summit in Rio de Janeiro.

There are a plethora of interpretations of sustainability depending highly on the political and economic perspective of the individual and on the overall goals of the industry promoting it. Critics contend that this concept is vague and inherently self-contradictory (O'Riordan 1993) and it is designed to be universal yet applicable to local conditions (O'Riordan *et al* 2000). It applies most readily and obviously to the idea of sustainable utilization of resources: such that the rate of use is equal to the rate of renewal (O'Riordan 1993). That early definition has expanded, and now it seems every organization has as its driving goal to be sustainable, but rarely if ever is that defined. Typically it refers to an organization striving to create less of an environmental impact, but that does not necessarily equate with sustainability. Hawken

(1993) describes sustainability as processes that imitate the cyclical processes in nature. In nature, all by-products are not considered "waste" but a needed input into another system; it is this assimilative capability that has kept the environment operative for millions of years (Peavy *et al* 1985). Sustainable practices should all follow that example and be cyclical, otherwise they are linear systems, which by function and definition, are limited and short-lived (Hawken 1993).

3.6.1 Sustainable Landfilling?

The vagueness surrounding the definition of sustainable development and sustainability, when applied from policy through to organizational practices, is both its strength and its weakness; for it can be embraced by industry, politicians and environmentalists. It is because of and despite of these criticisms, that sustainable development remains the approach most often used when discussing environmental issues or development goals. It is not surprising then that the waste management industries have also begun promoting "sustainable" practices. Like other practices that tout sustainability, sustainable landfilling is a concept that is often used when describing modern landfilling goals, but it is rarely defined (Robinson 1998; Rohrs and Fourie 2002). Most often researchers (such as Röhrs and Fourie 2002) use the term by equating it with the option that creates a limited environmental impact regardless of whether it is truly sustainable or not. While waste management via landfilling may be the Best Practical Environmental Option (BPEO) (Robinson 1995), that does not also mean that it is a sustainable practice. This is due to its very nature of being at the end of a linear system in which the end products of resources and energy inputs are neither cycled nor returned (Hawken 1993).

Since disposal of non-usable waste in landfills does not fit the general definition a sustainable process, then what is usually meant by a "sustainable landfill"? From literature describing "sustainable landfills", it seems that "sustainable" is used instead of the more fitting but no less vague adjectives of "stabilized" or "less polluting". For example Robinson (1995) suggests that the goal of sustainable landfilling should then be to strive to minimize potential risks as soon as possible. By aiming to reduce pollution potential, this should equate to a decrease long-term costs and environmental risks (Mathlener 2001). The most commonly used definition is that the polluting life of a landfill should be one generation (30-50 years), so that each generation shall dispose of their own waste without leaving problems for future generations (Stentsøe and Houe 2001). By the end of that time the waste should be completely degraded and therefore stabilized. Ideally all that remains should be an inert residue with no pollution potential (Röhrs and Fourie 2002). All leachate leaving the site should then resemble the groundwater flow similar to that of the adjacent soil matrix.

To reach the goal of sustainable landfilling, the siting, construction, management, and type of waste must also be focused on the goal of minimizing long-term pollution potential. Long-term pollution potential is based on the rate at which waste degrades and then leaves as leachate. This can be accelerated by modification in landfill operations and pretreatment of the waste. Cossu *et al* (2001) list the main options proposed for the reduction of landfill emissions: mechanical-biological processing or thermal pre-treatment of waste; *in situ* aeration of the waste mass by means of natural air inflow or by forced aeration (aerobic landfill); flushing of the waste mass *in situ* (flushing bioreactor (Robinson 1995)). While these have been shown to be relatively successful in reducing emissions, there are still problems with each option: pre-treatment often leaves high residual emission, there is drainage clogging in aerobic landfills; and flushing bioreactors have been shown to have difficulties with hydraulic circulation and other operational problems (Cossu *et al* 2001).

Consideration of the type of waste being landfilled is also critical so acceptable attenuation landfill can be created. In these situations there should be limits on the type of waste accepted or pre-treating the waste. The type of waste included, such as ash, can also encourage the onset of methanogenesis (Shamrock 1998) and improve water retention, which also aids in accelerating decay (Röhrs *et al* 2001). The other factors that effect leachate generation are described in Section 2.2. The need for artificial liners can be avoided if municipal waste is separated from hazardous waste, and the landfill is appropriately sited in an area with geological and hydrogeological conditions with acceptable retention and attenuation characteristics (UK DoE 1978; Stentsøe and Houe 2001).

The design of the landfill can assist in creating a stable equilibrium condition at the landfill site by controlling the leaching of contaminants. This may also be accomplished by modifying the natural clay liners to enhance the attenuating properties and/or designing the liners to deal with much higher flows (Robinson 1999). The goal should be to reintroduce waste products into the environment in an acceptable manner and acceptable rate (Robinson 1995). This may be accomplished by requiring that the waste be treated or managed in such a way that the leachate is acceptable from the first day it is released (Stentsøe and Houe 2001) and/or through the treatment and management of the emissions from the landfill. While the atmospheric emissions are outside the scope of this research, there are opportunities to use the landfill gas as an energy source. As described in Section 2.6, leachate can be treated in a variety of ways depending on restrictions and requirements. In general the overall goal of sustainable landfilling in regards to emissions is to create a landfill that does not pollute. While this has yet to be accomplished, the more practical option is to treat of leachate at the source by using an appropriate technological option.

3.6.2 Concerns with the Technical Definition of Sustainable Landfilling

The concerns with the parameters used to describe these environmentally sound landfills are due to the uncertainty and to a general lack of knowledge involved with landfill design, management and aftercare. This uncertainty ranges from the hydraulic properties of the waste to the life expectancy of the barrier systems used. All these schemes that use engineered containment and collection systems rely on the integrity of those systems. Therefore the environment can only be protected during the expected lifetime of those systems. Even the flushing-bioreactor approach to sustainable landfilling has not been shown to be able to stabilize waste within a generation, and through modeling this process it is predicted it will take hundreds of years to stabilize (Robinson 1995; Röhrs *et al* 2001). Even under the most desirable of conditions, the less soluble constituents, such as heavy metals may take hundreds of years to be leached. Futhermore there are preferential flow paths in the waste, which could leave areas undegraded (Robinson 1995). Regardless of these knowledge limitations, landfill will always be needed (Röhrs *et al* 2001). Perhaps in order for the waste to be non-polluting, it must be treated or managed in such a way that the leachate from its initial placement does not pose a pollution risk to the surrounding environment (Robinson 1995; Stentsøe and Houe 2001). How this will be accomplished in a set timeframe has yet to be determined.

If environmental policy and the waste management industry as a whole desire a more sustainable system, then the concept of sustainable landfilling should be broadened to include non-technical issues as well. As the current technical definition stands a sustainable landfill is one that should reach final storage quality within 30 years after closing. This concept should include more than just engineered changes to have a sustainable landfill. Real sustainability should involve social, environmental and economic elements (Turner 1993; Hajer 1995; O'Riordan *et al* 2000). One method of achieving this to develop a set of guidelines to address this need for integrated sustainable landfilling, such as the one the Global Reporting Initiative has completed as a way for external sustainability reporting (Mathlener 2001). While the concept of "environmentally sound" landfills is a worthy aim, it avoids the fundamental questions regarding the creation of waste (Hawken 1993). Stabilizing the waste and preventing pollution from the landfill over the short- and long-term should continue to be a focus of the waste management industry because for now there waste is being generated and must be disposed of (Röhrs *et al* 2001). If the true aspiration is sustainability, then landfills must be part of a holistic waste minimization focus that should then be part of a greater goal of a sustainable society.

3.7 Policy versus Legislation

Another issue surrounding the goal for environmentally sound landfills is the conflict between legislation regulations, which promote short-term environmental protection and policy directives, which have a long-term perspective on pollution. Policies should guide legislation, but in the case of landfills, governments aim for sustainability but then legislate unsustainable regulations. Policy wants to promote the most sustainable and environmentally-friendly landfill, but there is indecision regarding how current landfill practices should be modified. Currently landfills are designed to keep the leachate separate from the environment by limiting the amount of moisture entering the site through capping and using liners and collection systems (Robinson 1995; UK DoE 1996; Knox 2000). For example in South Africa (SA DWAF 1998) landfills are required to be "dry-entombed" through the use of liners and capping layers. The problem is that keeping moisture out of the site retards the degradation of the waste (Lee and Jones-Lee 1993) for as long as the containment holds. While this might be the best environmental option for the short-term, over the long-term the containment will not last and leachate will enter the environment as soon as moisture enters the waste (Stentsøe and Houe 2001; Röhrs and Fourie 2002).

3.7.1 Pollution over the Short or Long Term

This conflict between the difference in the short-term and long-term environmental focus is a worldwide one. For example all members of the EU must develop plans on how their existing landfills will be modified to comply with the new common minimum requirements set by the EU directives. This is a challenge because it is focused on further encapsulating the waste rather than supporting any sustainable landfill concepts, and it does not take into account any site or waste specific conditions (Robinson 1998; Stentsøe and Houe 2001). The local conditions should be taken into consideration as a requirement if an appropriate solution to landfilling is desirable (Robinson 1998). Also considering that the barrier systems have to be assured infinitely there is concern that these measurements will not be effective or efficient in the long-term (Mathlener 2001). This conflict between legislative requirements and policy directives can be seen in Table 3.7.1 and in the following description of the requirements for closure and aftercare legislation.

Table 3.7.1: A comparison between the focus of short-term and long-term environmental concerns.

Short-term environmental concern	**Long-term environmental concern**
Landfill Legislation	**Technical Sustainability**
Legislation demands that leachate be separated from the environment	Sustainability demands that the landfill does not cause pollution over the long term (30-50 years)
Promotes a containment landfill with 'dry-entombed' waste	Promotes a attenuation landfill that allows contact with the environment in a controlled and managed way
Ideally a permanent storage facility with unchanging waste	Ideally a 'flushing bioreactor' that degrades all waste leaving them in a stable, inert state.
Uncertainty regarding the lifespan of the barrier system	Uncertainty regarding the amount of moisture needed, the time scale required and preferential flows patterns.
Environment will still be impacted by the leachate once the barrier system fails	Environment will continue to be impacted for as long as the waste continue to degrade and the leachate from those waste can only be treated for as long as the liner and drainage system remains intact.
Alternatives: only allow this method to be used when the waste generated will cause harm regardless of dilution or attenuation properties (toxic waste). Focus on not creating this type of waste.	Alternatives: treating/managing the waste so from day one the leachate can be released into the environment. Designing a landfill with attenuation capabilities.

3.7.2 Current requirements and Their Effect on Closure/ Aftercare

Once the capacity of a landfill has been reached, the operator must follow the regulated guidelines for closure. In South Africa prior to closure the site must be deemed environmentally acceptable and suitable for its proposed end-use. The current legislative requirements governing landfill operations are based on the scheme to keep the waste dry and separate from the environment (a 'dry-entombment' landfill). It follows then that the closure requirements will strive towards the same. In the United States, the EPA requires that owners or operators of all municipal landfills must install a final cover system that is designed to minimize infiltration and erosion (USA CFR 2003). Table 12 in SA Section 12 of the Minimum Requirements (SA DWAF 1998) lists the Minimum Requirements for Rehabilitation, Closure and End-use. As in the initial design and operation requirements, the closure requirements are based on the classification of the landfill.

After the closure regulatory process is complete, most countries have required aftercare periods for landfills; such that after the fill is closed it must be monitored (SA DWAF 1998). It remains the responsibility of the owner/operator until the site reaches a stage where it no longer is polluting or has the potential to cause any pollution (Robinson 1995b). For some countries (e.g. Switzerland and Germany) this aftercare period is based on the time it takes for the leachate to reach a compliance limit for certain chemical parameters. Other countries (e.g. South Africa) only have a 30-year post-closure care period regardless of whether the waste has reached stabilization and therefore poses no further environmental risk (Röhrs *et al* 2001). This is the same in the US although the length of the post-closure care period may be increased or decreased depending on the potential of the landfill to continue polluting (USA CFR 2003)

3.7.3 Concerns with the Determination for Aftercare

There are two issues with depending on the requirements for the after-care period. If it is set by a definite time period, the barrier system will fail after the landfill has already been closed. This will allow leachate to be released to the environment with no drainage or treatment facilities in place and leave society to be responsible for the pollution. If the closure is based on a set limit for leachate quality to determine the duration of the aftercare period, it may require an expensive and extended active management period (Röhrs and Fourie 2002). This would create a prohibitory expenditure for the landfill operators (Robinson 1995b). Röhrs and Fourie (2002) suggest that neither of these options would be desirable and the post-closure maintenance period should be based on when the polluting potential of the landfill is low, rather than on a preset time limit. The polluting life of the landfill then would be the time required in order for a landfill to have undergone full biological decomposition and therefore reached a stable state. Full biodecomposition will require flushing the contaminants through the site (Robinson 1995b). The pore volumes of leachate needed to flush the waste gives a more accurate estimate for required aftercare periods instead of a predetermined universally legislated number (Röhrs *et al* 2001). This process has been estimated to take anywhere from several hundred to several thousand years to reduce the contaminant concentration to an acceptable or regulatory approved level (Robinson 1995b; Röhrs *et al* 2001). Under current legislative conditions, it will take even longer since current landfill legislation strives to reduce the amount of moisture entering the site (Robinson 1995b; Röhrs *et al* 2000).

Even if legislation was altered and flushing encouraged, ideal stabilization will never fully occur because there will preferred flow channels in the waste stream and some contaminants will remain for an indefinite timeframe (Robinson 1995b). Considering the large cost in attempting to contain the waste and the leachate over the short-term it would seem more cost-efficient to seek long-term solutions that do not rely on unsustainable barrier systems (Mathlener 2001). Instead of waiting for the containment to be disrupted or for the waste to slowly degrade, operators should take advantage of having the liner and collection system in place and use them and the resources at the site to treat the leachate. It has been proposed that the degradation process be not only encouraged but also accelerated (Robinson 1995b; 1996; 1998). While there are problems with this as well, especially in regard to timelines and the preferential flow of leachate in the waste, it is still a more appropriate option for it will not leave the next generation with the burden of solving the current generation's leachate and waste problem.

Chapter Four

Constructed Wetlands

4.1 Introduction to Constructed Wetlands

Wetlands are often described as "the kidneys of the landscape" because of the role they play as downstream receivers and transformers of chemicals (both anthropogenic and non-anthropogenic) in the environment. There is not an exact definition of a wetland because of the wide variety of hydrologic and geographic conditions in which they are found. The universal feature of wetlands is that they must have water present at the surface or root zone long enough to develop unique soil conditions (hydric soils) that can support vegetation adapted to saturated soil conditions (hydrophytes) (Rogers *et al* 1985; Mitsch and Gosselink 1993). The major factor that distinguishes wetlands from other ecosystems is that due to the waterlogged soils, the oxygen supply is limited thus creating anaerobic conditions (Rogers *et al* 1985). There is a dynamic biogeochemical relationship in wetlands that causes these ecosystems to be efficient sources, transforms and sinks for a variety of chemical constituents (Mitsch and Gosselink 1993). This characteristic, along with their natural ability to withstand fluctuations in hydraulic loadings, has allowed them to be used for wastewater treatment (Lehman and Rodgers 2000).

Historically wetlands have been used indiscriminately as dumpsites for both waste and wastewater (Mitsch and Gosselink 1993). This occurred not because of their attenuating properties but because they were not valued by society. As scientists began to uncover the processes and functions of wetlands, they were being purposefully used as a treatment option. Natural wetlands have been used to regulate sediment, nutrient or pollutant loadings with varying success (Brix 1993; Wetzel 1993). Their use may still occur, but in most countries (e.g. the United States of America and South Africa) the rise in conservation of these systems has lead to legal protection against pollution. Constructed wetlands (CW) engineered for the purpose of treating effluent bypasses this legal problem and ensures a much more reliable control and therefore higher treatment efficiencies than natural wetlands (Wetzel 1993; Reed *et* *al* 1995). CW have been designed to simulate, and even enhance, the wetland natural attenuating capabilities (Wetzel 1993).

Constructed wetlands (CW) have been used for over 50 years for wastewater treatment and there are now several thousand in operation throughout the world (US US EPA 2000). Prior to their use as an option for leachate treatment, they have been used for the treatment of sewage in both Europe and North America (Robinson *et al* 1993). Since then studies have generally shown that they are able to receive and treat any contaminated waste that can be treated by biological and physical/chemical means (Wood 1999). CW have been used as a secondary or tertiary treatment for a variety of municipal, commercial and industrial wastewater effluents. Focus has been given to this type of passive treatment over other options due to the high costs of advanced treatment systems (Rogers *et al* 1985; Surface *et al* 1993). Besides being used to treat landfill leachate, they have been used to treat agricultural runoff, livestock wastewater, stormwater runoff, combined sewer overflow mine drainage, and domestic wastewater (Reed *et al* 1995; Wood 1999).

4.1.1 Advantages and Disadvantages of Constructed Wetlands (CW

The design and operation of this type of passive treatment system is notably different than more convention treatment systems. Brix (1987; 1993) and Reed *et al* (1995) detail several advantages of implementing a CW to provide advanced or tertiary treatment of municipal waste wastewaters:

- Low operating, energy and maintenance requirements;
- Efficient decentralized approach to wastewater treatment and control;
- Robust, low-rate process that is able to tolerate a wide range of operational conditions;
- Aesthetically appealing with potential for wildlife conservation. CW often have a higher productivity since they are typically more eutrophic than natural wetlands (Kadlec 1994); and
- Able to integrate into existing forms of effluent treatment.

On the other hand, using a CW imposes some constraints on the treatment options (Brix 1987; 1993; Reed *et al* 1995), such as:

- Four to ten times more land area is required for standard treatment and up to 100 times more if zero discharge is desired;

- Lack of standardized design and operational guidelines for various applications and treatment objectives;
- Limited phosphate and total nitrogen removal;
- Geographical limitations and availability of suitable plant species;
- Locating reasonably priced suitable permeable media for subsurface flow CW;
- Decrease efficiency during winter periods in temperate regions; and
- Planted vegetation may not survive due to toxicity of the effluent (Surface *et al* 1993).

Some other difficulties with these operations have been ensuring optimal flow of effluent through both surface water or bed media, harvesting and disposal of removed plant biomass, maintaining discharge standards due to natural fluctuations in oxygen and nutrient uptake by plants during diurnal and annual cyclical changes, and shortcutting the system via preferential flow patterns and therefore the possibility of missing aerobic zones (Robinson *et al* 1993).

4.1.2 Efficiency

Performance expectations and treatment efficiencies of CW depend on the design parameters, type of substrate and loading rates. Wood (1994; 1999), Reed *et al* (1995) and Vasel (2002) analyze the treatment performance of a variety of types of CW under different conditions and loading rates. Knight (1992), Robinson *et al* (1993), Wood (1999), the US EPA (2000) and also describe case studies using CW to treat effluents and the resulting efficiencies.

4.2 Types of Constructed Wetlands

There are two general types of CW: Free Water Surface (FWS) wetlands and Vegetative Submerged Bed (VSB) wetlands. Both mimic the aspects of natural wetlands and share many characteristics, but they differ in that in FWS the surface water comes in contact with the atmosphere and in VSB wetlands the water level is maintained below the surface of the bed (Reed *et al* 1995). Depending on the treatment objective desired these types could be modified or used in combination with other treatment options.

4.2.1 Free Water Surface (FWS) Constructed Wetlands

Free water surface (FWS) constructed wetlands (CW) closely resemble natural wetlands in appearance and function. As the name implies, FWS allow water to flow over the bed of the wetland and through the planted vegetation (Wood 1999). This design has been used in the Netherlands since the early 1960s and is one of the oldest concepts for the use of a CW for wastewater treatment (Brix 1993). The water depth is typically 0.3 meters but can range from a few centimeters to 0.8 meters or more depending on the wetlands purpose. It may include various combinations of open-water areas and fully vegetated surface areas. The bed itself is comprised of a low permeable soil to serve as a rooting media for the emergent aquatic vegetation. The design of the bed should include appropriate inlet and outlet structures and typically a liner to reduce hydraulic losses and prevent the polluted effluent from reaching the ground and groundwater. The actual shape, size and complexity of the system may vary depending on site characteristics and treatment objectives (Reed *et al* 1995; US EPA 2000).

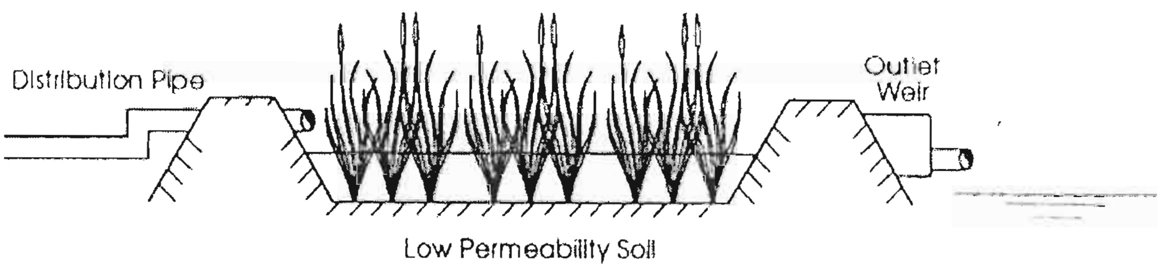

Figure 4.2.1: Typical cross sectional layout of a FWS constructed wetland (Kadlec and Knight 1996).

4.2.2 Vegetative Submerged Bed (VSB) constructed wetlands

VSB wetlands treat effluent by sending the water through a shallow, permeable medium (Wood 1999). This idea of treating wastewater in this manner was first developed in Germany in the 1970s (Brix 1993). Literature often refers to horizontal VSB, as Subsurface Flow (SF) (Reed *et al* 1995). The terms describe the same structure but for this dissertation they are referred to as VSB. These do not resemble natural wetlands for there should be no freestanding water. The design of VSB wetlands consists of a lined excavated basin usually 0.3- 0.6 meters deep and filled with a porous media (typically gravel) where the water level is maintained below the top of the media. VSB will vary in shape and size, type of treatment media and may include vegetation. If they do not include vegetation, they are essentially low-rate horizontal trickling filters (Lekven *et al* 1993). More detail of the design and operation of VSB is included in Section 5.2, which describes the pilot-scale study site used in this dissertation.

VSB can either be designed to have a horizontal or vertical flow. In the horizontal flow bed, the effluent enters the VSB through an inlet pipe and flows horizontally through bed substrate and subsequently through the plant root zone. (Reed *et al* 1995; US EPA 2000). Figure 4.2.2 demonstrates the effluent flow through a horizontal VSB.

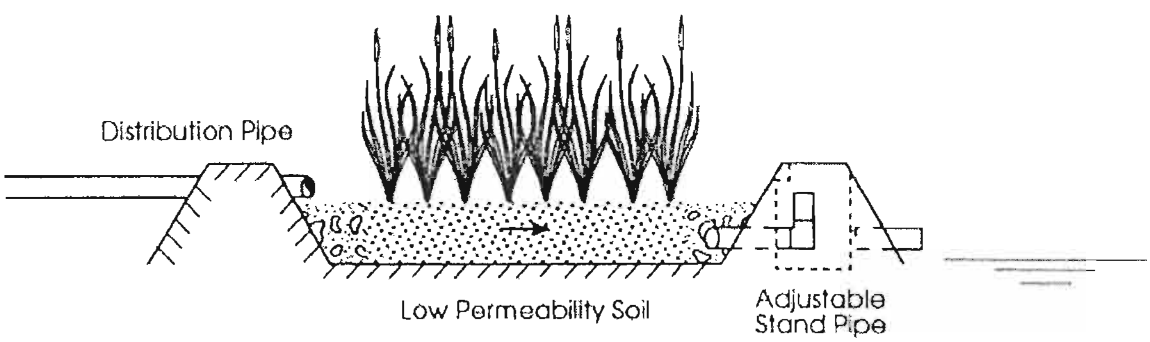

Figure 4.2.2: Typical cross sectional layout of a VSB constructed wetland (Kadlec and Knight (1996).

Vertical flow VSB wetlands are similar to horizontal flow systems, except that the effluent is applied uniformly over the top of the bed and therefore is allowed to filter vertically though the bed media instead of horizontally (US EPA 2000). Vertical flow sand or gravel filters (with or without aquatic plants) depends on its operation more as a filter with frequent dosing and draining cycles (Reed *et al* 1995).

4.2.3 The Choice of Constructed Wetland Types

There is no general consensus on which type of wetland is most advantageous (Wood 1999). Both FWS and VSB are considered to treat wastewater in a similar way to an attached growth reactor (Section 2.6 describes this type of treatment) since microbes are responsible for the majority of the pollution transformation in the wetland. The advantages of VSB are due to the wastewater being forced through the bed medium and not having freestanding water. Because of this characteristic:

- They can be smaller in area due to an increased surface area contact, and in turn increased reaction rates;
- Mosquitoes will not be able to use them as a breeding ground;

- They will have greater thermal protection in winter; and
- The lack of surface water will limit the desire for the public to have access.

The advantages of the FWS over the VSB are:

- Typically the FWS are less expensive to construct due to the high cost of obtaining and placing the gravel medium in the bed. This is particularly true for low-flow design requirements (Reed *et al* 1995).
- FWS potentially have simpler hydraulics because they rely on surface water flow (Wood 1999).

FWS systems are more common in the United States, while VSB systems are typically found in Europe, Australia and South Africa (Wood 1991). Overall the choice between the two is dependent on treatment objectives, land availability and construction costs (Reed *et al* 1995; Wood 1999; US EPA 2000).

4.3 CW Hydrology and Hydrological Design Characteristics

Hydrology is the key component of all types of wetlands (Mitsch and Gosselink 1993) and in the case of constructed wetlands, the hydrology is determined by the type of bed and local climate conditions. Therefore an understanding of the hydrology of the CW system is essential in the success of the treatment design (Reed *et al* 1995). This section gives a general overview of key hydrologic terms and descriptions of hydraulic design criteria as related to VSB.

4.3.1 Hydrological Balance

The general movement of water into and out of a CW is dependent on the system type and local climatic conditions. The over-all water balance may be expressed by Equation 4.1 (Kadlec and Knight 1996):

$$Q_i - Q_o + Q_c - Q_b - Q_{gw} + Q_{sm} + P \cdot A_S - ET \cdot A_S = \frac{dV}{dt} \qquad (4.1)$$

where ET = evapotranspiration rate, m/d

P	= precipitation rate, m/d
A_S	= wetland top surface area, m^2
Q_i	= input water flow rate, m^3/d
Q_o	= output water flow rate, m^3/d
Q_c	= catchment runoff rate, m^3/d
Q_b	= Infiltration rate out of the system through the side walls, m^3/d
Q_{gw}	= infiltration to groundwater, m^3/d
Q_{sm}	= snowmelt rate, m^3/d
t	= time, d

For the climatic conditions experienced in Durban and for the operational conditions used in this research, Equation 4.1 may be simplified to Equation 4.2, which describes a lined CW with no snowfall or peripheral cut off drains or walls (Olufsen 2003):

$$Q_i - Q_o + P \cdot A_S - ET \cdot A_S = \frac{dV}{dt} \tag{4.2}$$

4.3.2 Hydraulic Residence (or Retention) Time (HRT)

The hydraulic design of the CW is determined by the flow characteristics (i.e. VSB or FWS) and by the required treatment efficiency, which in turn dictates the hydraulic residence time (HRT) of the systems (US EPA 2000). The HRT is the measure of the average time taken for one constructed wetland bed volume to be replaced (Equation 4.3) (Kadlec and Knight 1996). The design of the CW is determined then by whichever pollutant requires the largest HRT to reach desired effluent concentration (US EPA 2000). While the HRT is the main criteria for sizing a CW, researchers have not agreed on the best approach for determining it (Reed *et al* 1995).

The theoretical HRT is the liquid volume of the wetland divided by the flow rate through it as seen in the following equation (US EPA 2000):

$$HRT = \frac{V}{Q} \tag{4.3}$$

where V = liquid volume, m^3

Q = average flow rate, m^3/d

The water storage (V) in the wetland may be calculated as (Reed *et al* 1995):

$$V = LWyn \tag{4.4}$$

where

L	=	length of wetland cell, m
W	=	width of wetland cell, m
y	=	depth of water in the wetland cell, m
n	=	porosity

Actual HRT has been reported to be 40-80 % less than the theoretical HRT. Complex flow models have been attempted but because of lack of data and varying conditions in VSB, there is little justification for using them (US EPA 2000). The true liquid volume in a CW is difficult to determine because of the loss of pore volume to roots and other accumulated solids. The lost pore volume varies seasonally and over the length of the media. Another problem determining the HRT is due to preferential flow as shown in Figure 4.3.1 (US EPA 2000).

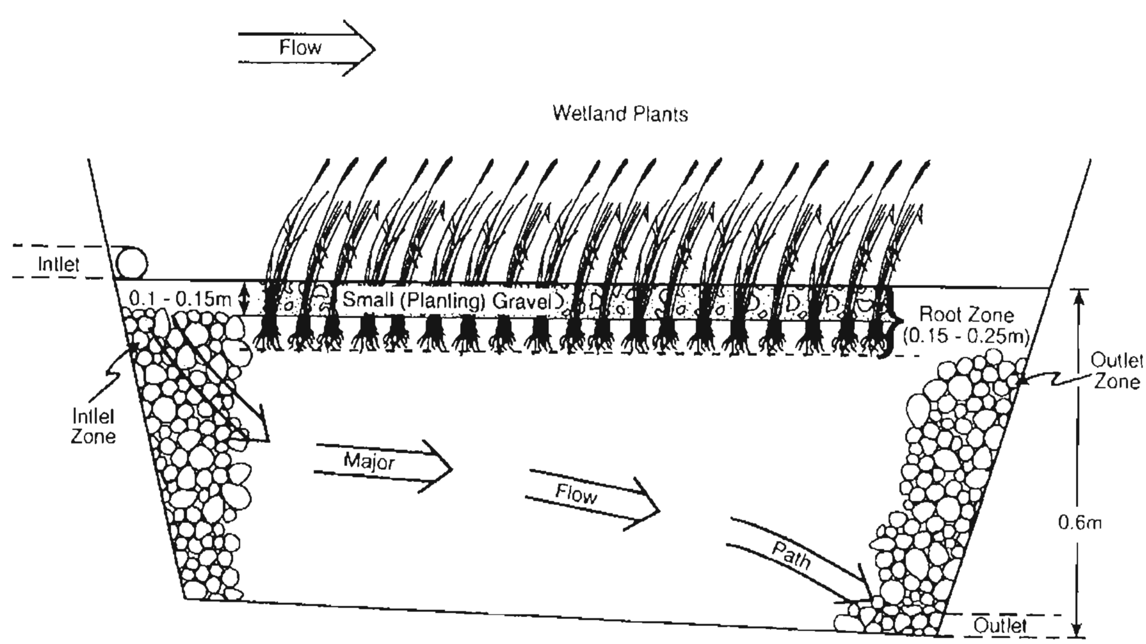

Figure 4.3.1: Preferential Flow in a VSB (US EPA 2000).

The US EPA (2000) suggests designers should assume that the lost pore volume is insignificant and therefore the theoretical HRT can be estimated using the average flow, system dimension, operating water level and the initial porosity of the medium. If accurate estimates of HRT are needed, a tracer study of the CW should be completed (US EPA 2000).

4.3.3 Precipitation and Evapotranspiration Impacts

Two local climatic factors that affect the design and treatment efficiency of the CW are precipitation and evapotranspiration (ET). Precipitation increases the water volume in the system thus diluting pollutants, raising the water level and decreasing the HRT. For most climates, precipitation will have a negligible impact on performance. If there is the likelihood of extreme rain events, they should be taken into account during the design in order to prevent surfacing of water in the VSB. If water is allowed to surface in a VSB, it will then shortcut the treatment processes and thus affect the quality of the effluent (US EPA 2000). Average monthly precipitation values should be used in determining the potential impact of precipitation on the CW (Kadlec and Knight 1996).

Evapotranspiration (ET) is the aggregate loss of water due to evaporation and transpiration from vegetation. While precipitation increases the water volume, ET decreases the level depending on climate, vegetation species and density, and moisture available at the evaporative surface. This results in a higher concentration of pollutants, a lower water level and an increased HRT. There are several ways to determine the ET rate, but because of the input data needed and the complexity involved, it has become common practice to use the simpler pan factor methods (Reed *et al* 1995; US EPA 2000). The simplest pan factor method assumes that the constructed wetland evapotranspiration is equal to 0.7 to 0.8 times the lake evaporation rate, which is also referred to as the Class-A-pan evaporation rate. The Class-A-Pan is standard meteorological site equipment made from 20-gauge, galvanized steel with a1.207 m internal diameter and a depth of 254 mm (Rogers *et al* 1985; Reed *et al* 1995; Kadlec and Knight 1996; SA DWAF 1998; US EPA, 2000). ET rates depend on plant species and density and therefore are difficult to determine. These rates must be seen solely as estimates for pan coefficients have been shown to be highly variable (Rogers *et al* 1985; US EPA 2000). Carter *et al* (1978) report estimates of ET from wetlands ranging from 0.54 to 5.3 times that of pan evaporation. In some literature the ET for VSB have been reported to be 1.5 to 2 times the pan evaporation rate (US EPA 2000). Although in the initial results from the previous study on the pilot scale CW used in this research, the ET was found to be almost half that of the Class-A-Pan evapotranspiration rate (Olufsen 2003).

4.3.4 Water Level Estimation (Flow modeling)

Estimating the water level within the CW is important to ensure that surfacing of the wastewater (thereby 'short-cutting' the system) does not occur (US EPA 2000). Water balance in a CW is difficult to estimate and evaluate under full-scale conditions because the leachate flow rates may show large seasonal variations (Vasel 2002). The water level can be estimated by examining the hydraulic flow in the wetland. Water flows in a VSB as it does through any porous medium: it is determined by the hydraulic gradient (slope) and hydraulic conductivity of the medium. Darcy's equation may be used to model this type of flow (US EPA 2000):

$$Q = K \cdot A_C \cdot S = K \cdot D_W \cdot \frac{dh}{dl} \quad (4.5)$$

or for a defined length of the VSB

$$dh = \cdot \frac{Q \cdot L}{K \cdot W \cdot D_W} \quad (4.6)$$

where Q = flow rate, m^3/d

K = hydraulic conductivity, m/d

A_c = cross-sectional area normal to wastewater flow, m^2

= W * D_w

W = width of VSB, m

D_w = water depth, m

L = length of VSB, m

dh = head loss (change in water level) due to flow distance,

S = dh/dL = hydraulic gradient, m/m

This form of Darcy's Law assumes laminar flow through a medium finer than coarse gravel. Some researchers have chosen to modify this equation to better reflect true flow through the VSB, but the US EPA (2000) still recommends using it without modifications. Reed *et al* (1995) suggest that the mathematical assessment of flow using this equation should be used with caution because it will not mimic the exact flow within the wetland.

The value of hydraulic conductivity (K) is critical in determining the flow through the wetland. It is difficult to determine for it will vary temporally and spatially in an operating VSB. It is affected by preferential flow patterns, by changes in root growth/death and by solid accumulation and degradation. Hydraulic conductivity has been shown to decrease with time and been shown to be less in the initial quarter to third of the wetland bed in comparison to the rest of the bed (US US EPA 2000)

Published data such as that listed in US EPA (2000) or estimates (Figure 4.3.4) can be used as guidelines (Reed *et al* 1995; Kadlec and Knight 1996) or it can be measured in the laboratory using standard SABS methods (SABS method 844, 1994; SABS method 845, 1994). These will give ideal hydraulic conductivities and therefore will not reflect the actual hydraulic conductivity in a treatment wetland. The US EPA (2000) recommends the following design values as conservative estimates of hydraulic conductivity:

- For initial 30% of VSB CW K = 1% of clean k
- For final 70% of VSB CS K = 10% of clean k

Clean refers to the ideal or theoretical values either taken from published data or measured in the laboratory.

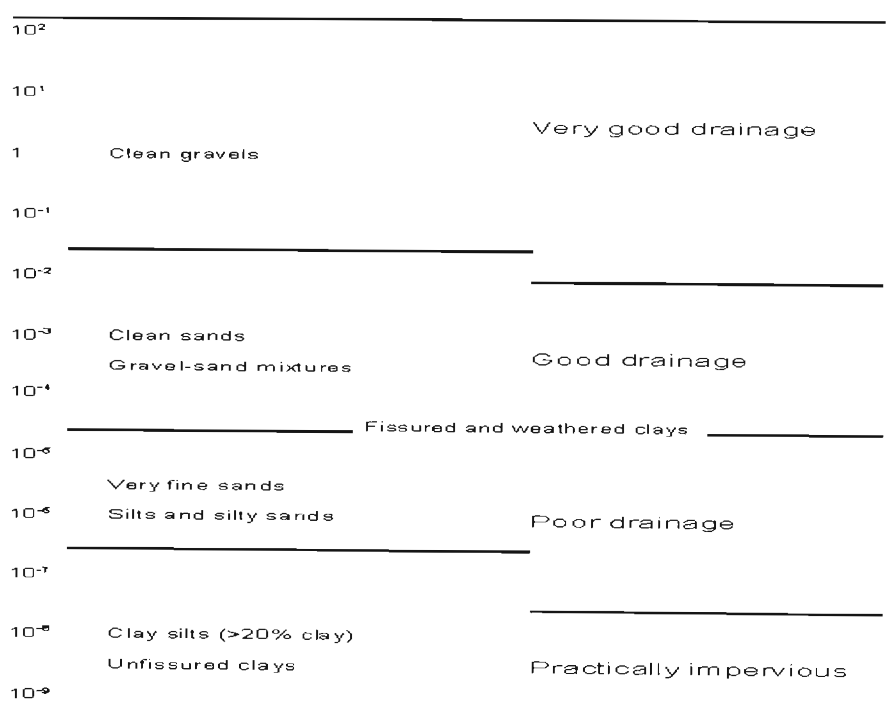

Figure 4.3.4: Average values of hydraulic conductivity (k) for various soils, note that the unit for hydraulic conductivity shown is m/s (Whitlow 1995).

4.4 Constructed Wetland Design Parameters

Designing a CW is determined by the type of effluent, concentration of pollutants and treatment goals. For example in comparison with CW used to treat raw sewage, the ones designed as a "polishing" treatment for effluents should be designed using coarser media beds in order to have a higher hydraulic conductivity and be able to allow a high mass loading rate for contaminates (Robinson *et al* 1993). It is critical that the design of the CW is appropriate to the strength of the influent. Overloading of organics may lead to clogging of the wetland, decreased efficiency and odors (Reed 1988). High organics (Armstrong 1990) and high ammonia could create a toxic environment for the plants and microbes (Surface *et al* 1993).

4.4.1 Sizing of the VSB Using Pollutant Loading Criteria

Currently there is no best approach for the design of CW (Tchobanoglous 1993) and there are disparities in design philosophy and treatment reliability of CW (Wood 1999). The choice depends on the cost restrictions (US EPA 2000), pollutant of interest, the removal efficiency needed and the data available (Reed *et al* 1995). There are three approaches currently used to determine the hydraulic design of a CW. The attached growth biological reactor approach which assumes that wetlands can be modeled as plug-flow reactors following first-order reaction kinetics, but these relationships have not been found to fit all the data available. Another method used is the multiple regression approach based on performance data from existing systems, but due to the variability of the systems the results of these comparisons have not demonstrated usable results (Reed *et al* 1995; US US EPA 2000). These methods were not used in the design of the pilot scale VSB used in this research, so the reader is referred to these references if more information is desired (Kadlec *et al* 1993; Reed *et al* 1995; Kadlec and Knight 1996; Wood 1999; US US EPA 2000).

The third method is the surface area or volumetric loading approach, which uses pollutant loading criteria (Reed *et al* 1995; US EPA 2000). The US EPA (2000) recommend that CW be designed using this method with the maximum pollutant loading rates that have been shown to meet effluent standards. This approach can take into account the variability and fluctuations that are expected when using dynamic natural systems. There are two types of pollutant loading rates that have been used by researchers to describe treatment performance: surface (or areal) loading rates (SLR or ALR), and volumetric loading rate (VLR). The measure of SLR determines the mass of a pollutant applied to the surface area of the wetland over a period of time.

$$SLR = \cdot \frac{C_i \cdot Q_i}{A_S} \qquad (4.7)$$

where SLR = Surface loading rate, $g/m^2/d$

Q_i = input water flow rate, m^3/d

C_i = Concentration of pollutant in the influent, g/m^3

A_s = Surface area of the VSB, m^2

The VLR is a measure of the mass of a pollutant applied to the pore volume of the wetland over a period of time:

$$VLR = \cdot \frac{C_i \cdot Q_i}{A_S \cdot y \cdot n} \qquad (4.8)$$

where VLR = volumetric loading rate, $g/m^3/d$

y = depth of water in the wetland, m

n = porosity

The SLR approach is based on the method used in land treatment systems; although in CW the wastewater is not applied uniformly over the treatment area and does not take into account water depth or temperature (Reed *et al* 1995), this measure can be used to yield plausible results (US US EPA 2000). The utility of VLR for design purpose is limited because the actual pore volume and HRT are seldom known (US EPA 2000). For these reasons the SLR approach was the one chosen for the design of the pilot scale VSB used in the study.

4.4.2 VSB Bedding Media and Layout

The choice of bed medium is dependent on the design function of the CW. It affects the retention time, the amount of surface area for microorganisms, and the availability of oxygen (Wood 1999). The bed medium determines the treatment efficiency of the system since the sediment surfaces are where most of the microbial activity affecting water quality occurs (Brix 1993). The VSB design depth is normally based on how deep the roots of the planted vegetation are likely to descend (Reed *et al 1*995). The following aspects should be taken into consideration when choosing the appropriate bed medium (Robinson *et al* 1993; Reed *et al* 1995; Kadlec and Knight 1996; US EPA 2000). The bed media should:

- Maintain the required range of hydraulic conductivity;
- Withstand clogging (Some media such as soil and sand are not appropriate media for they have been shown to clog systems);
- Function as a rooting material for vegetation;
- Provide surface area for microbial growth; and
- Be able to filter out particulate matter.

The medium may be chosen to obtain specific treatment need, such as phosphate or ammonium removal, but the removal capacity of the substrate is limited and will not contribute to treatment in the long-term (US EPA 2000).

As seen in Figure 4.4.22, the typical layout of a VSB is divided into three zones: the inlet zone, the treatment zone, and the outlet zone. The media used within these zones may vary in size and type within the CW bed (US EPA 2000).

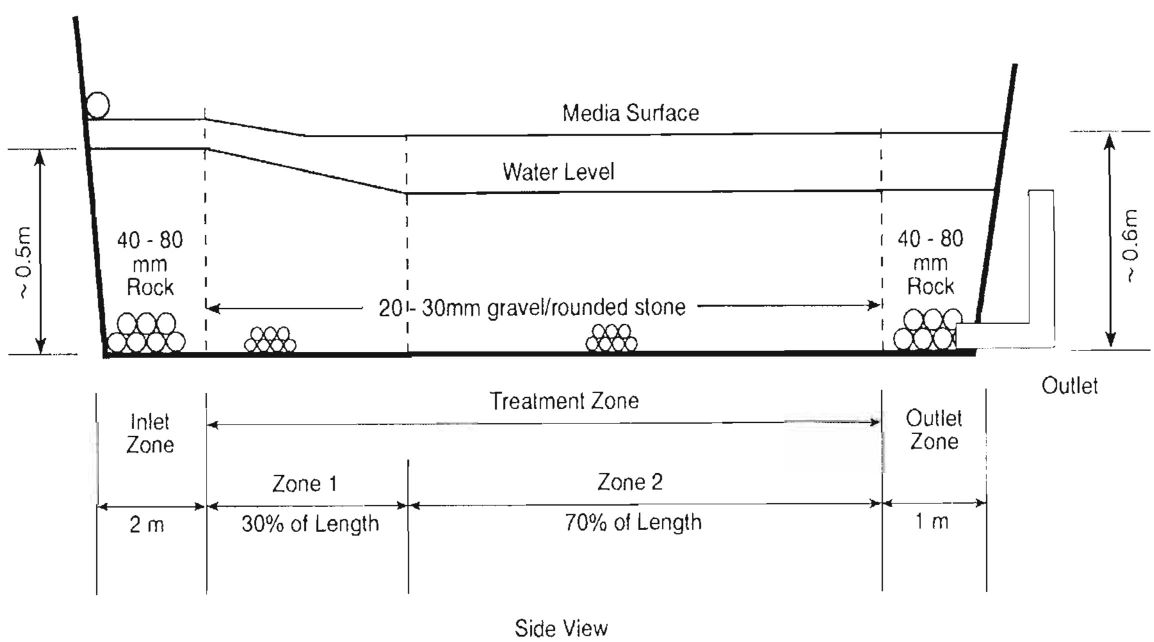

Figure 4.4.2: VSB system zones (US EPA 2000).

The bed medium provides a surface for microbes and a medium for plant growth. It also functions as a purifier through both its physical and chemical processes. Since the media affects the retention time, the surface for microbes, and the availability of oxygen, it plays a significant role in treatment (Wood 1999). Pollutants in the leachate can be immobilized in the soil or in the plants (Robinson *et al* 1993), but this immobilization is considered in steady-state equilibrium and therefore does not contribute to the overall removal process (Vasel 2002). As with organic removal, if the organic and hydraulic loads are low, the CW acts as low-rate nitrification biofilter. Therefore both ammonia and nitrates can be absorbed to the organic and inorganic fractions within the soil/plant microbial matrix (Wood 1999).

4.4.3 Vegetation

Vegetation should be chosen based on its availability, its environmental value and its species treatment ability (Wood 1999; Reed *et al* 1995). Using local indigenous hydrophtes can satisfy these first two parameters. These plants are readily available and adapted as part of the local ecosystem. While using local indigenous vegetation will aid the likelihood of survivability, this also depends on the environmental conditions within the wetland. Table 4.4.3 shows some common parameters that should be considered when choosing an appropriate plant species.

Table 4.4.3: Environmental conditions for the survival of specific macrophytes.

Macrophyte	Temp (°C)	Salinity tolerance (g/l)	pH range	Reference
Typha angustifolia	10-30	15-30	4-10 3.7-8.5	Reed *et al* (1995) US EPA (1988)
Typha latifolia	10-30	<1	4-10 3-8.5	Reed *et al* (1995) US EPA (1988)
Scirpus acutus	18-27	0-5	4-9	Reed et al (1995)
Scirpus validus	18-27	0-5	4-9 6.5-8.5	Reed *et al* (1995) US EPA (1988)
Scirpus lacustris	18-27	25	4-9	Reed *et al* (1995)
Phragmites communis	12-23	<45	2-8	Reed *et al* (1995)
Phragmites australis	12-33	<45	2-8	Reed *et al* (1995)

			3.7-8	US EPA (1988)
Juncus spp.	16-26	0-25	5-7.5	Reed *et al* (1995)
Carex spp.	14-32	<0.5	5-7.5	Reed *et al* (1995))

The third parameter, treatment capability or role, is contested in the literature. Numerous studies comparing treatment performance of those systems with and without plants have concluded that there is a higher efficiency when plants are present (Wetzel 1993; Kadlec and Knight 1996). Other studies, however, have demonstrated no significant difference in performance (US EPA 2000; Olufsen 2003). While their significance in treatment is still under investigation, plants can affect the treatment process in a number of ways:

- Provide a surface for the microbes -
  These microbes facilitate the transformation a number of chemical constituents in the effluent (Kadlec and Knight 1996). This characteristic allows the wetland to function as an attached bioreactor.
- Uptake of nutrients -
  Plants are a seasonal sink for nutrients in a wetland and therefore may be considered a removal mechanism if they are harvested (Reed *et al* 1995). Due to the costs of harvesting and the ineffectiveness of significant nutrient removal (since the majority of nutrients are located in the root tissue (Wetzel 1993)), studies have shown that substantial removal only occurs in systems with low nutrient loadings (Brix 1994).
- Release of organic carbon -
  During plant senescence there is potential for the organic carbon that is returned to be used to support denitrification if environmental conditions are favorable. Approximately 5-9 grams of carbon is needed to denitrify 1 gram of NO_3 (Reed *et al* 1995).
- Provide thermal insulation -
  Planted VSB systems have been shown to have the capacity to insulate the waste water during cold weather, but this has not been shown to make a significant difference in treatment ability even during the winter (US EPA 2000).
- Introduce oxygen into the bed media -
  Oxygen may 'leak' from the rhizomes into the anaerobic environment. These aerobic microzones that surround the rhizomes would create an oxidizing environment and support aerobic microbes (Brix 1993; 1994). The rate of oxygen release varies depending on environmental conditions (such as temperature), oxygen demand of the aerobic microbes,

and the type of vegetation (Kadlec and Knight 1996). Brix (1993) measured a release rate from 0.5 to 5.2 $g/m^2/d$. The function of vegetation as oxygen suppliers is inconclusive and the discussion surrounding this possible function is described in more detail in Section 4.5.4.

- Provide aesthetics -
  A vegetated wetland has more aesthetic appeal than a simple soil or gravel filter designed for similar purposes (Wood 1999).
- Limit nuisance insect development -
  Having plants results in plant litter that can absorb any water that has ponded on the surface and thereby eliminate a breeding habitat for mosquitoes and gnats (Wood 1999).

Having plants in the CW can also assist in filtering suspended solids. The plants and their corresponding litter do have a mitigating effect on odors released from the leachate (US EPA 2000).

The most commonly used plant species in CW are macrophytes; they are chosen for variety of reasons (Kadlec and Knight 1996):

- They are hydrophytes that are adapted to saturated soil conditions;
- They have vascular tissue that transports oxygen from the atmosphere to the plant's rhizomes and roots;
- They are aesthetically pleasing; and
- They have high 'habitat' value by providing nesting opportunities and food sources for animals.

There is no best choice species to be used. The most common *are Phragmites spp, Typha spp., Scirpus spp., Juncus spp.*, and *Carex spp* (Reed *et al* 1995). Typical species used in South Africa are *Typha spp* (Wood 1999).

Macrophytes can remove pollutants by directly assimilating them in the tissue and providing surface and a suitable environment for facultative microbes (Brix 1993). Studies have shown that plants do have limited nutrient (nitrogen and phosphorus) uptake (Wood 1994; Rogers *et al* 1995), but even then those assimilated nutrients are released back into the CW during plant senescence (Richardson 1985). The significant part of the removed nitrogen removed by the plants would remain in the CW in the form of refractory nitrogen contained in the plant litter (Brix

1997; IWA 2000). The inputs of nitrates may be partially removed if the plants are harvested; Reed *et al* (1995) report that harvesting removes less than 20% of total nitrogen. When considering harvesting as an option, one should examine disposal options of the harvested material. When plants are not harvested the estimated value of nitrogen intake may average 10% of the total nitrogen removed (IWA 2000).

If macrophytes are not needed in a CW (i.e. a difference in the system with our without macrophytes is not noticeable), then their natural function in a wetland has been either bypassed or overloaded. Wetzel (1993) states that in this case CW are unnecessarily complex attached-growth reactors and should not be complicated by being converted into wetlands. More efficient and manageable systems such as trickling or other filtration systems should be used if the function needed is one of a reactor

.

4.4.4 The Biotic Element In and Around CW

As in natural wetlands, a wide variety of organisms ranging from bacteria to mammals can exist in a CW. Each has a function in the wetland ecosystem, but that may or may not contribute directly to the treatment efficiency. They may in fact be detrimental to the system so operation considerations must be addressed during the design of a CW. Table 4.4.2 from the US EPA (2000) summarizes animal species that may be found in CW and considerations that should be made regarding them. In the case of VSB, the US EPA (2000) notes that only avian species play a significant role, but other researchers (Reed *et al* 1995; Olufsen 2003) also include parasitic insects such as aphids as influences to the wetland ecosystem that may significantly affect treatment efficiencies.

In CW, especially VSB, it is the microbial ecology that plays the most significant role in the transformation of nutrients and organics in the wastewater. Both nitrogen and phosphorus uptake by the plants depends on microbial activity. Microbes also consume organic compounds while releasing carbon dioxide during aerobic respiration, or carbon dioxide, hydrogen sulfide, and methane during anaerobic respiration (US EPA 2000). These processes are discussed in more detail in Section 4.5, which describes the pollution removal mechanisms.

Table 4.4.4: Characteristics of Animals Found in Constructed Wetlands (US EPA 2000)

Animal Group	**Members commonly found in CW**	**Function or importance to Treatment Process**	**Design & Operational Considerations**
Invertebrates, including protozoa, insects, spiders and crustaceans	A wide variety will be present but diversity and populations will vary seasonally and spatially	Undoubtedly play a role in chemical and biological cycling and transformations and in supporting food web for higher organisms, but exact functions have not been defined.	Mosquito control must be considered, mono-cultures of plants are more susceptible to decimation by insect infestations.
Fish	Species adapted to living at or near the surface; species adapted to living in polluted waters	Consumers of insects and decaying material (e.g. mosquitofish eat mosquito larvae)	Anaerobic conditions will limit populations; nesting areas required; bottom-feeders can uproot plants and resuspend sediments.
Amphibians and Retiles	Frogs, alligators, snakes, turtles	Consumers of lower organisms	Turtles have an uncanny ability to fall into water control structures and to get caught in pipes, so turtle exclusion devices are needed; monitoring of control structures and levees for damage or obstruction is needed.
Birds	A wide variety (35-63 species) are present, including forest and prairie species as well as waterfowl, but diversity and populations vary seasonally and spatially.	Consumers of lower organisms	Heavy use, especially by migratory waterfowl, can contribute to pollutant load on a seasonal basis.
Mammals	Small rodents (shrews, mice, voles); large rodents (rabbits, nutria, muskrats, beaver); large grazers (deer); large carnivores (raccoons, foxes)	Consumers of plants and lower organisms	Nutria and muskrat populations can reach nuisance levels, removing vegetation and destroying levees; structural controls and animal removal may be required

4.4.5 Environmental Consideration in Design

Although CW are artificially engineered treatment systems, they are based on natural processes and located in natural environments. The benefits of the use of natural systems for treatment is that they do not require added energy sources or chemicals, but since they rely on the natural processes, they will not behave with the reliability of engineered systems. Also they will be influenced by meteorological processes, such as evapotranspiration, temperature, precipitation and solar radiation. These processes are cyclic, both diurnally and annually, influencing the constructed wetland's overall performance to follow the same cyclic trends (Kadlec 1999).

Diurnal and annual cycles also control the life-death cycle of the vegetation and dependent organisms. These cycles have a direct effect on the treatment efficiency of the CW. In order to continually meet the water quality standards for the effluent the effect of the processes must be included during the design of the system (Kadlec 1999; US EPA 2000).

4.5 Pollution Removal Mechanism in Subsurface CW

The removal of pollutants in constructed wetlands is not based solely on the individual mechanisms of the fauna, microflora or the inert media, but on the function of each as part of the entire ecosystem of the constructed wetland. It is the synergistic effects of the biological, physical and chemical interactions that allow the constructed wetland to be used in wastewater treatment. As wastewater moves through a VSB, contaminants are removed through separations and/or transformation processes. Separations typically include gravity separations, filtration, absorption, adsorption, ion exchange, stripping and leaching. The contaminants may also undergo chemical transformation through oxidation/reduction reactions, flocculation, acid/base reactions, and precipitation. Some will be transformed by microbes in biochemical reactions occurring in aerobic, anoxic or anaerobic conditions (Brix 1993; US EPA 2000). As the chemicals change, some of the end products will be gases and as such be removed from the wastewater. Others may be transformed in a way that does not achieve the treatment objectives. For example a biochemical reaction may produce biomass or organic acids that could then be released in the effluent (US EPA 2000). In general CW are effective in treatment of wastewater, but when systems do not perform as efficiently as intended it is typically because of the lack of understanding of the mechanisms involved in meeting the treatment objectives (Wood 1999; Rogers *et al* 1985; Vasel 2002).

4.5.1 Organic Matter

Organic matter in the effluent can be comprised of a range of organic constituents from those that are readily biodegradable to the highly refractory substances. All these compounds are broken down by microbial degradation, but they differ in the rate of degradation. The rate is also affected by temperature, oxygen concentration, pH, nutrient availability and substrate concentration because these are the general parameters that affect the metabolic rate of the aerobic microbes involved (Rogers *et al* 1985). The pollution removal mechanisms for organics in CW are like those of any low-rate, attached-growth biofilter (Wood 1999). Organics are removed in a VSB via physical separations and biological conversions (Rogers *et al* 1985). The bed media and plant roots may filter out some of the particulate organic matter and refractory organics. Then they may either be resuspended in the water column (US EPA 2000) or decompose very slowly within the sediments (Tchobanoglous *et al* 1979).

Filtration may remove some of the organic fraction, but biological conversions are the most important removal mechanism for the organics (US EPA 2000). The organic pollutants are degraded by microorganisms that use the plants, sediments, and bed media as a surface (Wood 1996). These organisms consume the organics to sustain life and to reproduce (US EPA 2000). The end products of these biological reactions depend on the terminal electron acceptors. For aerobic reactions, oxygen is the acceptor and the end products are mineralised products, gasses and new biomass. Anoxic reactions use nitrates, sulphates or carbonates as the terminal electron acceptors and the end products are the reduced versions of the acceptors (e.g. nitrogen oxides, free nitrogen, sulphur, and thiosulfate). Some biomass is also produced, but since the reaction is less efficient than the aerobic reaction, less biomass is produced per unit of substrate converted. The least efficient reaction is anaerobic metabolism, where organic matter is the electron acceptor and donor (US EPA 2000). Since these transformations result in new biomass, organics may continue to be included in the effluent but there will be a higher percentage of refractory compounds or products from wetland primary production (Rogers *et al* 1985).

The amount of organic matter in wastewater can be measured a number of ways as shown in Table 4.5.1 (US EPA 2000). Analyses are typically performed on the aggregate amount of organic matter rather than on individual compounds. This will not supply information on specific organic molecules or their fate in the treatment processes, but these analyses can assess the general polluting potential of the organics (US EPA 2000).

Table 4.5.1 Analytical techniques used to measure the amount of aggregate organic matter in waste water (Rogers *et al* 1985; Kadlec and Knight 1996; US EPA 2000).

Analytical measurement	Definition of measurement	Concerns with measurement
BOD_X	Biological oxygen demand measures the oxygen used in the breakdown of organic matter and in the oxidation of inorganic substances over x number of days. It shows the amount of oxygen that will be removed from water as the pollutants are transformed.	It is a dynamic measurement conducted over a finite timeframe, and it may or may note include nitrogenous oxygen demand
COD	Chemical oxygen demand is measure of the total quantity of oxygen required to oxidize all organic material into carbon dioxide and water.	It does not differentiate between biologically available and inert organic matter, and there is no simple way to relate measurements since they measure different organic constituents.
TOC	Total organic carbon is measured by chemical oxidation followed by analysis for carbon dioxide.	

According to Vasel (2002) there is a lack of information regarding the removal efficiency of refractory COD in CW and even less about the removal mechanisms involved. For full-scale CW, treatment efficiencies from low-loaded systems (18-21.6 g COD/m^2/d) were in the range of 3-50%. Pilot-scale systems with slightly lower COD concentrations (5-6.3g/m^2/d) resulted in 28-54% efficiencies (Vasel 2002). This is a similar result to the 2001 study using the VSB at Bisasar Road Landfill site. The leachate used then had a mass of 5 g/day and the mass removal efficiency was found to be generally between 30 and 40% (Oulfsen 2003). BOD removal rates tend to be constant at 70-90% regardless of the load once it exceeds 300 kg/ha/d (Knight 1992). The majority of soluble organic compounds are degraded aerobically by attached bacteria, although in some cases anaerobic degradation may be significant (Brix 1993).

4.5.2 Nitrogen

The most important forms of inorganic nitrogen in CW are ammonia (NH_3^+), nitrate (NO_3^-), nitrous oxide (N_2O) and dissolved nitrogen gas (N_2). Nitrogen may also be in many organic forms: urea, amino acids, amines, purines and pyrimidines (Kadlec and Knight 1996). Biologically controlled transformations of nitrogen (i.e. nitrification and denitrification) are the most influential mechanisms for nitrogen removal (Brix 1993; Reed *et al* 1995), but some organic nitrogen may be removed through physical separation followed by ammonification of the settled sediments (US EPA 2000). If the goal of the system is to remove nitrogen, it should be designed to promote sedimentation and denitrification (Johnston *et al* 1993).

As described in Section 2.5.1, ammonia can be removed by being nitrified (i.e. be biologically oxidized) to nitrate under certain environmental conditions. Available oxygen is the critical parameter to have an effect on the rate of nitrification; it requires 4.3 grams of dissolved oxygen and 7.14 grams of alkalinity to nitrify 1 gram of ammonia. Other factors such as pH, temperature, alkalinity and ammonia concentration also influence the process (US EPA 2000). High ammonia concentrations or the presence of any constituent at levels toxic to the nitrifying bacteria will also impact the rate, but in the case of leachates, there are typically no toxic substances to restrict the nitrification bacteria (Vasel 2002). As mentioned in the Section 2.5.1 when discussing the nitrogen cycle, ammonia can also be volatilized as nitrogenous gases to the atmosphere at high temperature and elevated pH. This may occur in CW during active photosynthesis (Kadlec and Knight 1996 and US EPA 2000), but there is very little research that has quantified removal rates (Vasel 2002). Some ammonia may be removed through ion exchange within the bed media, but this will be a short-term loss and last only until the exchange capacity has been depleted (Reed *et al* 1995; US EPA 2000).

Nitrate can be removed by denitrification to nitrous oxides and nitrogen gas. This process requires, and is often limited by, available organic carbon. In order for the denitrification process to be financially feasible it must not require an external source of carbon. Instead it should rely on the available organic load, which is significant in landfill leachate especially in young leachate (Reed *et al* 1995). A varying amount of carbon can also be provided by the plants and plant litter (Van Oostrom 1995). Requirements for denitrification (Haandel and Marais 1981; Wood 1994) are:

- Presence of a facultative bacterial population immobilized in the bed media;
- Presence of nitrate (the electron acceptor) in an aqueous solution;
- Absence of dissolved oxygen;
- Suitable environmental conditions for growth of microorganisms;
- Absence of inhibitory toxic substances; and
- Presence of carbon (or electron donor). A minimum theoretical value of around 3 grams of biodegradable COD is needed for each gram of NO_3^- (Vasel 2002)

Some of the nitrogen gas formed as an end product of denitrification may be converted back to organic nitrogen through nitrogen fixation (US EPA 2000), but in general denitrification has been shown to the be largest potential source of nitrogen removal (Johnson *et al* 1993).

Anaerobic ammonia oxidization is another process that may explain nitrogen removal. Hill *et al* (2001) have identified anamox or anamox-like bacteria in CW. As Vasel (2002) notes, even if this process is found to be significant, it still requires the oxidation of ammonia to NO_2^- prior to being used by these bacteria.

While early studies suggested that significant amounts of nitrogen could be removed, subsequent and full-scale studies of VSB systems have not been shown to significantly nitrify or denitrify the influent (US EPA 2000). Ammonia transformation rates tend to be most limited by the lack of available oxygen which is found in VSB constructed wetlands. CW can be designed to remove trace amounts of nitrogen through design or incorporating devices to enhance aeration (Robinson *et al* 1993). If nitrification is needed for the treatment and the use of a CW is desired, FWS should be considered because it has naturally higher ammonia removal rates. Often this issue is solved by using the CW treatment process in conjunction with aerobic treatment processes, such as SBRs or aerobic lagoons, as described in Section 2.6.

4.5.3 Other Pollutants

CW have been shown to remove other pollutants such as metals, phosphorous and even pathogens. These pollutants, however, were not of interest in this research because they do not appear in significant concentrations in landfill leachate. The reader is referred to the references for more information (Rogers *et al* 1985; Reed *et al* 1995; Kadlec and Knight 1996; Robinson *et al* 1998; Wood 1999; and US EPA 2000).

4.5.4 Debate over the Oxygen in System

Several of the pollutant removal mechanisms in the process rely on the amount of oxygen that can penetrate the anaerobic environment of the subsurface wastewater. It has been theorized that there can be direct exchange of oxygen at the surface of the bed (Robinson *et al* 1993), but in the case of VSB there should not be any surfacing of wastewater. In general, the focus has been on the idea that plants (and dead stalks of reed plants) can transfer a significant amount of oxygen through the root systems into the wastewater, thus creating aerobic pockets in the bed.

The anaerobic liquid could then pass through these aerobic oxidizing microzones. In this way, ammonia can be oxidized into nitrates. The nitrate can then be used by the plant or may undergo reduction in the anaerobic soil, and then released as nitrogen gas (the product of denitrification).

There is debate on how significant the vegetation's role is in creating these aerobic zones via oxygen exchange. It often claimed that plants could provide adequate oxygen via its root zone, thereby creating aerobic conditions needed to degrade certain pollutants (i.e. ammonia). Through experience with VSB, there is only a nominal amount of oxygen released by plants to the area around its roots, but it seems to be insufficient in delivering the amount of oxygen needed for any significant reduction in ammonia through nitrification (Armstrong 1990; Brix 1992; 1993). The possible range of values for the amount of oxygen that reaches the anaerobic wastewater is large and depends on the type of CW (Robinson *et al* 1993; US EPA 2000). The transfer rate for oxygen has not been determined directly, but is estimated from the stoichiometry of BOD and nitrification mass balances. This is assuming that the total nitrogen removed is due to nitrification and none is lost via stripping, and the flow rate at the inlet and outlet are the same (IWA 2000). While there is some oxygen that is leaked from the plant roots into the surrounding soil (Brix 1997), CW are primarily an anaerobic treatment process. If the oxygen demand of the effluent is low, then there may be sufficient amounts for treatment, but if the wastewater is more contaminated, the systems could be overloaded and the treatment process will be impaired or fail completely.

4.6 Constructed Wetlands as Appropriate Leachate Treatment

In attempts to promote sustainability not only are landfills being modified to be "sustainable" as described in Section 3.5, but the desire is also to incorporate sustainable leachate treatments (Vasel 2002). While the management and rate of production of leachate contains aspects of sustainability, the actual treatment process itself does not, unless it can be designed and maintained with a sustainable leachate collection and drainage system. When the definition is stipulated by the use of treatment systems, it assumes that the leachate collection systems and liners will last for the polluting life of the landfill. This could continue indefinitely, but the integrity of the barrier system will not (Mathlener 2001). The leachate treatment systems are only as sustainable and reliable as the life of the liners and drainage systems. Therefore the choice of leachate treatment does not make a landfill sustainable, nor can the treatment option be sustainable in isolation. The goal then should be to incorporate *appropriate* rather than *sustainable* leachate treatment.

The US EPA (2000) states that technology can be considered appropriate if it meets the following key criteria: It must be –

- Affordable – The total costs (including initial capital and annual operating and maintenance costs) are within the user's ability to pay;
- Operable – The local workforce must have the skills and knowledge to be able to operate and maintain the systems; and
- Reliable – The effluent quality requirements can be consistently met

4.6.1 Affordability

Typically leachate is released untreated to the local environment or sent to the local wastewater treatment plant. Traditional approaches to treat landfill leachate, such as conventional onsite treatment, are undesirable because their operation and maintenance is costly and will continue to be required after the landfill closure. As previously mentioned, discharge leachate to the sewer system is also costly (Surface *et al* 1993), may be dangerous (methane explosions) and is not always a local option (Robinson 2001). The focus then is choosing the most inexpensive option that can reliably meet the required standard (Robinson 1999).

Since the majority of conventional treatments require continual operating and maintenance costs, there has been an increased focus on using passive treatments that do not rely on active operations, making them more affordable during the aftercare of the landfill. The concern with affordable post-closure care is a result of the uncertainty regarding the time period required to achieve stability and the other aftercare requirements. While the body of knowledge regarding the biochemical processes during the life of a landfill and the modeling of these processes over time even during the aftercare period is growing (Vasel 2002), there are no definite timeframes. The aftercare period can range from minimum of 30 years (SA and USA standards) to indefinitely depending on concentrations of chemicals. Given the high costs of most treatment options and the time involved for the aftercare period, many municipal solid waste (MSW) landfill companies want to implement a technology that has a low-cost and can successfully treat leachate for years after the landfill has closed. For these reasons there has been an increased interest in designing and using constructed wetlands as part of leachate treatment processes (Surface *et al* 1993; Vasel 2002). Constructed wetlands (CW) have been shown to have low construction and maintenance costs (Brix 1993) except when local land is expensive which is an important cost consideration since a large amount of land is required (US EPA 2000).

4.6.2 Operability

Constructed wetlands (CW) are considered a passive post-treatment option because they rely on the natural pollution removal mechanisms, so they require less inputs and control than conventional systems. Overall they are considered to have low operating, energy and maintenance requirements (Brix 1987 and Reed *et al* 1995), and can be established and run by relatively untrained personnel (Brix 1993). These systems will require some monitoring and general maintenance, but the operations do not demand the engineering skills needed by other treatment options. The major operation and management requirements will be (Wood 1999):

- Maintaining the hydraulic controls and structural integrity of the wetland;
- Managing the vegetation and the wildlife as needed; and
- Monitoring the water quality parameters in the effluent and changes in appropriate legislation.

The US EPA (2000) suggests that they are appropriate, even when there is a dearth of skilled labor.

4.6.3 Reliability

Constructed wetlands (CW) have been found to be successful in treating certain landfill leachates (Brix 1993; Robinson *et al* 1993; Wood 1999; US EPA 2000). The reliability of a CW will be site specific due to the differences in leachates and standards that the effluent must meet, and the type of CW design implemented. Treatment options should be appropriate to the type of leachate being generated as the leachate changes overtime due to waste degradation. CW are currently being used to treat leachate at various stages with varying degrees of success (Vasel 2002). CW can withstand varying loading rates and are generally more flexible than conventional systems (Brix 1993). This is critical for leachate quality is highly variable and the quality of leachate will change with age, therefore the treatment must be flexible or mutable.

While CW have shown success with treating leachates, they should not be used to treat raw acetogenic leachates (from young landfills) due to the high levels of biodegradable COD and ammonia (Maehlum 1995; Vasel 2002). Other treatment options should be used during the acetogenic period instead of or prior to the use of constructed wetlands (Vasel 2002). As the waste undergoes decomposition the strength of leachate will decline, in which case CW may provide a low-cost, long-term treatment option if used in isolation. However it is likely that the high ammonia levels in raw leachate may limit the treatment capabilities of a CW. If that is the case the CW should follow an initial aerobic stage, until the ammonia concentration falls to values where a CW in isolation can provide adequate treatment. This may take decades and/or require the use of a large land area (Robinson *et al* 1993).

4.6.4 Appropriate Treatment in the South African Context

As described in Section 2.7, there are many options available for the treatment of landfill leachate. The cost of the containment and treatment must be appropriate to the situation at a specific landfill. Many sites in South Africa are in water deficient areas so will not produce a significant amount of leachate. Therefore, expensive containment and treatment systems are not required. Those that do produce significant volumes will have a greater pollution potential and therefore the Minimum Requirements (SA DWAF 1998) stipulate barrier and treatment systems (Ball 2002). As different landfill leachates will require different containment and treatment options due to the varying degree of decomposition and moisture content, different cells within a landfill may also require different treatment options. In the case of Bisasar Road landfill, the chemical composition varies between the older, unlined cells and the new containment cells.

At the Bisasar Road Landfill site, in order to meet South African discharge standards in a cost and technology effective way, pilot-scale biological treatment processes were chosen as an appropriate technology to treat the leachate. This type of biological treatment process has been able to reduce or remove pollutants efficiently (Ehrig and Stegmann 1992; Robinson *et al* 1997). Pilot-scale Sequencing Batch Reactors (SBR) were designed to treat the complete mixture of leachate from all the cells within the Bisasar Road landfill. The SBR were found to be successful in achieving complete biological removal of nitrogen and some decrease in COD levels from 1076 mg/liter to 526 mg/liter (Strachan *et al* 2000a). The COD levels were still higher than the level required by the General Standard for effluent (75 mg/L COD), so pilot-scale VSB were added as a final stage polishing treatment to remove this residual COD. VSB were chosen because they have been shown to be successful in removing trace amounts of pollutants from low strength raw leachates and treated effluents from biological treatment plants (Robinson 1993; Robinson *et al* 1997, 1998; Cossu *et al* 1997). The VSB were found to be easy to operate and inexpensive in comparison with other treatment options, but they could not reliably reduce the COD concentration to a level required by the General Standard (Trois *et al* 2002). The researchers have noted this was not a fault with the CW, but because the organics are mainly unbiodegradable fulvic and humic acids and therefore difficult to reduce to the required levels.

As previously mentioned, CW are not a reliable treatment option for young leachates due to the high organic and ammonia loads, but they have been shown to be successful in treating methanogenic leachate (Robinson *et al* 1997). For this reason, they were chosen as an appropriate treatment technology for leachate from an older, unlined cell at the Bisasar Road landfill.

Chapter Five

Study Site and Experimental Procedures

5.1 Study Site

5.1.1 Bisasar Road Landfill

Bisasar Road landfill is situated ten kilometers from the central business district of Durban, South Africa (Plate 5.1.1), and was established in 1980. It is one of the busiest sites in South Africa: receiving an average of 3000 tons of waste per day from the Durban Metropolitan Area (DMA). The total airspace capacity of the landfill is twenty-one million cubic meters and it is predicted to service the area until 2016 (Strachan *et al* 2000a; Trois *et al* 2000). Details of the local climate are discussed in Section 5.2.3.

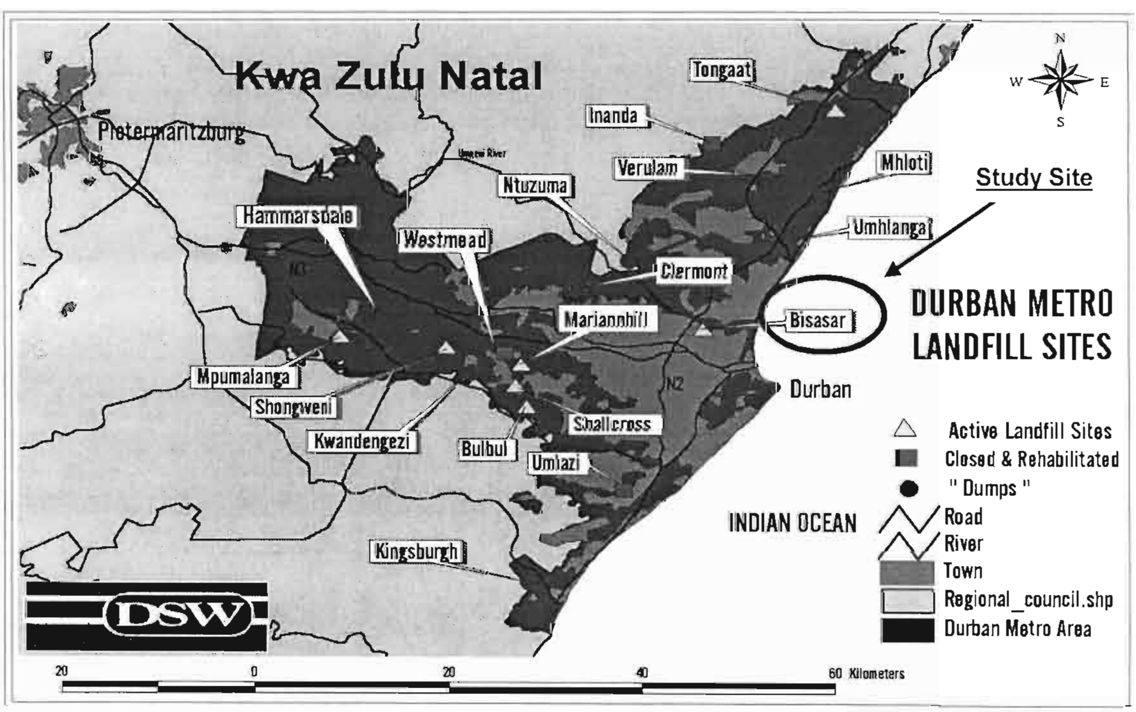

Plate 5.1.1: All the landfill sites in the Durban Metropolitan Area with the Bisasar Road Landfill site highlighted (courtesy of Durban Solid Waste).

As mentioned when discussing landfill design and management (Section 2.7), Bisasar Road Landfill contains both older, unlined areas that are attenuation areas within the landfill, as well as newly lined containment cells that rely on liners and leachate drainage systems. Leachate generated in the new lined cells of the landfill is collected by collection blankets under the waste body. In the unlined cells, there is a significant amount of leachate produced, and it is extracted by a combined eductor and pumping system via gas wells that extend to the full depth of the waste (±40m) (Strachan 1999; Griffith and Trois 2002). Leachate that is not extracted in the wells is partially intercepted by a sub-soil drain at the base of the stability berm. The leachate used in this study was taken from this sub-soil drain because it is the oldest leachate in the landfill and therefore more representative of the leachate that will be produced during the aftercare period. The chemical composition of this leachate is listed in Table 5.1.1. Table 5.1.1 also compares this leachate with the full Bisasar Road leachate and with similar leachates in the United Kingdom, which represents typical leachates found in developed countries. The Bisasar Road Landfill discharges from 250,000 to 500,000 liters of leachate per day into the local sewerage system. This high flow releases an ammonia discharge of 600 kg/day, which is equal to that of the discharge from a population of 350,000 people. There is concern that the already high levels of ammonia will continue to increase until they exceed the capability of the sewage line (Strachan *et al* 2000b).

Table 5.1.1: Comparison of three methanogenic leachates from municipal solid waste landfill sites.

Parameter	**Bisasar Road Leachate** [(b)]	**Leachate used in this study**	**UK leachates** [(c)]
pH	8.0	8.00	7.52
Alkalinity (as $CaCO_3$)	6440	1526	5376
NH_4-N	1274	132	889
BOD_5	320	181	374
COD	2427	390	2307
BOD_5/COD	0.13	0.45	0.16
Chlorides	1790	868.59	2074
Sulphate (as SO_4)	48	30*	67
Conductivity (uS/cm)	1291	646	11502
Cadmium	<0.01	<0.05*	no data given
Nitrate/Nitrite	27	15	1.05
Lead	0.02	<0.1	no data given
Sodium	897	771*	1480
Magnesium	56	97	250
Potassium	1022	245*	854
Calcium	36	75	151

Chromium	0.05	<0.10*	0.09
Manganese	0.12	1.04*	0.46
Iron	2.70	0.40	27.4
Nickel	0.09	<0.10*	0.17
Copper	<0.01	<0.10*	0.13
Zinc	0.08	0.13*	1.14

Note: (a) Results in mg/liter except pH and conductivity

(b) Typical characteristics of the Bisasar Road landfill site (Strachan 1999)

(c) Summary of composition of methanogenic leachates sampled from 29 large, relatively dry landfills in the UK with a high waste input rate (Robinson and Gronow 1995).

(d)* Mean of two samples taken from first batch of raw leachate from the oldest section of the Bisasar Road landfill

As described in Section 3.3.1, in South Africa the regulations that determine if a landfill will require a leachate barrier and a leachate drainage system are dependent on the classification of the landfill (SA DWAF 1998). This classification is determined by the type of waste received, the total size of the landfill, and water balance of the landfill. The type of waste at each site is described as general (G), low hazardous (L), or hazardous (H) waste. The size of the landfill is stated as being a small (S), medium (M), or large (L). As mentioned in Section 2.2, the water balance is also a factor in classification. Landfills are either sited in a water positive area (B+) or water negative area (B-). The Bisasar Road landfill is classified as a GLB+ landfill.

Plate 5.1.2: Bisasar Road Landfill Site (courtesy of Durban Solid Waste)

5.1.2 Initial Biological Treatment Research

As previously stated, the Bisasar Road Landfill is considered an old landfill and the leachate reflects an established methanogenic phase (described in Section 2.3.4). The leachate from Bisasar Road contains high concentrations of ammonia and refractory organics (Strachan *et* *al* 2000b; Olufsen 2003). In 1999, research into methods of reducing the nitrogen concentration in the leachate was undertaken by Durban Solid Waste (DSW) in collaboration with EnvirosAspinwall of the United Kingdom (UK). A pilot-scale sequencing batch reactor (SBR) was chosen as an appropriate treatment method. Through the studies carried out by Strachan (1999) and Olufsen (1999), it was shown to be able to completely remove ammonia and residual nitrogenous compounds and to have a reasonable reduction in COD levels (Strachan *et al* 2000b). While this process was successful, the COD levels were not reduced enough to meet the General Discharge Limit of 75 mg/liter required for discharge into local watercourses (SA Government Gazette 1999). This was mainly due to the refractory nature and low biodegradabity of the organics as seen by the low BOD:COD ratio in Table 5.1.1.

In order to reduce this organic concentration, constructed wetlands were chosen as an appropriate passive post-treatment for final "polishing". In 2001, four pilot scale vegetated submerged bed (VSB) constructed wetlands (CW) were designed and constructed at the Bisasar Road landfill to determine if the COD concentration in the effluent from the SBR could be further degraded. Due to the low biodegradability of the organics in the leachate and the high evapotranspiration rate of the VSB, effluent discharge limits could not be met by using this system (Olufsen *et al* 2001). This VSB treatment system has been dormant since the end of the pilot scale studies in September 2001.

One of the original objectives of this research was to use this system to study the efficiency of the VSB in denitrification of the leachate. This focus was chosen as a possible alternative method for denitrification because the carbon-sources used in the SBR for denitrification are expensive. This required modifying the SBR from a unit that completed both nitrification and denitrification to one solely for nitrification. Until that could be completed, the VSB was fed with previously treated leachate. There was a shortage of previously treated leachate, therefore the treatabity trials were conducted on one VSB. The VSB planted with *Phragmites australis* was chosen because this species is commonly used in constructed wetlands. Before the SBR could be reinstated, the treated leachate was exhausted. Due to time constraints, it was decided to bypass the use of the SBR and refocus the study. Instead of using nitrified leachate, raw leachate from the oldest section of the landfill was chosen as the VSB influent. This methanogenic leachate has a fairly low ammonia concentration (245 mg/liter), and once diluted it should not create a toxic environment in the VSB. The new objective became to study the efficiency of the pilot scale VSB in the removal of pollutants (specifically nitrogen and organics) from older, methanogenic leachate. In order to accomplish this the design of the pilot scale VSB and the previous experimental procedure used in the 2001 study was reviewed. For more details and background information of the 2001 VSB pilot scale study see Trois *et al* (2002) and Olufsen (2003).

5.2 Summary of Design

The purpose of the use of the VSB was to remove the organics that remained in the leachate after treatment in the SBR. The majority of the organics in the leachate are refractory (as noted by the low BOD:COD ratio) and therefore will be degraded extremely slowly (Cossue *et al* 1992; Robinson 2001). In order to remove the refractory organics, the VSB was designed based on the need for a large hydraulic retention time (HRT) and low surface loading rate (SLR). The beds were sized using the rational surface area loading (SLR) approach (discussed in Section 4.4.1) based on a COD SLR of 3 g/m^2/day. The COD loading rate from the diluted influent (a 1:1 ratio of leachate to borehole water) was 7.2 g/day with a flow rate of 20 liters/day. This flow rate was chosen because it was dependent on the effluent from the pilot scale SBR, which was 10 liters/day. The loading and flow rate required a surface area of 2.4 m^2. A length to width ratio of 3.75 to 1 was chosen to lessen the possibility of shortcutting the system. With this ratio, the treatment zone needed to have a width of 0.8 meters and a length of 3 meters. The addition of the 0.5m inlet and 0.5m outlet structures added one meter to the total length. A depth of 0.7 meters was chosen to allow for unrestricted root growth. Two sampling pipes were placed in the middle of the bed: one at 1.3 meters from the inlet and the other 2.6 meters from the inlet. Table 5.2.1 summarizes the design parameters and overall layout dimensions. Figure 5.2.1 shows a schematic drawing of the flow through the VSB with system dimensions.

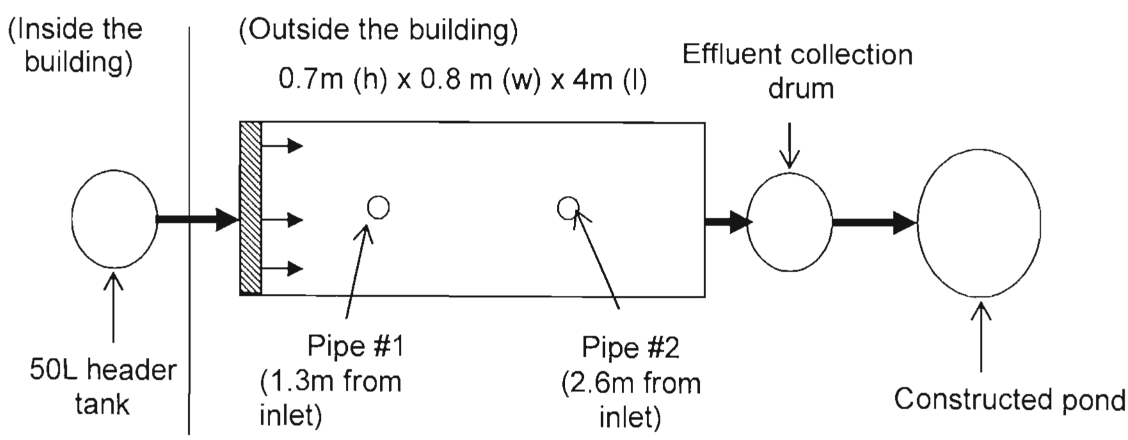

Figure 5.2.1: A schematic drawing of the flow pattern of the leachate and the dimensions of the system. Drawing not to scale.

Table 5.2.1: Design summary and overall layout of the Vegetative Submerged Bed (VSB) Constructed Wetland (CW) (modified from Oulfsen 2003)

Parameter	Design Value	Comment
COD SLR	3 $g/m^2/d$	For treatment zone
Total cell length	4 m	-
Inlet zone length	0.5 m	-
Outlet zone length	0.5 m	-
Treatment zone length	3 m	-
Cell width	0.8 m	-
Total surface area	3.2 m^2	-
Treatment surface area	2.4 m^2	-
Treatment zone depth	0.2 m	-
Rooting medium depth	0.5 m	-
Total cell depth	0.7 m	-
Length to width ratio	3.75 : 1	For treatment zone
Design freeboard	10 mm	From top of wetland surface
Design ET rate	4 mm/d	-
Design porosity	30 %	-
Design hydraulic conductivity	150 m/d	For treatment zone
Design head loss	2.3 mm	Over total cell length
Design influent flow rate	0.02 m^3/d	-
Design effluent flow rate	0.0072 m^3/d	-
Average daily flow rate	0.0136 m^3/d	-
Design superficial velocity	0.085 m/d	-
Design HLR	8.3 mm	For treatment zone
Design HRT	10.6 days	For treatment zone
Treatment zone stone size	13.2 mm	-
Rooting medium mix	50% top soil / 50% 13.2 mm stone	
Vegetation	*Phragmites australis*	-

Part of the treatment system was located inside an adjacent building (Plate 5.2.1). It included two 1000-liter storage tanks: one for the raw leachate and the other for onsite borehole water and a 50-liter high-density polyethylene drum header tank that allowed a batch feed for the wetland. The influent entered the inlet system (Plate 5.2.2) where it was evenly distributed over the width of the wetland (as suggested by Wood (1999)). It then flowed through the system until it reached the outlet (Plate 5.2.3) where it was collected in an effluent collection drum, which was then manually poured into a constructed pond (Plate 5.2.4).

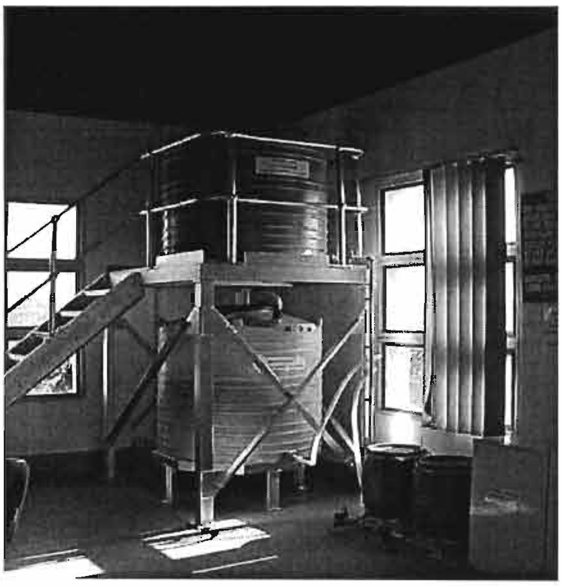

Plate 5.2.1: Leachate (bottom) and borehole water (top) storage tanks and header tank (far right)

Plate 5.2.2: Inlet distribution svstem.

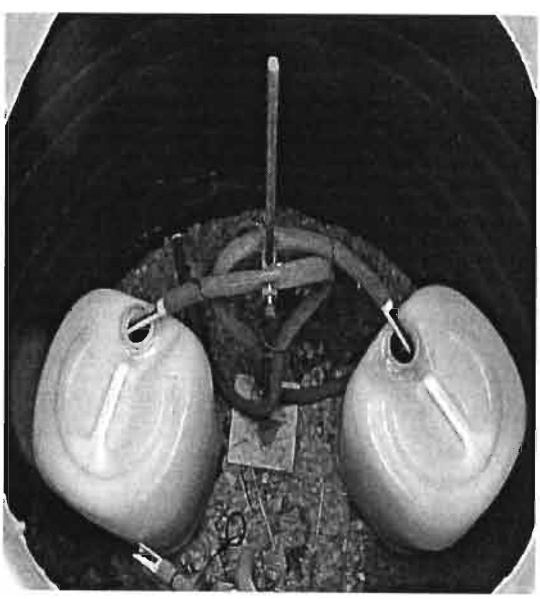

Plate 5.2.3: Effluent collection drums in the manhole tank.

Plate 5.2.4: Picture of the outside treatment system including the CW, manhole, and pond

The constructed ponds were two meters in diameter and were designed to simulate a local natural receiving environment. These constructed ponds were initially filled with spring water and planted with local vegetation (*Juncus krausii, Zantesdeschia aethiopica, Papyrus spp and Nymphaea alba),* and fresh water fish species (*Tilapia Sparmannii and Poecilia veticularus*) (Plate 5.2.5). Their function was to qualitatively assess whether the effluent from the VSB had a negative impact on the flora or fauna in the ponds.

Plate 5.2.5: The constructed receiving ponds immediately after planting (Olufsen 2003).

The depth of the bed was divided into two sections. The bottom layer was 0.5 meters deep and comprised of a mixture of 50% topsoil and 50% 13.2 mm stone to encourage root growth. On top of this was a 0.2 m gravel layer comprised of 13.2 mm stone. The bed was a prefabricated 5 mm thick HDPE box placed in a constructed brick shell with a flexible geomembrane liner. A geofabric barrier was installed at the outlet system to limit the amount of particulate matter that could reach the outlet. Figure 5.2.2 shows a schematic cross sectional drawing for the unplanted VSB.

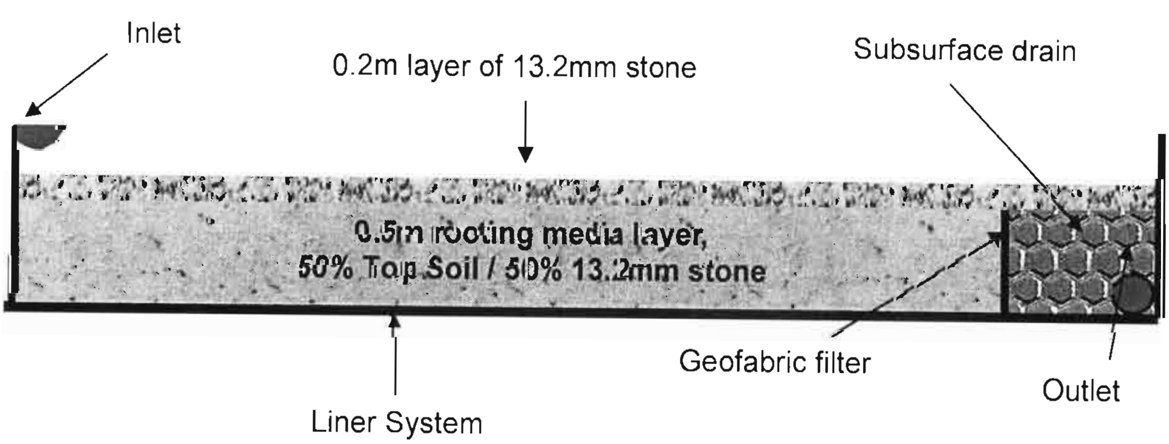

Figure 5.2.2: Schematic cross section of the VSB (Olufsen *et al* 2001).

5.2.1 Planted Macrophyte Vegetation

The original research design of the 2001 study examined the treatment efficiency of four VSB (Olufsen 2003). One was left unplanted as a control and the other three were planted. Two were planted with grass species: *Vetiveria zizanioides* and *Leersia hexandra*, and the other one was planted with the reed *Phragmites australis*. There was no significant difference in treatment efficiency between the VSB (Olufsen 2003), so only the *Phragmites australis* treatment VSB was used in this study due to an initial limited supply of treated leachate from the SBR.

Plate 5.2.6: Fully grown *Phragmites australis*.

This VSB was chosen out of the four because *Phragmites australis* is a common wild reed found worldwide. A concern is some consider it an invasive pest species in natural wetlands. CW containing *Phragmites australis* have been shown to successfully remove BOD, phosphorous, ammonia, iron, manganese and potassium from landfill leachates in the United States and Europe (Surface *et al* 1993). This species was selected due to its high tolerance of a wide range of environmental conditions. The species optimum pH range is 2 to 8 and it can withstand a salinity concentration of up to 45 g/liter. *Phragmites australis* growth is rapid and spread laterally via rhizomes. It can survive permanent inundation and draught conditions. The low food and habitat (mainly use as nesting cover) value for most birds and animals make it less prone to damage caused by animals, but it is not desirable if one of the goals for the CW is to increase local habitats (Reed *et al* 1995). *Phragmites* has deep root penetration in gravel (0.4m), and its extensive roots have been thought to give better treatment performance due to enhanced oxygen supply (Reed and Hines 1993).

5.2.2 Modifications from the Initial Experimental Study

The VSB was originally designed to reduce the organic concentration in a pretreated leachate. The full design parameters used in that 2001 study are listed in Table 5.2.2

For the purpose of this experiment the following modifications were made:

- An increased influent rate from 20 liters to 40 liters while still maintaining the 1:1 dilution ratio of leachate with borehole water
- An increased pollutant load due to using raw leachate instead of treated leachate. The comparison between the contaminant load in the leachate used in the 2001 study (which was the effluent from the SBR) and the one used in the 2003 study (which used raw, methanogenic leachate) can be seen in Table 5.2.2.
- Only the VSB containing *Phragmites australis* was used instead of the three planted and one unplanted beds used in the 2001 study.

Table 5.2.2: A comparison of the previously treated leachate used in the 2001 study (Olufsen 2003) and the raw methanogenic leachate used in this 2003 study.

Parameter	2001 study	2003 study
Alkalinity (as $CaCO_3$)	776	1526
Ammonia (free)	0.85	132
BOD_5	7.1	181
Calcium	no data	72
Chloride	1017	869
COD	271	390
Conductivity	421	646
Magnesium	no data	101
Nitrate + Nitrite	1.63	15
Ortho phosphate	no data	3.74
pH	8.5	8.0

Note: (a) Results in mg/liter except pH and conductivity

5.2.3 Local climatic conditions and modeling

Local climatic conditions affect the performance of any constructed wetland (CW), but it is especially significant on a small scale CW such as the one used in this study. The climatic data used in the design of this project was taken from the South African Weather Bureau. Ideally the precipitation data for the local area should be used due to the variability of precipitation in space and time. However, these data were not available and an overall monthly average for the Durban area was used. The monthly total for precipitation and the Class-A-Pan evapotranspiration were compared and the average monthly water budget for Durban was determined and presented in Table 5.2.2 (Olufsen 2003).

Table 5.2.3: Total average monthly precipitation and predicted evapotranspiration values for the Durban area (raw data from the South African Weather Bureau)

Months of the year	Rainfall	A-Pan	Difference in Rainfall and ET
Jan	134	203	-69
Feb	113	182	-69
Mar	120	184	-64
Apr	73	139	-66
May	59	111	-52
Jun	28	92	-64
Jul	39	103	-64
Aug	62	129	-67
Sep	73	139	-66
Oct	98	165	-67
Nov	108	175	-67
Dec	102	210	-108

Note: All results in mm/month

From the data results seen in the water budget (Table 5.2.3), it shows that the monthly ET followed an expected seasonal cyclical trend with the highest rates occurring during the summer months (November to March). It also shows that Durban is a water deficient area. Given the high ET rates for the area, the water loss from the VSB would be substantial and therefore it was included as part of the experimental parameter. It also is an important consideration when comparing the concentration of the pollutants in the influent and the effluent.

For design purposes an ET rate was estimated as 4 mm/day: the annual average rate of ET when using the Class-A Pan evaporation rate multiplied by 0.8. By using the design evapotranspiration rate of 4 mm/day and the treatment surface area of 2.4 m^2, it was determined that there would be a loss of 9.6 liters/day over the treatment area. This high ET loss in combination with the small scale of the system has a net concentration effect on the pollutants in the leachate (Olufsen 2003). To minimize the impact of the high ET loss and to dilute the pollutant concentrations, twenty liters of borehole water were added to the mix. Without this dilution the ammonia concentrations of the raw leachate may have created a toxic environment for the plants.

5.3 Operations

Before the treatment trial began the VSB was filled to 10 mm below the surface with water in order to prepare the system for use. Forty liters of water were fed daily into the wetland for two weeks beginning on February 3, 2003. On February 17, 2003 the first round of treatment trials began using leachate that had undergone both nitrification and denitrification in the pilot-scale sequencing batch reactors (SBR). This batch of previously treated leachate was manually mixed with twenty liters of water in the 50-liter header tank. The VSB was then fed from this header tank every afternoon when the timed solenoid valve opened. The influent was evenly distributed across the width of the VSB. Once the liquid reached the outlet, the effluent from the VSB was collected in 25-liter drums and emptied manually into the receiving pond. The effluent from the receiving pond flowed into the nearby sewer line.

On March 6, 2003 the treated leachate was exhausted. As mentioned previously, it was decided to bypass the use of the SBR and instead raw leachate from the oldest section of the landfill was chosen to be used in the study. The raw leachate was pumped from the sub-soil drain described in Section 5.1.1 and stored in the 1000-liter storage tank. On March 26, 2003 the second round of treatment trials began using this new batch of raw methanogenic leachate. As with the first round, 20 liters of raw leachate and 20 liters of borehole water were mixed in the header tanks and the CW was fed daily with this new diluted leachate mix. Three separate batches of untreated leachate were used in this study due to the limited holding capacity of the storage tank. All three batches were collected from the same sampling point in the landfall.

5.4 Sampling

Sampling began on February 27, 2003 and the samples were taken every Thursday for the duration of the study. For the first month of testing (until April 17), samples were taken of the raw leachate influent, from pipe #1 (1.3 m from the inlet), and from pipe #2 (2.6 m from the inlet). From April 24 to July 31, samples were taken of the raw leachate influent, from pipe #1 and from the outlet. The reason for this is due to design hydraulic retention time in the VSB.

5.5 Testing

All chemical testing was completed at the accredited Durban Metro Water Services laboratory. The following is a list of the parameters analyzed and the number of the ASTM Standard Method used (Clesceri *et al* 1993):

- Alkalinity = 2320 (B)
- Ammonia (NH_3)= 4500-NH3 (H)
- BOD = 5210 (B)
- Ca= 3120 (B)
- Chloride= 4500-Cl (G)
- COD= 5220 (C)
- Conductivity = 2510 (B)
- Iron (Fe) = 3120 (B)
- Lead (Pb) = 3120 (B)
- Magnesium (Mg) = 3120 (B)
- Nitrate/Nitrite= 4500-NO3 (I)
- Phosphate (PO_4)= 4500-P (G)
- pH= 4500-H (B)

Chapter Six

Results of the VSB Treatability Trials

6.1 Presentation of the Data

In each graph the results of the effluent analysis determined on days 10, 17, 38 are the results of the analysis from the samples taken from pipe #1. These samples were used instead of those from the outlet because during these first few weeks the effluent would consist of mainly water due to the design retention time.

As mentioned in Section 5.3, four separate batches of leachate were used during the treatability trials. Since the CW was operated as a batch reactor, the changes in concentration over time are reported in batches with the first feeding time for each batch noted in each graph at day 38, day 94, and day 129. The dates of each batch feeding and their corresponding dates in the study are listed in Table 6.1.1.

Table 6.1.1: Listing of the dates within each batch of leachate and a comparison between actual date and day in the trial.

Batch 1		Batch 2		Batch 3		Batch 4	
20-Feb	3	27-Mar	38	22-May	94	26-Jun	129
27-Feb	10	3-Apr	45	29-May	101	3-Jul	136
6-Mar	17	10-Apr	52	5-Jun	108	10-Jul	143
13-Mar	24	17-Apr	59	12-Jun	115	17-Jul	150
20-Mar	31	24-Apr	66	19-Jun	122	24-Jul	157
-	-	1-May	73	-	-	31-Jul	164
-	-	8-May	80	-	-	-	-
-	-	15-May	87	-	-	-	-

6.2 Hydraulic Balance

The treatment trials were carried out from February to July 2003, encompassing both the end of the rainy season (summer) and the beginning of the dry season (winter). In South Africa, February through April is the summer season and May through July is considered the dry season. This is reflected in Figure 6.2.1, which shows the precipitation experienced at the

Bisasar Road Landfill site over the course of the six months during treatability trial. This figure shows the daily mean rainfall that occurred in Durban during each week of the trial (Raw data presented in Table C1, Appendix C). The majority of the rain experienced at the landfill occurred during week 5 and week 9. Large rainfall events such as these have a significant effect on small-scale constructed wetland systems.

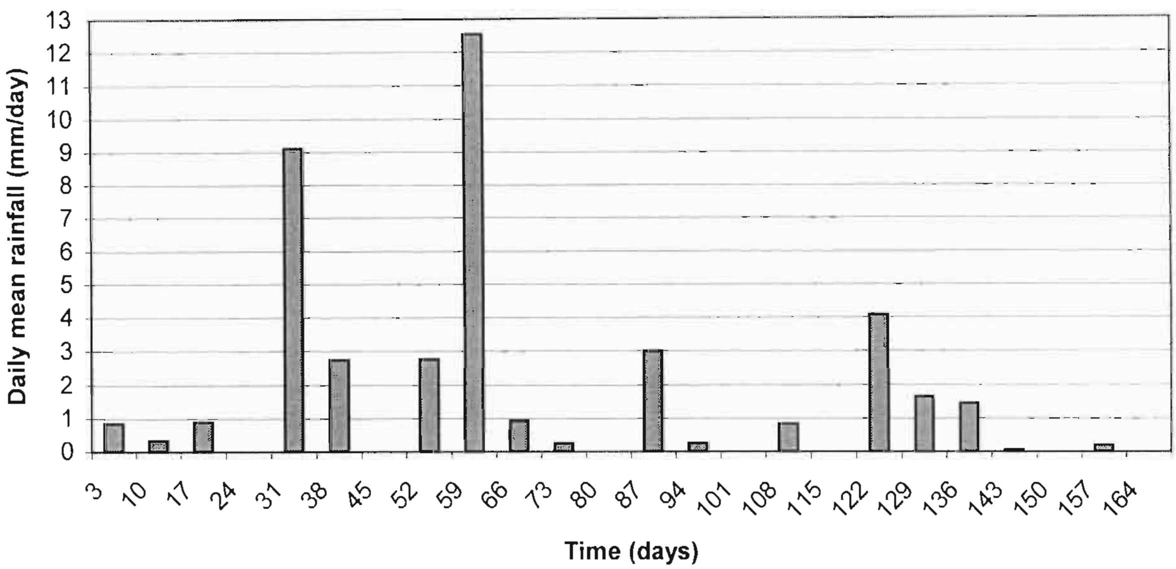

Figure 6.2.1: Weekly mean daily rainfall experienced in Durban, SA during the 24-week treatability trails (courtesy of the South African Weather Bureau). Day 1 was February 17, 2003 and Day 165 was August 1, 2003.

Evapotranspiration (ET) is another climatic factor that has a significant impact on the treatability of the CT. The ET data used in this study was taken from the measured ET rates experienced during the 2001 treatability trials (Oulfsen 2003). These values are assumed to be representative of the ET during 2003 because both treatability trials occurred during the same seasons and had similar rainfall. The results of the 2001 measured ET are presented in Figure 6.1.2 (Raw data presented in Table C2, Appendix C). There were erratic variations in the evapotranspiration data for the *Phragmites australis*, which did not show any seasonal correlation. The ET rate ranged from 2.48 mm/day to 5.97 mm/day with the mean ET rate being 4.57 mm/day. This loss due to ET reduced the leachate from 40 liters/day to a mean of 25.38 liters/day.

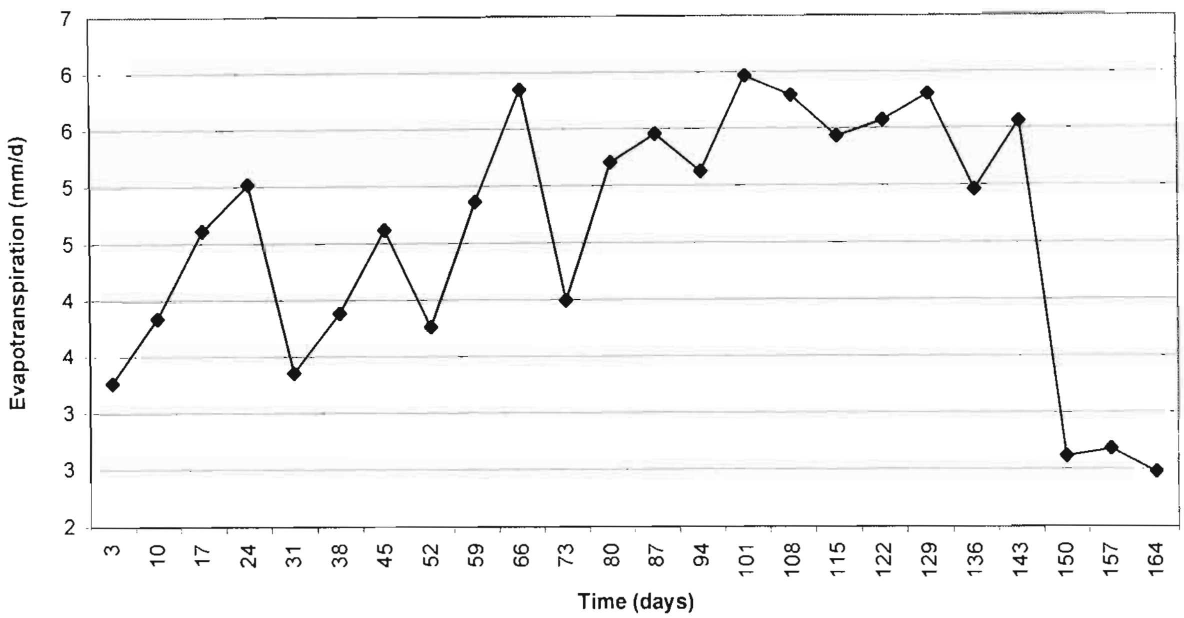

Figure 6.2.2: *Phragmites australis* evapotranspiration rates (Oulfsen 2001)

6.3 Characteristics of the Influent

Table 6.2.1 lists the parameters monitored during the treatability trials. The characteristics of the influent varied slightly between the four batches of leachate used. The largest variation occurred between the first batch of leachate, which had undergone nitrification and denitrification previously in the SBR, and the raw leachate which was used in the other three batches. Another reason for the large standard deviation is the changes that occurred over time as the leachate remained in the storage tank prior to being fed into the wetland.

Table 6.3.1: Concentration of the VSB CW influent

Parameter	Mean	# Of Samples	Standard deviation
Alkalinity (as $CaCO_3$)	1526.23	22	365.35
Ammonia (free)	132.26	22	97.87
BOD	181.67	18	48.54
Calcium	71.86	22	26.35
Chloride	868.59	22	338.50
COD	390.41	22	130.99
Conductivity	646.64	22	109.36
Iron	0.39	22	0.34
Lead	0.07	22	0.05

Magnesium	101.00	22	31.53
Nitrate + Nitrite	14.78	22	17.88
Ortho phosphate	3.74	22	4.74
pH	8.00	22	0.27
BOD/COD	0.47	18	0.09

Note: 1. Results in mg/liter except pH (unitless) and conductivity (mS/m)

2. Analyses conducted by Durban Metro Water Services Laboratory.

6.4 Influent and Effluent Concentrations

The key parameters in this study were nitrogen, in the form of ammonia and nitrates/nitrites, and organics, as determined by BOD and COD. The results of these analyses are presented in Figures 6.3.1 to 6.3.4 (Raw data presented in Table D1, Appendix D). They show the changes in the concentrations of the influent and the effluent for each parameter with time. The general description of the presentation of the graphs is described in Section 6.1.

6.4.1 Ammonia

Since one of the main pollutants of concern in landfill leachate regardless of the age of the waste is ammonia, one of the research objectives was to determine if the VSB CW would be able to reduce the ammonia concentration sufficiently in order to meet the 3 mg/liter South African General Discharge Limit. The influent had a mean ammonia concentration of 132.26 mg/liter and an effluent mean of 15.37 mg/liter. The results of the ammonia concentration are presented in Figure 6.4.1 (Raw data presented in Appendix D).

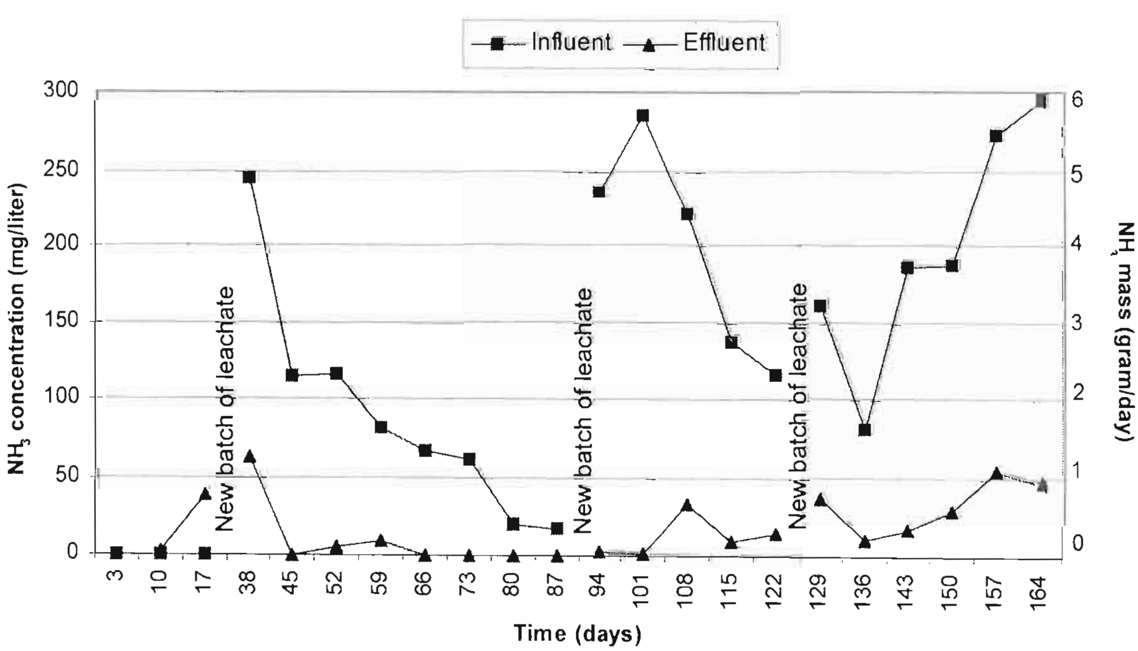

Figure 6.4.1: Ammonia concentrations and mass of the influent and effluent.

It is evident from Figure 6.4.1 that the ammonia concentrations in the influent were variable throughout the final three batches of treatability trials. The ammonia concentration in Batch 1 was constant and negligible (mean of 0.17 mg/liter) because this batch had undergone nitrification and denitrification in the Sequencing Batch Reactors (SBR) as described in Section 5.1.2. Figure 6.4.1 also shows a notable reduction in ammonia in the influent for Batches 2 and 3 due to nitrification within the feeding tank. The opposite was found with Batch 4, which showed an increase in ammonia in the influent.

6.4.2 Nitrate/Nitrite

The influent mean had a nitrate/nitrite concentration of 14.78 mg/liter, so the influent concentration was already under the 15 mg/liter South African General Discharge Limit. The effluent had a mean concentration of 8.36 mg/liter and only on the last three days (Day 150-164) did the effluent not meet the discharge standards. The results of the nitrate/nitrite concentration are presented in Figure 6.4.2 (Raw data presented in Appendix D).

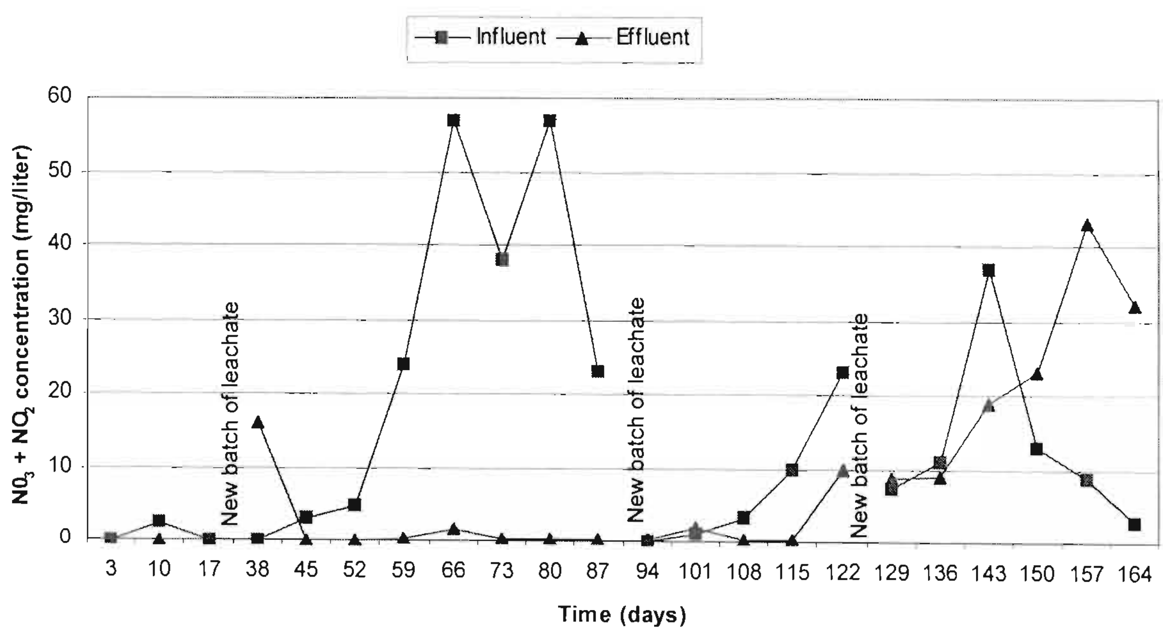

Figure 6.4.2: Nitrate and nitrite concentrations of the influent and of the effluent

It is apparent from Figure 6.4.2 that the nitrate/nitrite concentrations in the influent were variable as well. These erratic values in the influent were due to the transformation of the ammonia in the feeding tank. The changes in nitrate/nitrite concentrations in the influent are coupled with the nitrification and ammonification processes occurring within the feeding tank.

6.4.3 COD

The influent mean had a COD concentration of 390.41 mg/liter and an effluent mean of 319.44 mg/liter. At no point during the study did the effluent meet the 75 mg/liter required by the South African General Discharge Limit. The results of the COD concentration are presented in Figure 6.4.3 (Raw data presented in Appendix D).

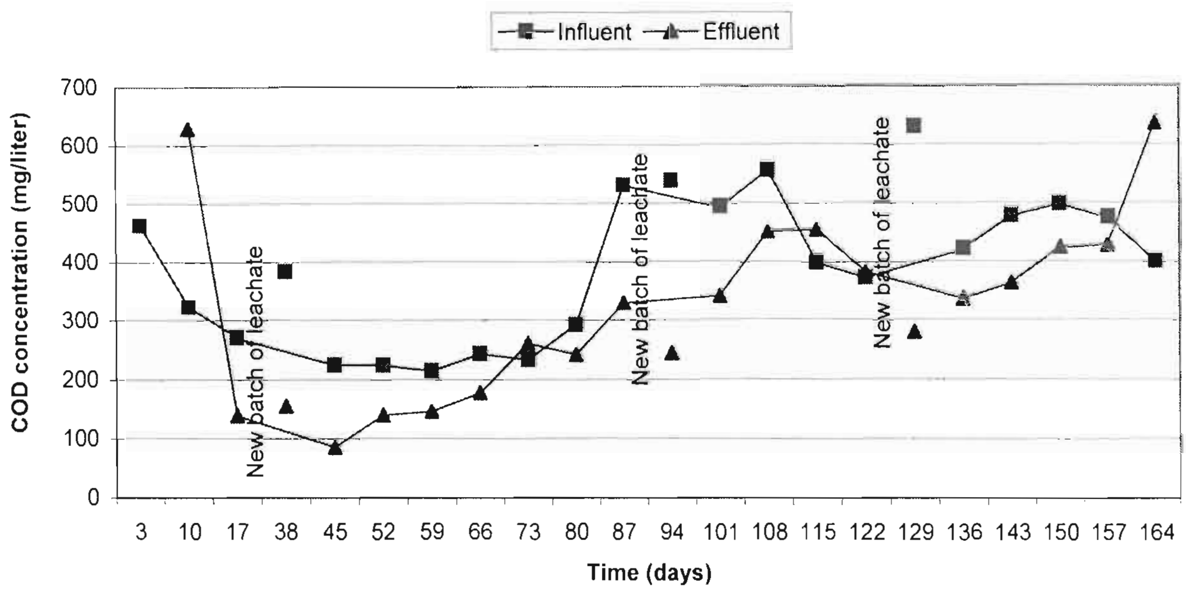

Figure 6.4.3: COD concentrations of the influent and of the effluent.

6.4.4 BOD

Analysis for Biological Oxygen Demand (BOD) began on day 45 of the treatability trail, so only Batch 2, 3, and 4 were analyzed for BOD. The influent mean of the BOD concentration was found to be 181.67 mg/liter, and the effluent mean was 139.44 mg/liter. The results of the BOD concentration are presented in Figure 6.4.4 (Raw data presented in Appendix D).

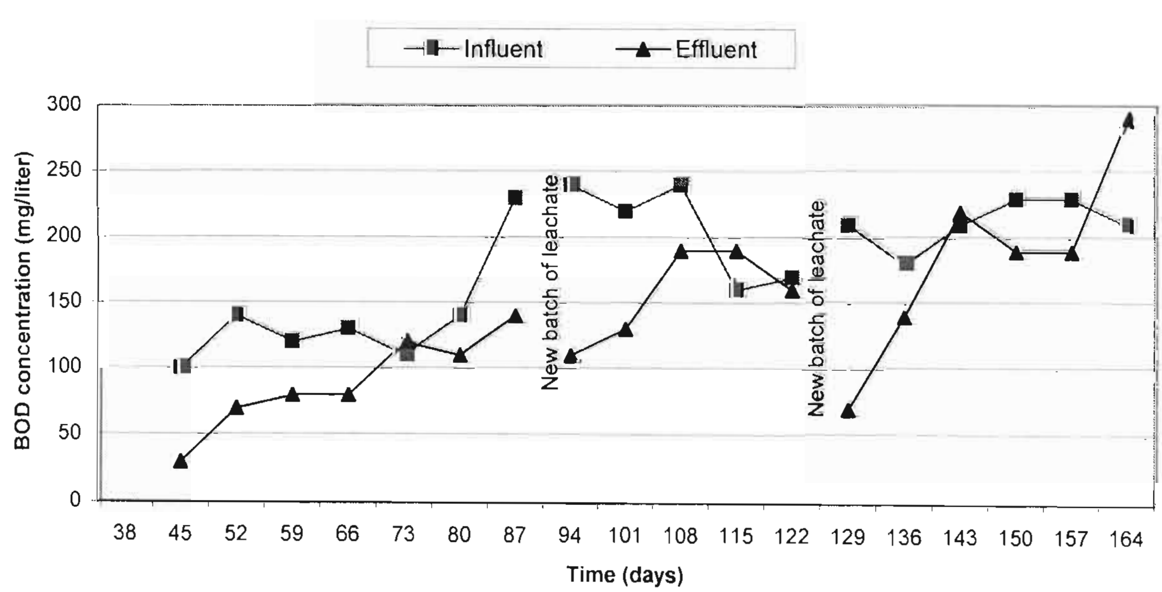

Figure 6.4.4: BOD concentrations of the influent and of the effluent.

6.5 Mass of Influent and Effluent

As mentioned in Section 6.3, the key parameters in this study were nitrogen, in the form of ammonia and nitrates/nitrites, and organics, as determined by BOD and COD. In order to analyze the results independent of the effects of evaportranspiration, a mass balance was conducted and the results for these parameters are presented in Figures 6.5.1 to 6.5.4 (Raw data presented in Appendix D). They show the changes in the mass of the influent and the effluent for each parameter. The general description of the presentation of the graphs is described in Section 6.1.

6.5.1 Ammonia

The mean mass of the ammonia in the influent was 2.59 g/day and the mass in the effluent was 0.41 g/day. The results of the mass of ammonia in the influent and the effluent are presented in Figure 6.5.1 (Raw data presented in Appendix D).

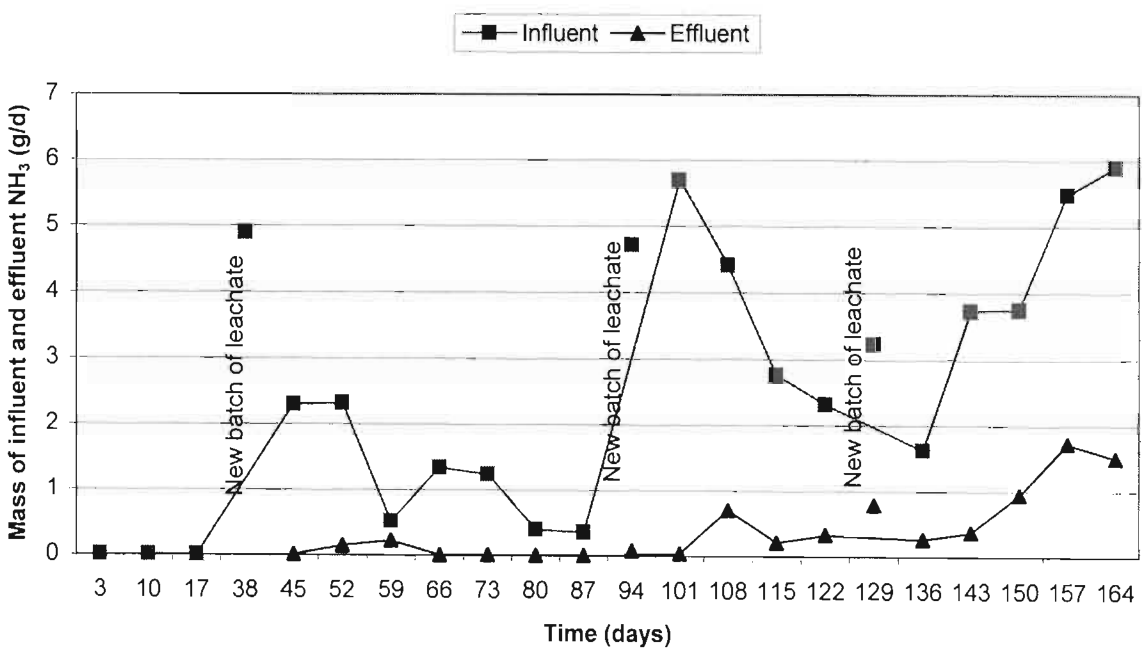

Figure 6.5.1: Influent and effluent ammonia masses

6.5.2 Nitrate and Nitrite

The mean mass of the nitrate/nitrite in the influent was 0.32 g/day and the mass in the effluent was 0.24 g/day. The results of the mass of nitrate/nitrite in the influent and the effluent are presented in Figure 6.5.2 (Raw data presented in Appendix D).

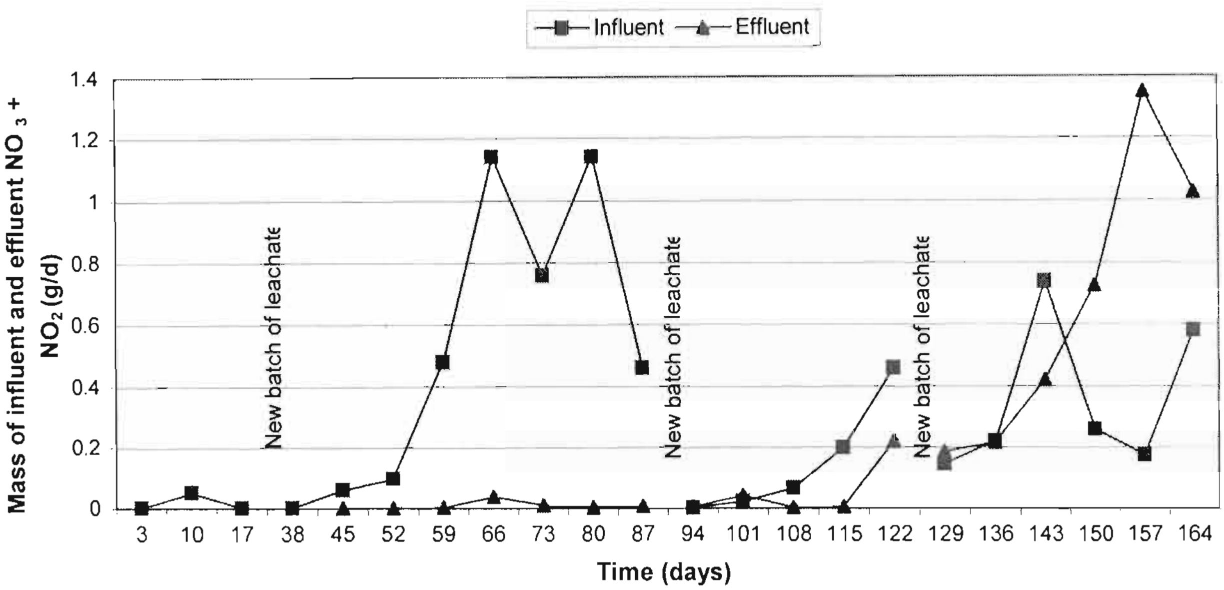

Figure 6.5.2: Influent and effluent nitrate and nitrite masses

6.5.3 COD

The mean mass of the influent COD was 7.89 g/day and the mean mass of the effluent was 8.06 g/day. The results of the mass of COD in the influent and the effluent are presented in Figure 6.5.3 (Raw data presented in Appendix D).

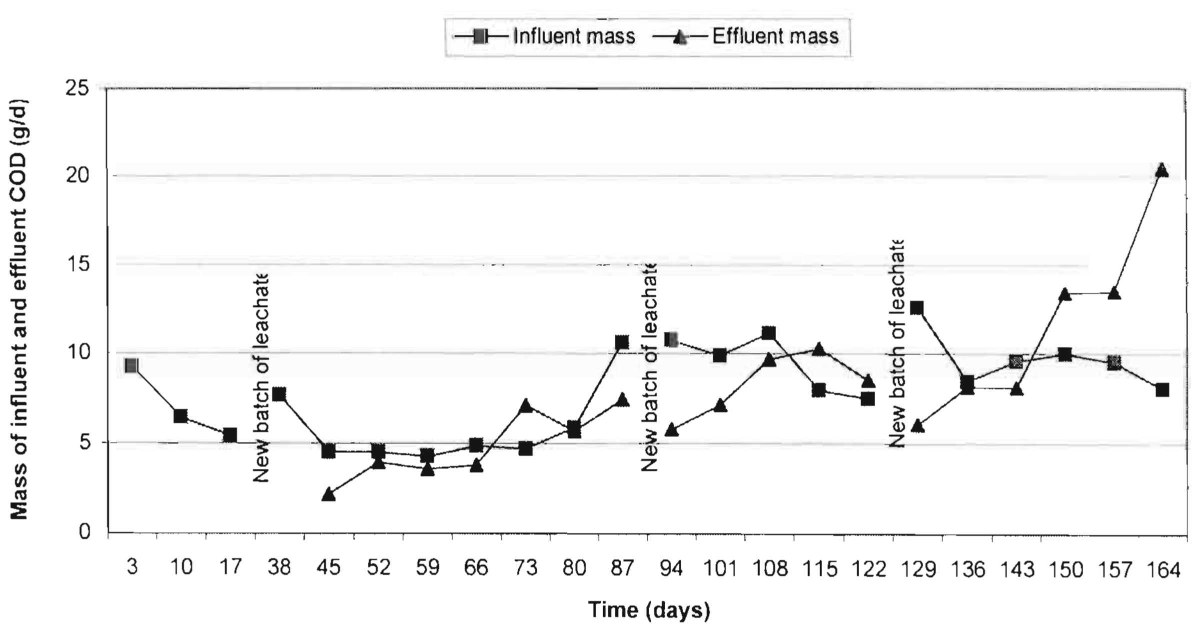

Figure 6.5.3: Influent and effluent COD masses

6.5.4 BOD

The mean mass of the influent BOD was 3.63 g/day and an effluent mean mass of 3.53 g/day. The results of the mass of BOD in the influent and the effluent are presented in Figure 6.5.4 (Raw data presented in Appendix D).

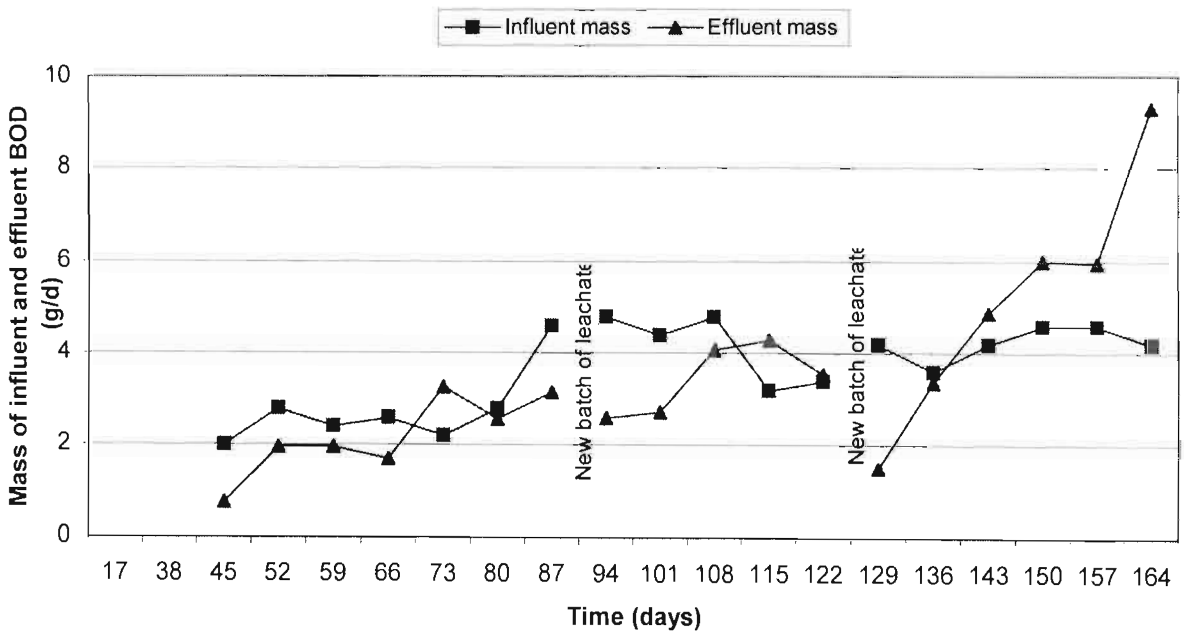

Figure 6.5.4: Influent and effluent BOD masses

6.6 Ratio of BOD to COD

The mean BOD to COD ratio for the concentration of organics in the influent was 0.47 with a standard deviation of 0.06. This ratio is characteristic of an early methanogenic leachate as expected from this section of the Bisasar Road Landfill. The ratio in the effluent was similar at 0.44 with a standard deviation of 0.07. The results of the BOD to COD ratio in the influent and the effluent are presented in Figure 6.6.1 (Raw data presented in Appendix D).

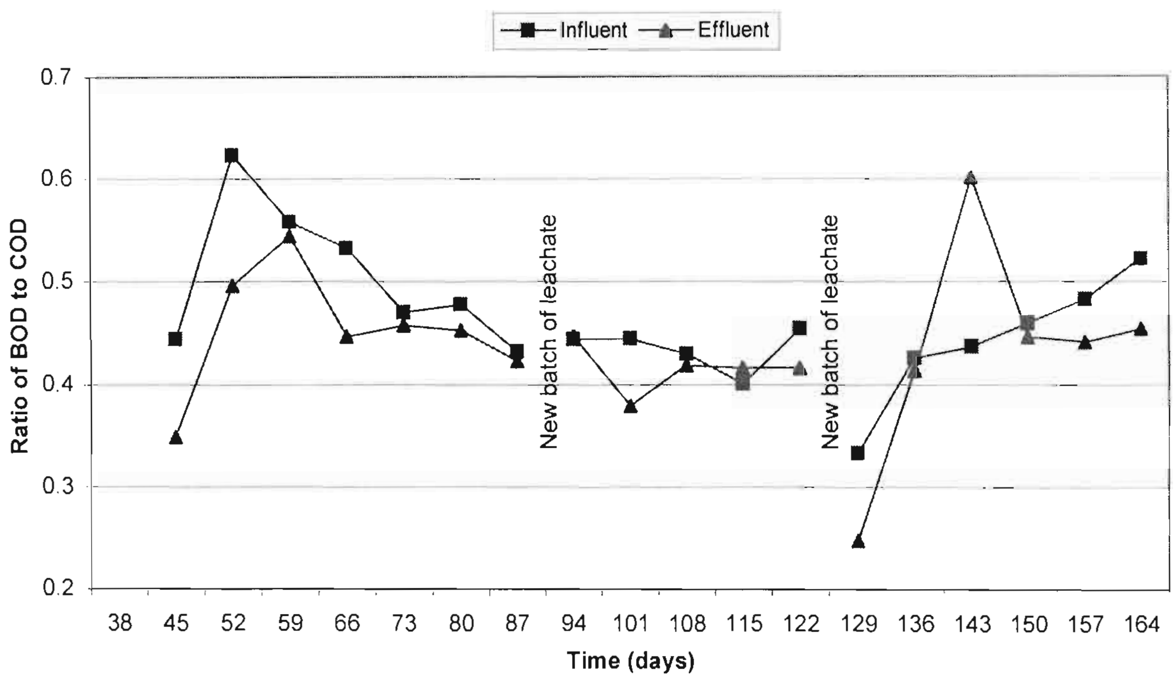

Figure 6.6.1: The ratio of concentration of BOD to the concentration of COD

As noticeable from Figure 6.6.1 both the influent and effluent BOD to COD ratios remain relatively constant throughout the treatability trails. It should be noted from Figure 6.6.1 and comparing the means mentioned previously that there no significant change between the influent and effluent ratios.

6.7 Removal Efficiencies

The wetland performance was determined by examining the reduction in pollutant concentration and reduction in pollutant mass. Reduction in concentration is important in order to meet the effluent requirements as stated in the South African general discharge limits. Although the reduction of the mass of the parameter is itself not used in the determination for regulations (since the discharge limits are given in concentration), it is even more critical for environmental protection for it is the amount of pollutant not the concentration that has the impact on the receiving environment. The two types of reduction may be described by equations 6.7.1 and 6.7.2 (Mulamoottil *et al* 1998).

$$\% \text{ concentration reduction} = 100\,(C_i - C_e)/C_i \qquad \text{(equation 6.7.1)}$$

$$\% \text{ mass reduction} = 100\,(Q_i\,C_i - Q_e\,C_e)/Q_i\,C_i \qquad \text{(equation 6.7.2)}$$

where, C_i = influent concentration, (mg/l)

C_e = effluent concentration, (mg/l)

Q_i = input water flow rate, l/d

Q_e = output water flow rate, l/d

6.7.1 Ammonia

Both the concentration and the mass removal efficiency of the VSB CW was found to be quite high. The mean concentration removal efficiency of ammonia was 91.26% with a standard deviation of 7.38.The mean mass removal efficiency of ammonia was 87.12% with a standard deviation of 12.66. The results in terms of concentration removal are presented in Figure 6.7.1 (Raw data presented in Appendix D), while the results in terms of mass removal are presented in Figure 6.7.2 (Raw data presented in Appendix D).

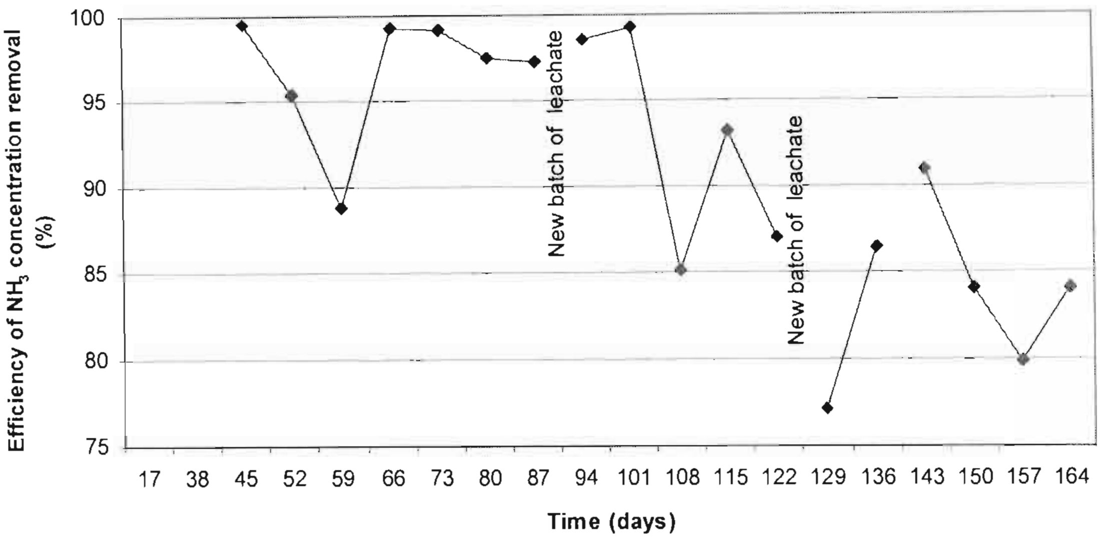

Figure 6.7.1: Percentage of ammonia concentration removal efficiency

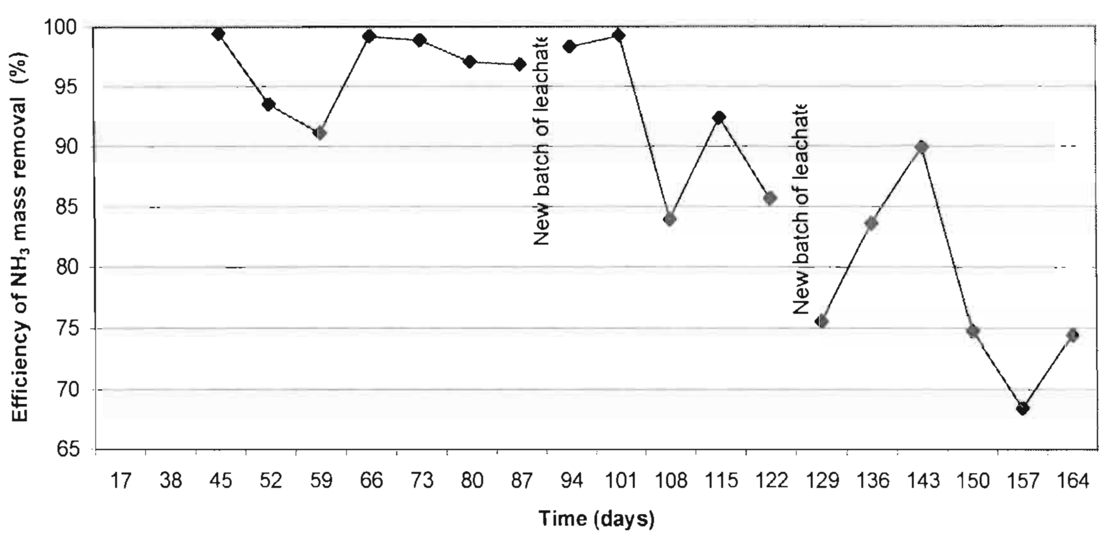

Figure 6.7.2 Percentage of ammonia mass removal efficiency

As evident in Figure 6.7.1 and Figure 6.7.2, there was an erratic trend in the ammonia removal efficiency, but overall it remained high even though there were large variations in the influent as discussed in Section 6.4.1.

6.7.2 Nitrate/Nitrite

Due to the nitrification experienced in the VSB, there was an increase in nitrate/nitrites as shown in Figure 6.4.2 and Figure 6.5.2, so there is no removal efficiency to report.

6.7.3 COD

The mean concentration removal efficiency for COD was 19.55% with a standard deviation of 28.84. The results in terms of concentration removal are presented in Figure 6.7.3 (Raw data presented in Appendix D). The mean mass removal efficiency of COD was –1.87% with a standard deviation of 49.13. The results in terms of mass removal are presented in Figure 6.7.4 (Raw data presented in Appendix D).

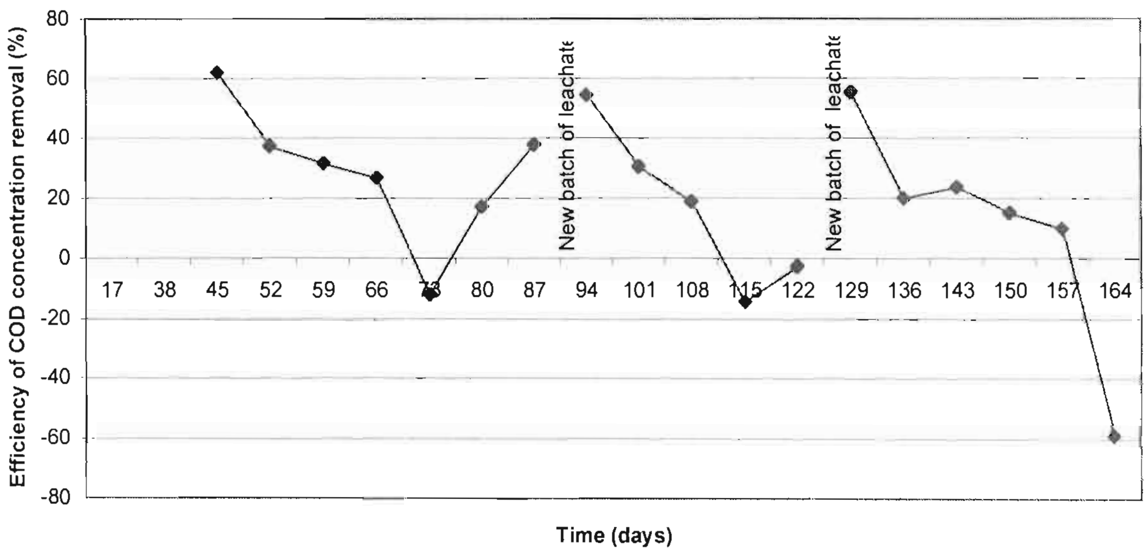

Figure 6.7.3: Percentage of COD concentration removal efficiency

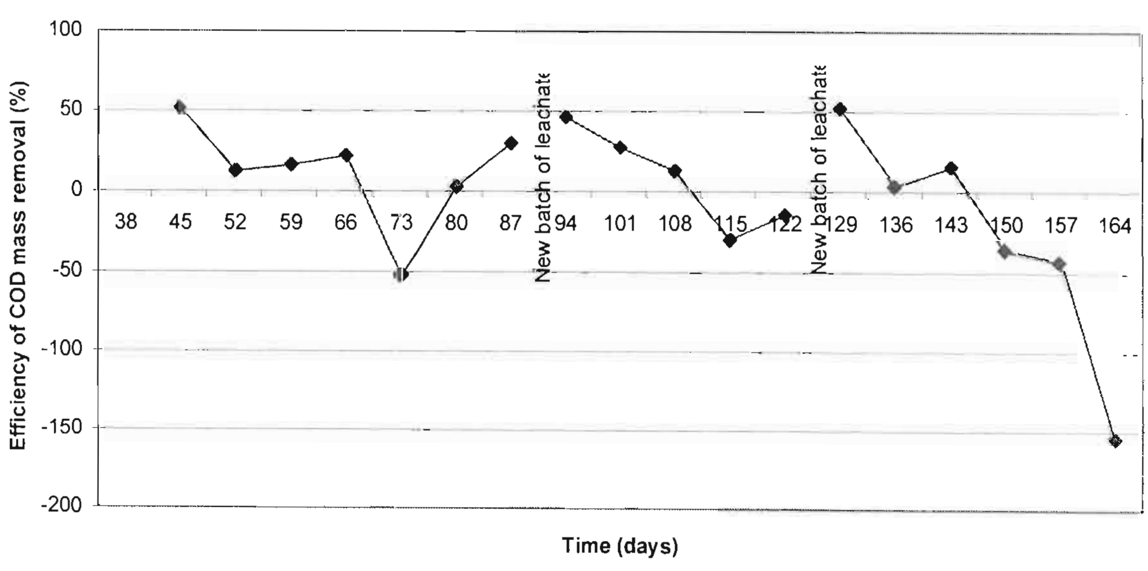

Figure 6.7.4: Percentage of COD mass removal efficiency

It is clear from the results of mass of COD in the influent and the effluent presented in Figure 6.5.3 and the removal efficiency of COD presented in Figure 6.7.3 and 6.7.4 that there is no evidence of consistent reduction of COD during the treatability trials, thus demonstrating that the organics that remain within the leachate are refractory and are unlikely to be degraded.

6.7.4 BOD

The mean concentration removal efficiency for BOD was 23.72% with a standard deviation of 29.08. The results in terms of concentration removal are presented in Figure 6.7.5 (Raw data presented in Appendix D). The mean mass removal efficiency of BOD was 23.84% with a standard deviation of 45.66. The results in terms of mass removal are presented in Figure 6.7.6 (Raw data presented in Appendix D).

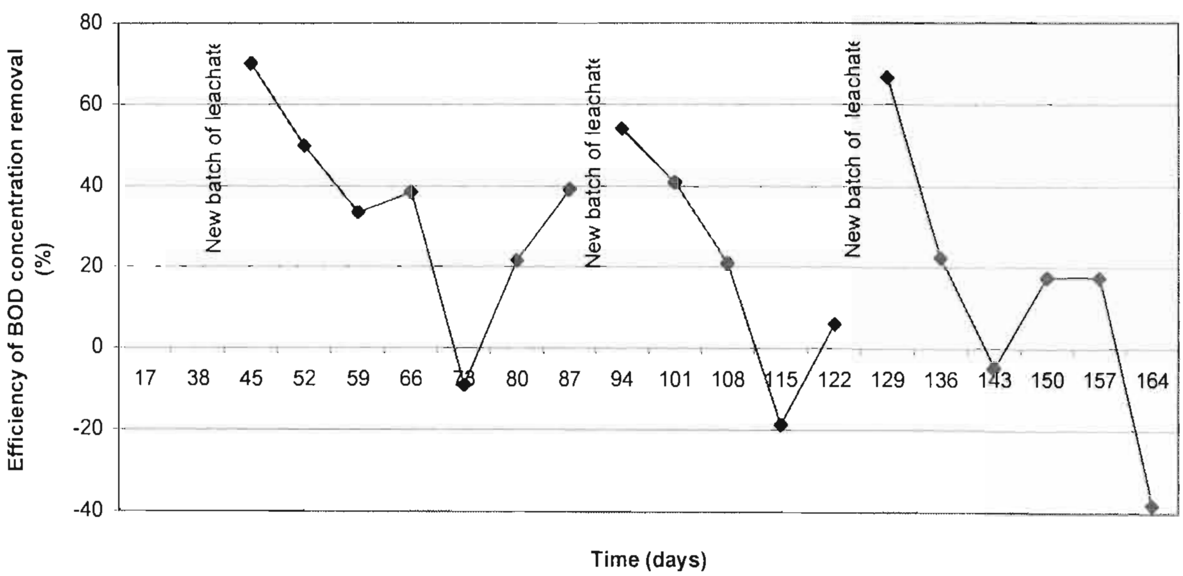

Figure 6.7.5: Percentage of BOD concentration removal efficiency

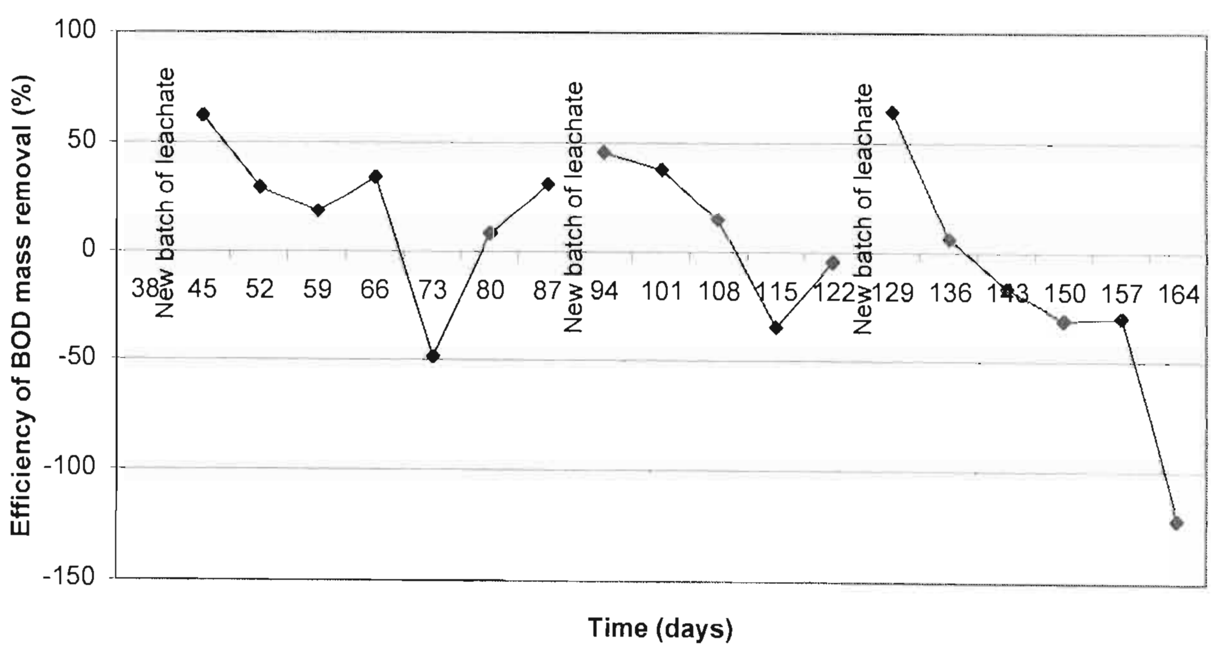

Figure 6.7.6: Percentage of BOD mass removal efficiency

As with the removal efficiency of COD (described in Section 6.7.3), the results of the removal of BOD also clearly show that the organics that remain in the leachate are mainly refractory and in general the reduction of the organic fraction will be a very slow.

6.8 Changes in Other Constituents

The results of the analysis of other constitutes in the leachate in terms of mean concentration and corresponding standard deviation are presented in Table 6.8.1 (Raw data presented in Appendix D). There are no significant changes in any of these parameters.

Table 6.8.1: Concentrations of the influent, at sampling pipe #1 and the effluent

Parameter	Influent		Pipe 1		Effluent	
	Mean	Std Dev	Mean	Std Dev	Mean	Std Dev
Alkalinity (as $CaCO_3$)	1526.23	365.35	1049.76	295.77	1085.78	225.15
Calcium	71.86	26.35	82.48	22.11	100.50	18.56
Chloride	868.59	338.50	869.81	322.62	955.22	328.98
Conductivity	646.64	109.36	485.05	143.76	557.22	141.34
Iron	0.39	0.34	0.22	0.19	0.35	0.37
Lead	0.07	0.05	0.05	0.01	0.05	0.01
Magnesium	101.00	31.53	95.14	34.99	108.72	28.66
Phosphate	3.74	4.74	1.31	1.08	0.73	0.80
pH	8.00	0.26	7.56	0.38	7.70	0.29

Note: 1. Results in mg/liter except pH (unitless) and conductivity (mS/m)

2. Analyses conducted by Durban Metro Water Services Laboratory.

6.9 Notable Qualitative Observations

There were no quantitative studies conducted on the *Phragmites australis* that was planted in the VSB, but qualitative observations suggest that the relatively high ammonia concentration did not create a toxic environment for the plants. As noted from comparing Plate 6.9.1, which was taken immediately after the rhizome sections of *Phragmites australis* were planted in early July 2000, with Plate 6.9.2, which was taken at the end of the July 2003, there was significant growth that occurred during these three years. This supports the studies found in literature which state that it usually takes two to three years for the new growths to become established (Robinson *et al* 1993; Reed *et al* 1995).

Plate 6.9.1: *Phragmites australis* after planting in July 2000 (Olufsen 2003)

Plate 6.9.2: *Phragmites australis* three years after planting in July 2003.

The receiving ponds were used to qualitatively assess the toxicity threat of the VSB effluent to the receiving environment. While no quantitative data was gathered from the receiving ponds, they were visually monitored throughout the study and showed no signs of degradation due to the effluent. This reflects what was noted in Chapter 6 that the organics remaining in the effluent are refractory and posed no biological oxygen demand to the receiving environment. Even with an ammonia concentration above the discharge limit of 2 mg/l, the plants and fish that were established in the ponds continued to grow. A visual comparison between the pond when they were initially established in 2001 and at the conclusion of the 2003 treatability trails can been seen in Plate 6.9.3 and Plate 6.9.4.

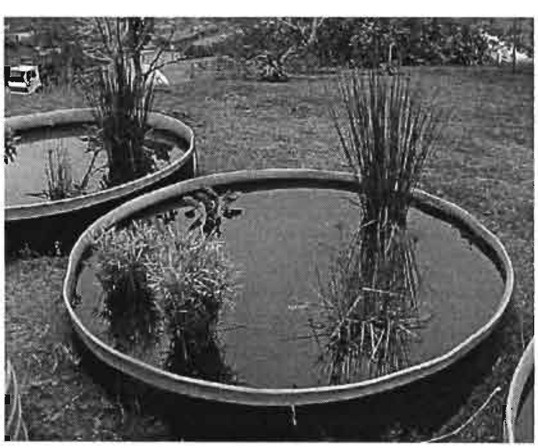

Plate 6.9.3: Newly established constructed ponds July 2000 (Olufsen 2003).

Plate 6.9.4: Constructed ponds three years after being established in July 2003.

6.10 Summary of the Results

A significant reduction in both ammonia and organics was observed although the actual results do not follow a specific trend. This is due to the time needed for the system to acclimatize and to the scale of the system. The removal efficiency may become less fluctuating once the bacteria and plants adjust to the influent and communities of appropriate microflora have been established. External factors, such as climate, may influence the system and also demonstrate the need for an acclimatization period. Comparing the climatic factors at the study site in Figure 6.2.1 and Figure 6.2.2 to the contaminants' fate within the reactor (Figures 6.4.1 – Figure 6.5.4), it is evident that there is not a noticeable effect of either the rainfall or evapotransporation on the process of removal. The large rainfall events during weeks 31 and 59 do not show a decrease in the concentration of contaminates nor does the ET seem to increase the concentrations. This lack of impact suggests that more time is needed for the CW to be established to be able to react to loadings and rainfall events.

While this study did not evaluate in depth the mechanisms involved in this reduction, there are several possibilities. The increase of nitrates in the effluent suggests that the ammonia removal in the wetland was predominately through nitrification. The rapid growth of *Phragmites australis* implies that the plants may have incorporated some of the nutrients into the plant biomass. Mass reduction may be due to the evaportranspiration impact to the system and filtration through the soil medium. The loss of moisture due to ET may cause an increase in the retention time of contaminates within the CW thus allowing the bacteria to further reduce the solutes. This is particularly true with ammonia and the COD that is in particulate form. On the basis of the age of the leachate analyzed, we can argue that the majority of the COD in the samples was in dissolved form and therefore not easily affected by filtration of other mechanical removal mechanisms.

6.11 Maintenance of the VSB

The Vegetative Submerged Bed (VSB) was fairly easy to maintain, and the few problems encountered were fairly insignificant. The following is a list of some of the issues that arose during the study:

- Clogging of the solenoid valve

The solenoid valve used to release the leachate from the header tanks would occasionally become clogged with solid contaminants in the raw leachate and with algal growth. This was noticed when the header tank began draining slowly. Once the solenoid valve was removed and cleaned, it functioned normally.

- Difficulties with timers/lack of system control

The original design of the system was to have solenoid valves connected from both the leachate and borehole water tanks to the header tank. They could then be set on timers like the one used for the release from the header tank to the VSB. This would eliminate the need for manual feeding of the tank. Unfortunately due to changes in the volume in the tanks and the lack of control over the environment in which the valves are located, this system did not function as desired. So for the majority of the study the header tank was manually filled.

- Clogging of the inlet system

The inlet system would occasionally become clogged by debris from the *Phragmites australis* and the environment. This problem was also noted by Olufsen (2003) and solved by routinely monitoring the system and manually removing all debris.

- Aphids

The *Phragmites australis* were attacked by aphids starting in March and continuing throughout the study. These parasitic insects did not seem to interfere with the health of the plants or retard the growth in anyway, so nothing was done to remove them. During the 2001 study the aphids did cause a reduction in the *Phragmites australis* and in that case an insecticide was applied (Oulfsen 2003).

- Overgrowth of the *Phragmites australis*

Despite the aphids and the high ammonia concentration in the influent, the *Phragmites australis* thrived in the VSB to such an extent that sampling within the wetland (at pipe #1) became difficult. If the treatability trails were to continue, harvesting a percentage of the plants should occur so as to reduce the chances of clogging in the system.

Chapter Seven

Conclusions of the Research

7.1 Overall Conclusions

The aim of this dissertation was to ascertain the use of constructed wetlands as an appropriate treatment option for untreated methanogenic landfill leachate by determining the efficiency of ammonia and organic removal in a pilot-scale vegetated submerged bed (VSB) constructed wetland (CW) planted with *Phragmites australis*. Theoretically, VSB were found to be an appropriate treatment option for landfill leachate due to their affordability and ease of operation, but in this study they were not found to be able to reliably meet the discharge standards. Therefore in the context of this study they could not provide a low-cost, long-term treatment option for landfill leachate at the Bisasar Road Landfill. As will be discussed in the remainder of this chapter, there are several reasons for this and possible modifications that could be made to affectively use VSB during an extended aftercare period

7.2 Study Limitations

During this study there was significant removal efficiency for both contaminates of concern, ammonia and organics, it is difficult to correlate the reduction to a specific aspect of the CW. This is because the study examined the use of a pilot-scale wetland during a relatively short time, so both the scale of the reactor and the time involved will limit the analysis of the conclusions. Also the treatability trials were conducted during the acclimatization period of the CW, which is normally the most important and most difficult to analyze. This difficulty arises as both the bacteria and the plants adjust to the change in influent to the system. While this was not a new CW, it had not be subjected to the constituents in leachate since August of 2001. Therefore the bacteria needed to reduce the contaminants may require a longer time to be reestablished in the CW. This information and that from other studies (Olufsen 2003) suggest that in order to be able to properly determine the effectiveness of a CW in treating effluent a longer study time is required. This will allow the effects of the system to be studied and not the impact of external factors.

7.3 Treatment Performance

7.3.1 Nitrogen

The treatability trials have shown that the VSB can significantly reduce the amount of ammonia in the leachate. These results support the work of other researchers who have shown reed beds to be successful in treating low-level ammonia concentrations (Robinson *et al* 1997). In this treatability trial, the constructed wetland demonstrated a mean concentration removal efficiency of ammonia of 91%. This is a high reduction in comparison to full-scale constructed wetlands that have been successfully implemented to treat methanogenic leachates. At the Monument Hill landfill site near Devizes, UK, a full-scale (1800 m^2) constructed wetland planted *Phragmites australis* has been able to show a consistent reduction of 50% of ammonia concentration in its methanogenic leachates (Robinson and Harris 2000). The difference in the reduction efficiencies may be due to the variation in original concentration levels, in time allowed for stabilization of the plants, or the length of time during the study. The ammonia reduction in the Monument Hill constructed wetland was subject to seasonal variability despite its significantly larger size (Robinson and Harris 2000), but the variability was not as extreme as caused by the scale of this pilot constructed wetland.

The mechanism for the ammonia removal in the wetland was predominately through nitrification as confirmed by the increase of nitrates/nitrites in the effluent as seen in the difference between the nitrates/nitrites in the influent and the effluent in both Figure 6.4.2 and 6.5.2. The variation in the removal efficiency for ammonia may be due to the need for an increased acclimatization period. Previous studies have also shown that the VSB require longer than eight months to acclimatize (Oulfsen 2003). As mentioned in Section 6.10, despite the aphids and the high ammonia concentration in the influent, the *Phragmites australis* thrived in the VSB.

Throughout the study there was a general reduction of ammonia in the 1000-liter storage tank as seen in the change of concentration in the influent over time (Figure 6.4.1). This was noted for Batch 2 and Batch 3, but in Batch 4 there was an increase in ammonia concentration. A change of ammonia concentration of 200 mg/l throughout the feeding is not considered significant and may be related to sampling techniques and the presence of a large enough airspace above the leachate to allow for oxygen diffusion into the sample. The nitrification process occurring during Batch 2 and 3 is confirmed by the increase of nitrate/nitrite concentration during the same period (Figure 6.4.2). For Batch 4 the slight increase in ammonia may be due to the occurrence of localized anaerobic conditions (possibly at the bottom of the tank) or due to the absence of a significant amount of oxygenated air within the tank. This increase is also seen in the comparison between the change in ammonia influent concentrations over time (Figures 6.4.1) and the nitrate/nitrite influent concentrations over time (Figure 6.4.2).

7.3.2 Organics

There were erratic fluctuations in both the treatment efficiencies for COD and BOD. The mechanism for removal was primarily through bacteria degradability and filtration. The ET may have increased the retention time of contaminates therefore possibly accelerating their biodegradability. This may also lead to a mass reduction as particulate contaminates are retained within the bed media. For the majority of the study a positive removal efficiency was recorded, but at no point during the study did the effluent meet the required discharge limit of 75 mg 0_2/liter (for COD) or the 30 mg 0_2/liter (for BOD). From the results stated in Chapter 6, it is clear that there is no evidence of constant reduction of COD during the treatability trials. This may be due to the refractory nature of the organics that characterize methanogenic leachate as suggested by a low BOD to COD ratio (Figure 6.6.1).

These results support the conclusion determined from the 2001 study, which also found that a reduction in COD could not be met using a biological treatment process (Olufsen 2003). In order for the leachate to meet the discharge requirements, other treatment methods such as ozonation, activated carbon absorption or chemical oxidation will have to be included as part of the process. As mentioned in Sections 2.6.1.2, 2.6.1.3 and 2.6.1.4 respectively, these are all expensive treatment options and require significantly more maintenance. Because of these concerns, they would not be considered an appropriate treatment option but a necessary measure to meet the current discharge standards.

7.4 Full-scale Recommendations

As discussed in Chapter 4, constructed wetlands are considered an appropriate technology if they are shown to be affordable, to be easy to operate and maintain, and to be reliable. The VSB used in this study met the first two criteria but as mentioned it was not able to reliably reduce ammonia and organics to the levels required to be able to discharge into local receiving waters. An increase in the scale of the system and a longer retention time may slightly reduce the pollutants, but this may be prohibitive in areas where land is expensive or the receiving environment is highly sensitive. Also due to the low biodegradability, the biological processes may still be shown to not be able to meet the standard. For this reason, the effluent from the VSB would impose little to no oxygen demand on the receiving waters. This was demonstrated by the health of the receiving pond used in the study. In cases like the one investigated, the Department of Water Affairs and Forestry (DWAF) should consider altering the discharge limit to allow this type of non-impacting effluent to be released while still maintaining the quality of the receiving waters.

A Free Water Surface (FWS) constructed wetland may be more appropriate treatment for methanogenic leachates. As described in Section 4.2.1, FWS wetlands allow water to flow over the bed of the wetland and through the planted vegetation, thus allowing diffusion of oxygen into the surface water. This type of system would require a larger land area, but it would be less expensive to construct, and require even less maintenance and monitoring than a VSB, since FWS potentially have simpler hydraulics because they rely on surface water flow. One of the notable drawbacks of this system is its lack of thermal protection, but in the context of Durban, South Africa, this is not a problem due to the relatively mild winters. Of concern would be maintaining the water at a sufficient depth so as to limit the use of the wetland as a mosquito breeding area. Another concern may be related to the difficulty in achieving both nitrification and denitrification within a single CW system (Tchobanoglous 1993). A combination of CW with another type of system may be required. Using a constructed wetland in combination with a sequencing batch reactor (SBR) has been shown to be able to meet the ammonia and nitrate/nitrite discharge limits (Strachan 1999; Olufsen 2003).

7.5 Future Research

Although the concept of using constructed wetlands to treat wastewater has been an established practice for over 50 years, there remains a dearth of specific information regarding the chemical transformation mechanisms in the CW. In order to gain more of an understanding of constructed wetlands, long-term monitoring of CW is critical. Monitoring during each aspect of the project phase that is ongoing through the use of the CW is needed. Such information can be used to enhance the process efficiencies and aid in defining the transformation and movement of specific constituents in the wetland.

In regard to the desire for sustainable landfills, research to improve waste management practices at the source abound. There are needs for designing systems of operations that produce little or no waste by managing waste practices, increasing the rate of recovery and reuse of waste materials, and improving standards of landfill design, operation, and aftercare (as recommended by Hawken 1993, Robinson 1995b, and Röhrs *et al* 2001). Concern has also been raised in regard to the final destination of the constructed wetland (Vasel 2002). There should be evaluations now on how the are will be classified at the end of the aftercare period; if it is classified as a contaminated soil, that may be a significant drawback to using this type of land based system.

Future research examining appropriate aftercare treatment for landfill leachate should examine other types of passive treatment options in addition to CW, such as stabilization ponds, which have been shown to successfully remove BOD (Reed *et al* 1995) and ammonia (Reed 1984). The concern with this type of system would be the ammonia concentration, which even at low levels, may create a toxic environment for aquatic life. If the ammonia concentration is substantially high a type of pretreatment or preliminary pond unit would be needed. Another option could be the use of aerated lagoons, which have been shown to have more success in the treatment of wastewater than constructed wetlands (Maehlum and Haarstand 2001). Studies should also include non-treatment benefits from constructed wetlands such as their recreational, educational and habitat values.

REFERENCES

Ahnert L. and Ehrig H-J. 1992. Co-Treatment of Leachate with Sewage. *In*: T.H. Christensen, R. Cossu and R. Stegmann (eds), *Landfilling of Waste: Leachate*. Elsevier Science, Barking, Essex, UK. pp. 403-416.

Andreottola G., Cannas P. and Cossu R. 1990. Overview of Landfill Leachate Quality. Technical Note No. 3. CISA, Environmental Sanitary Engineering Center, Cagliari, Italy.

Andreottola G. and Cannas P. 1992. Chemical and Biological Characteristics of Landfill Leachate. *In*: T.H. Christensen, R. Cossu and R. Stegmann (eds) *Landfilling of Waste: Leachate*. Elsevier Science, Barking, Essex, UK. pp. 65-88.

Armstrong W. 1990. Measurements and Modeling of Oxygen Release from Roots of Frogmouths australis. Proceedings from the International Conference for Wetlands in Water Pollution Control. Cambridge, England. Sept 24-28.

Ball J.M. 1984. Water Pollution Control in Solid Waste Disposal. Proceedings of the Municipal Engineers' Conference, Cape Town.

Ball J.M. 2002. Milestones Toward Minimum Requirements. Proceedings from International Waste Management Biennial Congress and Exhibition. WasteCon2002. Vol.1 pp 20-28.

Ball J.M. and Blight, G.E. 1988. The Fate of Leachate from a Landfill Site. Proceedings of International Waste Management (SA)Biennial Congress, Durban, South Africa.

Barber C and Maris J.J 1992. Leachate Recirculation: Full-Scale Experience. Landfilling of Waste: Leachate. *In*: T.H. Christensen, R. Cossu and R. Stegmann (eds), *Landfilling of Waste: Leachate*. Elsevier Science, Barking, Essex, UK. pp 381-401.

Blakey, N. 1992. *Model Prediction of Landfill Leachate Production* Landfilling of Waste: Leachate. *In*: T.H. Christensen, R. Cossu and R. Stegmann (eds), *Landfilling of Waste: Leachate*. Elsevier Science, Barking, Essex, UK. pp. 17-34.

Blakey N. 1997. Leachate Recirculation and Cost-Effective Treatment. Points raised in the discussion in Workshop W15, Sardinia 1997. Sixth International Landfill Symposium, Workshop Report by H. Robinson, pp1-6.

Blakely N.C. and Maris P. 1990. Methane Recovery from the Anaerobic Digestion of Landfill Leachate. Report ETSU B 1223, prepared by Water Research Commission (WRC) for the Department of Energy's Renewable Energy Research and Development Programme, managed by the Energy Technology Support Unit (ETSU).

Blight G.E., Hojem D.J., and Ball J.M. 1992. Production of Landfill Leachate in Water-Deficient Areas. Proceedings from WasteCon2002: the International Waste Management Biennial Congress and Exhibition. Vol.1 pp 29-39

Blight, G.E. and Novella, P.H. 2000. A Re-appraisal of the Minimum Requirements for Leakage Outflow from Landfill Liners. Proceedings from the International Training Seminar on Control, Management and Treatment of Landfill Emissions. Unv of Natal, Durban, SA. 6-8 December 2000.

Bosman C. 2002. New Legal Implications on South African Local Authorities' Waste Management Responsibilities. Proceedings from WasteCon2002: the International Waste Management Biennial Congress and Exhibition. Vol1 pg. 230-242.

Boswell J.E.S. and Bell S.E. 2000. Practical Methods to Address the Biochemical Clogging of Drainage Systems in Landfills. Proceedings from WasteCon2000: the International Waste Management Biennial Congress and Exhibition. Vol I pp159-168.

Bowers A. 1999. Preliminary Investigation into MSW Landfill Leachate and Biogas Emissions Under a Sub-tropical Climate. Undergraduate Dissertation, Department of Civil Engineering, Construction and Surveying University of Natal Durban, South Africa.

Boyle W.C. and Ham R.K. 1974. Treatability of Leachate from Sanitary Landfills. *Journal Water Pollution Control Federation*, 46(5): 860-872.

Bredenhann L. and Ball J. 2002. Third Edition of Minimum Requirements and DOC.4: Shaping the Future of Waste Management. Proceedings from WasteCon 2002:the International Waste Management Biennial Congress and Exhibition. Vol1 pg. 243.

Brix H. 1987. Treatment of Wastewater in the Rhizosphere of Wetland Plants – The Root Zone Method. *Water Science and Technology*. Vol 1 No.10. pp 107-118.

Brix H. 1992. Functions of Macrophytes in Constructed Wetlands. Proceedings from the International Conference for Wetlands in Water Pollution Control. H J Bavor Pergamon Press, London.

Brix H. 1993. Wastewater treatment in Constructed Wetlands: system design, removal processes and treatment performance. *In:* G.A. Moshir (ed), *Constructed Wetlands for Water Quality Improvement.* CRC Press Inc. Bota Raton, Florida. pp 9-22.

Brix H. 1994. Functions of Macrophytes in Constructed Wetlands. Water Science and Technology, Vol.29, No.4, pp 1-6.

Brix H. 1997. Do macrophtes play a role in constructed wetlands? *Waste Science and Technology*. Vol 35. N 5. pp 11-19.

Brune M., Ramke H.G., Collins J.J., Hanert H.H. 1993. Incrustation Processes in Drainage Systems of Sanitary Landfills. Proceedings from Sardinia 1991: the third International Landfill Symposium. CISA, Cagliari, Vol II, pp 999-1035.

Burton S.A.Q. and Watson-Craik I.A.1998. Ammonia and Nitrogen Fluxes in Landfill Sites:

Applicability to Sustainable Landfilling. *Waste Management and Research*. Vol.16 No.1. pp.41-53.

Carter V., Bedinger M.S., Novitzki R.P. and Wilen W.O. 1978. Water Resources and Wetlands. *In:* P.E Greeson, J.R. Clark and J.E. Clark (eds), *Wetland Functions and Values: the State of Our Understanding*. American Water Resources Association, Minneapolis, Minnesota.

Canziani R. and Cossu R. 1989. Landfill Hydrology and Leachate Production. *In*: T.H. Christensen, R. Cossu and R. Stegmann (eds), *Sanitary Landfilling: Process, Technology and Environmental Impact. pp 185-212.*

Chen K.Y and B.A. Eichenberger. 1981. Evaluation of Costs for Disposal of Solid Wastes from High-BTU Gasification. Energy and Environmental Systems Division. Argonne National Laboratory, Argonne, Illinois, pp.127.

Christensen T.H., Cossu R. and Stegmann R. 1992. Landfill Leachate: An Introduction. *In*: T.H. Christensen, R. Cossu and R. Stegmann (eds), *Landfilling of Waste: Leachate*. Elsevier Science, Barking, Essex, UK pp. 3-14.

Christensen T.H., Cossu R. and Stegmann R. 1992. *Landfilling of Waste: Leachate*. Elsevier Applied Science London and New York.

Christensen T.H., Cossu R. and Stegmann R. 1998. Problems and Strategies in Leachate Management. International Training Seminar – Management and Treatment of MSW Landfill Leachate, CISA, Venice, Italy, pp I.1-I.8.

Christensen T.H and Cossu R. (1998). Basic Processes in Landfills. International Training Seminar for the Management and Treatment of MSW Landfill Leachate, CISA, pp.II-1 to II-20.

Christensen T.H. and Kjeldsen P. 1989. Basic Biochemical Processes in Landfills. *In*: T.H. Christensen, R. Cossu and R. Stegmann (eds), *Landfilling of Waste: Leachate*. Elsevier Science, Barking, Essex, UK.

Chian E.S.K. and DeWalle F.B. 1997. Evaluation of Leachate Treatment, Volume II, Biological and Physical-Chemical Processes. EPA-600/2-77-186b, US EPA, Cincinnati, Ohio.

Cleseri L.S., Greenberg A.E. and Trussel R.R. 1989. *Standard Methods for the Examination of Water and Wastewater.* American Public Health Association, 17th edition, Washington DC. USA.

Cole T. 2001. Educating the Public about Watershed Management. Presented at Water Environment Federation's 74th Annual Conference and Exposition, Atlanta, GA October 13-17, 2001.

Collivignarelli C., Avezzu F. and Bertanza G. 1998. Wet Oxidation and Other Oxidation

Processes Leachate Treatment. International Training Seminar – Management and Treatment of MSW Landfill Leachate, CISA, Venice, Italy pp.xxi-1 to xxi-18.

Couth B. 2000. Landfill Hydraulics. In International Training Seminar on Control, Management and Treatment of Landfill Emissions. Unv. of Natal, Durban, S. Africa. 6-8 Dec 2000.

Cossu R., Fois G., Haarstad K., Lavagnolo M.C. and Littarru R. 1997. Constructed Wetlands for Landfill Leachate Treatment- A Comparison of Four Species. Proceedings from Sardinia 1997: the Sixth International Waste management and Landfill Symposium. Cagliari, Vol II, pp.363-376.

Cossu R, Raga R and Rossetti D. 2001. Experimental Reduction of Landfill Emissions Based on Different Concepts. The PAF Model. Proceedings from Sardinia 2001: the Eighth International Waste management and Landfill Symposium. Vol 1 pp 220-230

Crawford J. F. and Smith P. G. 1985. *Landfill Technology.* Butterworths Publishers. London, England.

Durban Solid Waste (DSW), Durban Metro Water Services. (1997/8). Sampling and Analyses of Leachates, and Associated Ground and Surface Waters. Several reports submitted by Lombard and Associates: Environmental and Waste Management Consultants, South Africa.

Eden R. 2001. Options for Landfill Leachate Treatment in Hong Kong. Proceedings from Sardinia 2001: the Eighth International Waste management and Landfill Symposium. Vol 1. pp. 311-318.

Ehrig H-J. 1984. Laboratory Scale Tests for Anaerobic Degradation of Municipal Solid Waste. Proceedings from the International Solid Wastes and Cleansing Association Congress, Philadelphia, Sept 15-20.

Ehrig H-J. 1987. Leachate Treatment: Physico-Chemical Processes. Procedings from Sardinia 1987: the First International Landfill Symposium, Cagliari, Vol 1.

Ehrig H-J. 1989. Leachate Quality. *In*: T.H. Christensen, R. Cossu and R. Stegmann (eds), *Sanitary Landfilling: Process, Technology and Environmental Impact.* Academic Press, London, pp.213-229.

Ehrig H-J and Stegmann R. 1992. Biological Processes. *In*: T.H. Christensen, R. Cossu and R. Stegmann (eds), *Landfilling of Waste: Leachate.* Elsevier Science, Barking, Essex, UK. pp. 185-202.

Emcon Associates. 1974. Sonoma Country Solid Waste Stabilization Study. EPA-SW 530 65D, Washington, D.C: U.S EPA.

Feuillade G. and Lagier T. 2001. Treatment of Municipal Solid Waste Leachate by Electrocoagulation. Proceedings from Sardinia 2001: the Eighth International Waste Management and Landfill Symposium. Vol 1. pp 329-336.

Farquhar G.J. and Rovers F.A. 1993. Gas Production During Refuse Decomposition. Air, Water and Soil Pollution, 2,2 pp. 483-495.

Farquhar G.J. 1989. Overview of Landfill Liners Using Natural Media. Proceedings from Sardina 1989: the Second International Landfill Symposium. CISA, Cagliari, paper IX.

Fourie A. and Glight G.E. 1995. Research works on the Effects of Evaporation on Landfilled Waste bodies. Personal interview with Prof. A. Fourie. From Strachan 1999.

Fuller W.H. 1978. Investigation of Landfill Leachate Pollutant Attenuation by Soils. EPA-600/2-78-158, Cincinnati, Ohio: US EPA.

Gibbs D.C, Longhurst J., and Braithwaite C. 1998. Struggling with Sustainability: Weak and Strong Interpretations for Sustainable Development within Local Authority Policy. *Environment and Planning,* Vol 30 p. 1351-1365.

Griffith M. and Trois C. 2002. A Preliminary Investigation into the Hydraulic Properties of a Waste Body. Proceedings from WasteCon2002: the International Waste Management Biennial Congress and Exhibition Durban, South Africa. 3 September – 4 October. Vol 1. pp 116-125.

Haandel A.C. van and Marais G.v.R. 1981. Nitrification and Denitrification Kinetics in the Activated Sludge Process. University of Cape Town. Research Report No.W.39: for the Water Research Commission of South Africa.

Ham R.K. 1988. Landfill Gas, Leachate and Related Problems. Paper presented to the South African Biennial Congress. Ninth Biennial Congress and Equipment Exhibition, Durban, South Africa, 9-11 August 1988.

Hammer, M.J. and Hammer, M.J.Jr. 2001. *Water and Wastewater Technology.* Prentice-Hall Inc., Englewood Cliffs, New Jersey, USA.

Hajer M.A. 1995. The Politics of Environmental Discourse. *Ecological Modernisation and the Policy Process,* Oxford University Press, Oxford, Chapter 1.

Hawken, P. 1993. The Ecology of Commerce: A Declaration of Sustainability. HarperBusiness. New York, New York, USA.

Heyer K-U, Stegmann R. and Ehrig H-J. 1998. Leachate Treatment: Principles and Options. Proceedings from the International Training Seminar on Management and Treatment of MSW Landfill Leachate, CISA, Venice, Italy. pp x-1 to x-18.

Hill T., Walsh K., Shaw P., Harris J., Moffett B. 2001. Quantification of N Transforming Bacteria in Surface Flow Wetland Systems by Real PCR. Proceedings from Sardina 2001: the Eighth International Landfill Symposium. Vol II, pp 293-302

Howard A.G. 1998. *Aquatic Environmental Chemistry.* Oxford University Press. 1998.

IWA (Specialist group on the use of macrophytes). 2000. Constructed wetlands for pollution control – Scientific and technical report, Number 8. IWA-publishing.

Johnston C.A.1993. Mechanisms of Wetland-Water Quality Interaction. *In:* G.A. Moshiri (ed), *Constructed Wetlands for Water Quality Improvement.* Lewis Publishers. pp 293-299.

Kadlec *R*.H. 1999. Chemical, Physical and Biological Cycles in Treatment Wetlands. *Water Science and Technology.* Vol 40, No.3, pp 37-44.

Kadlec R.H., Bastiaens W. and Urban D.T. 1993. Hydrological Design of Free Water Surface Treatment Wetlands. *In:* G.A. Moshiri (ed), *Constructed Wetlands for Water Quality Improvement.* Lewis Publishers.

Kadlec H.R. and Knight L.R. 1996. *Treatment Wetlands.* Lewis Publishers.

Keenan J.D., Steiner R.L, and Fungaroli A.A. 1983. Chemical-Physical Leachate Treatment. *Journal of Environmental Engineering Division*, ASCE. Vol. 109. No. 6. pp 1371.

Keenan J.D., Steiner R.L, and Fungaroli A.A. 1984. Landfill Leachate Treatment. *Journal Water Pollution Control Federation.* Vol 56. No 1. pp 27-33.

Koerner R.M and Soong T.Y. 2000. Stability Assessment of Ten Large Landfill Failures. *In:* J.G. Zornberg and B.R. Christopher (eds), GeoDenver 2000: Conference on Advances in Transportation and Geoenvironmental Systems Using Geosynthetics. GeoInstitue of ASCE, Reston, Virginia, USA.

Knight R.L. 1992. Wetlands for Wastewater Treatment Database. Proceedings from the International Conference of Wetlands in Water Pollution Control Sydney, Australia. 30 November - 3 December.

Knox K. 1992. Rotating Biological Contactors. *In*: T.H. Christensen, R. Cossu and R. Stegmann (eds), *Landfilling of Waste: Leachate.* Elsevier Science, Barking, Essex, UK pp. 211-228..

Knox K. 2000. MSc in Containment Land Management – Landfill Module: 6th to 10th November 2000 – Lecture 3a – Leachate Production and Control. In International Training Seminar on Control, Management and Treatment of Landfill Emissions. Unv. of Natal, Durban, S.Africa. 6-8 Dec 2000.

Lehman R. and Rodgers J.H. Jr. 2000. It's Not That Easy Being Green. Water Environment and Technology, Vol. 12, No. 3, pp.54-59.

Lekven C.C., Crites R.W., and Beggs R.A. 1993. Subsurface Flow Wetlands at Mesquite, Nevada. *In:* G.A. Moshiri (ed), *Constructed Wetlands for Water Quality Improvement.* CRC Press Inc. Bota Raton, Florida. pp 261-267.

Lee G.F. and Jones-Lee A. 1993. Landfills and Groundwater Pollution Issues: "Dry Tomb" vs. Wet-Cell Landfills. Proceedings from Sardina 1993: the Fourth International Landfill Symposium. October 1993, Vol 2 pp1787-1798.

Lu J.S.C., Eichenberger B. and Stearns R.L. 1984. Production and Management of Leachate from Municipals Landfills: Summary and Assessment. EPQ-600/2-84-092, Cincinnati, Ohio: US EPA Municipal Environmental Laboratory.

Maehlum T. and Haarstad K. 2001. Treatment Wetlands for Landfill Leachates – case studies from cold temperate climate. Proceedings from Sardinia 2001: the Eighth International Waste management and Landfill Symposium. Vol 1. pp 337-334.

Mathlener. R.A. 2001. The Savings of Sustainable Landfilling. Proceedings from Sardinia 2001: the Eight International Waste management and Landfill Symposium. Vol 2. pp 265-269.

Mitsch W.J. and Grosselink J.G. 1993. *Wetlands*. Second edition. Van Nostrand Reinhold. New York. USA

Mulamootti G., McBean E.A., and Rovers F. 1998. *Constructed Wetlands for the Treatment of Landfill Leachate*. Lewis Publishers.

Nathanson, J.A. 2000. Basic Environmental Technology. Third edition, Prentice Hall.

Novella P.H, Robinson H.D., Ekama G.A. 1998. Landfill Leachate Treatment in South Africa: The Aerobic Option. Publication of the Landfill Interest Group, IWM (SA), 1C-6.

Olufsen J.S., Trois C., Stretch D. and Strachan L.J. 2000. Investigation of Constructed Wetlands for Treating Landfill Leachates. Proceedings from WasteCon2000: the Biennial Conference and Exhibition. September 2000. Vol 1 pp 59-68.

Olufsen J. S. 2003. An investigation into the Technical Feasibility of Using Vegetated Submerged Bed Constructed Wetlands for the Treatment of Landfill Leachate. Masters Dissertation, Department of Civil Engineering, Construction and Surveying University of Natal Durban, South Africa.

O'Riordan T, Preston-White R, Hamann R. and Manqele M. 2000. The Transition to Sustainability: A South African Perspective. *South African Geographical Journal*, in press.

O'Riordan T. 1993. The Politics of Sustainability. *In:* R.K. Turner (ed), *Sustainable Environmental Economics and Management, Principles and Practice*. John Wiley and Sons, Chichester.

O'Riordan T. and Jordan A. 1995. The Precautionary Principle in Contemporary Environmental Politics. *Environmental Values*. Vol 4. pp 191-212.

Peavy H.S., Rowe D.R., and Tchobanoglous G. 1985. *Environmental Engineering*. McGraw-Hill International Editions.

Peggs I.D. 1994. Testing Program to Assure the Durability of Geomembrances. *In*: T.H. Christensen, R. Cossu and R. Stegmann (eds), *Landfilling of Waste: Barriers*. Elsevier Science, Barking, Essex, UK. pp 413-429.

Peters T. 1999. Past end future of membrane filtration for the purification of landfill leachate. Proceedings from Sardinia 1999: the Seventh International Waste Management and Landfill Symposium. Vol 2. pp 335-344.

Pietrelli L., Buday J., and Ciafani S. 2001. Leachate Treatment: Processes Comparison.

Proceedings from Sardinia 2001: the Eight International Waste management and Landfill Symposium. Vol 2. pp 343-350.

Pohland F.G. 1975. Sanitary Landfill Stabilization with Leachate Recycle and Residual Treatment. EPA 600/2-75-043, U.S. EPA, Cincinnati, Ohio.

Pohland F.G., Stratakis, M., Cross W.H., and Tyahla S.F. 1990. Controlled Landfill Management of Municipal Solid and Hazardous Wastes. Proceedings from the 1990 Water Pollution Control Federation (WPCF) National Specialty Conference on Water Quality Management of Landfills, Chicago, Illinois, July 15-18, p.3-16.

Powrie W. 2003. Sustainable Waste Management. http://www.spiked-online.com/articles/00000006DC1A.htm. Dated 29 January 2003. Accessed on 8 July 2003.

Pulles W., Howie D. Otto D. and Easton J. 1996. A Manual on Mine Water Treatment and Management Practices in South Africa. Appendix Vol 1. Literature Reviews. WRC Report No. 527/1/96.

Qasim S.R. and J.C. Burchinal. 1970a. Leaching from Simulated Landfills. *Journal of the Water Pollution Control Federation.* Vol 42. No 3. pp 371-379.

Qasim S.R. and J.C. Burchinal. 1970b. Leaching of Pollutants From Refuse Beds. *Journal of the Sanitary Engineering Division*, ASCE. Vov 96 No SA1.pp 49-58.

Qasim S.R and Chiang W. 1994. *Sanitary Landfill Leachate, Generation, Control and Treatment.* Western Hemisphere. Technomic Publishing Company, Inc.

Reed S.C. 1984. Nitrogen Removal in Wastewater Ponds. CRREL Report 94-13. Cold Regions Research and Engineering Laboratory (CRREL), Hanover, NH. USA

Reed S.C. and Hines M. 1993. Constructed Wetlands for Industrial Wastewaters. Proceedings from the 1993 Purdue Industrial Waste Conference, Lewis Publishers, Chelsea, MI, 1994.

Reed S.C., Crites W.R. and Middlebrooks E.J. 1995. *Natural Systems for Waste Management and Treatment.* Second edition. McGraw-Hill. USA.

Richardson C.J. 1985. Mechanisms Controlling Phosphorous Retention Capacity in Freshwater Wetlands. *Science.* Vol 228. No. 1424. pp 7.

Robinson H. D. 1987. Unsaturated Zone Aspects of Landfill. Proceedings from Sardinia 1987: the First International Waste Management and Landfill Symposium.

Robinson, H.D. 1989. The Development of Methanogenic Conditions within Landfilled Wastes, and the Effects on Leachate Quality. Proceedings from Sardinia 1989: the Second International Waste management and Landfill Symposium, Second Int. Landfill Symposium, Paper XXIX, Porto Conte, Italy, Oct. 9-13 1989, pp. 9.

Robinson H.D. 1993. The Treatment of Landfill Leachates Using Reed Bed Systems.

Proceedings from Sardinia 1993: the Fourth International Waste management and Landfill Symposium. Vol 2. pp 907-922.

Robinson H.D. 1995a. A Review of the Composition of Leachates from Domestic Wastes in Landfill Sites. Report prepared for the Department of the Environment (UK), Ref: DEO918A/FR1, Aspinwall & Company. August 1995. Shrewsbury, UK.

Robinson H.D. 1995b. The UK: Leading the Way in Leachate Treatment. Proceedings from the International Waste Management (IWM) Conference 1995. UK.

Robinson, H.D. 1996. Sustainable Waste Management: Is There a Future for Landfills? Proceedings from WasteCon1996: the Biennial Conference and Exhibition, pp. 18-37.

Robinson H.D. 1999. State-of-the-art landfill leachate treatment systems in the UK and Ireland. Proceedings for the 101st International Waste Management Annual Conference & Exhibition, Torbay, England, 8-11 June 1999.

Robinson H.D. 2001. Managing Dissolved Methane in Leachates. *Monthly Institute of Waste Management Journal*, March 2001, pp. 36-37.

Robinson H.D., Barr M.J. and Last S.D. 1992. Leachate Collection, Treatment and Disposal. *Journal of the Institute of Water and Environmental Management*. Vol 6. Nov 3. pp 321-332.

Robinson H., Barr M., Formby B., and Morag A. 1993. Using Reed Bed Systems to Treat Landfill Leachate. Presented IWEM Central Southern Branch Annual Training Day. Planning and Engineering for Sustainable Landfill Management, 14 October 1992. In Waste Management Proceedings April 1993.

Robinson H.D. and Gronow J.R. 1995. A Review of Landfill Leachate Composition in the UK. International Waste Management Proceedings. January 1995 pp 3-8.

Robinson H.D., Last S.D., Raybould A., Savory D. and Walsh T.C. 1997. State of the Art Landfill Leachate Treatment Schemes in the United Kingdom. Proceedings from Sardinia 1997: the Sixth International Waste management and Landfill Symposium. Vol II pp 269-276.

Robinson H.D., Harris G.R., Carvill M.S. and Formby R.W. 1997. Engineered Reed Bed Treatment of Old Leachates. Proceedings Sardinia 1997: the Sixth International Waste Management and Landfill Symposium, Cagliari, Italy.

Robinson H.D. and Gronow J.R. 1995. Leachate Composition from Different Landfills. Proceedings from the International Training Seminar – Management and Treatment of Municipal Landfill Leachate, Venice, Italy, 2-4 December, pp.iii.1 to iii.13.

Robinson H.D, Chen C.K., Formby R.W. and Carville M.S. 1998. Leachate Treatment Plants at Hong Kong Landfill Sites. Proceedings from the International Training Seminar - Management and Treatment of MSW Landfill Leachates, Venice, Italy 2-4 December pp 6.

Robinson H.D. and Harris G. 2000. Use of Reed Bed Polishing Systems to Treat Landfill Leachtates to Very High Standards. Proceedings from the International Training Seminar – Control, Management and Treatment of Landfill Emissions. Unv of Natal, Durban, SA. 6-8 Dec 2000.

Robinson H.D. and Harris G. 2001. EnvirosAspinwall (UK) Leachate Safari, April 2001. Personal interview. In Olufsen 2003.

Rodic-Wiersma L.J. and Goossens L.H.J. 2001. Assessment of Landfill Technology Failure. Proceedings from Sardinia 2001: the Eighth International Waste management and Landfill Symposium. Vol I pp 695-704.

Rogers F.E.H, Rogers K.H. and Buzer J.S. 1985. *Wetlands for Wastewater Treatment – With Special Reference to Municipal Wastewaters.* Witswatersrand University Press. Johannesburg, South Africa.

Röhrs L.H., Fourie A.B., and Blight G.E. 2000. Is 30 Years an Appropriate After-Care Period for Municipal Landfills? Proceedings from WasteCon2000: the Biennial Conference and Exhibition. Vol 2 p 369-378.

Röhrs L.H., Fourie A.B., and Blight G.E. 2001. Modeling Landfill Leachate Quality to Determine Long-Term Pollution Potential of Landfills. Proceedings from Sardinia 2001: the Eighth International Waste management and Landfill Symposium. Vol I pp169-177.

Röhrs L.H. and Fourie A.B. 2002. The Myth of Sustainable Landfills in South Africa. Proceedings from the Third International Conference and Exhibition on Integrated Environmental Management in Southern Africa. Johannesburg, South Africa. 27-30 August.

Rollin A.L., Mlynarek J., Lafleur J., Zanescu A. 1991. The Investigation of a Seven Year-Old HDPE Geomemebrane Used in a Landfill. Proceedings from Sardinia 1991: the Third International Landfill Symposium. CISA, Cagliari, Vol II, pp 667-677.

SABS, method 845:1994. Particle and Relative Densities of Aggregates. 1st revision. South Africa: South African Standards Institution.

SABS, method 844:1994. Bulk Densities and Voids Content of Aggregates. 1st revision. South Africa: South African Standards Institution.

South Africa Department of Environmental Affairs and Tourism. 1996. An Environmental Policy for South Africa – Green Paper for Public Discussion.

South Africa (SA) Department of Water Affairs and Forestry (DWAF), 1998. Minimum

Requirements for Waste Disposal by Landfill. Waste Management Series, Pretoria, SA. First Edition 1984, Second Edition 1998.

South Africa (SA) Government Gazette (No. 20526 8 October). 1999. Department of Water Affairs and Forestry (DWAF), Section 21(f) and (h), pp. 12-18.

Shamrock J.R. 1998. A Comparative Study of the Decomposition Processes and Products of Rich and Poor Refuse in South Africa. MSc dissertation, Unv of the Witwatersrand. Johnannesburg, South Africa.

Stegmann R. and Spendlin H-H. 1982. Enhancement of Degradation: German Experiences. Saniatry In T.H. Christensen, R.Cossu and R.Stegmann (eds). Landilling: Process Technology and Environmental Impact. pp 61-82.

Stentsøe S. and Houe N.E. 2001. Sustainable Landfilling for the European Community. Proceedings from Sardinia 2001: the Eighth International Waste management and Landfill Symposium. Vol I pp 533-544.

Strachan L.J 1999. An Investigation into Biological Treatment Systems for Landfill Leachate. Masters Dissertation, Department of Civil Engineering, Construction and Surveying University of Natal Durban, South Africa.

Strachan L.J., Robinson H.D., Harris, G.R. and Olufsen J.S. 2000a. Full Scale, On-Site, Complete Treatment Solution for Landfill Leachate: A First for South Africa. Proceedings from WasteCon2000: the International Waste Management Biennial Conference and Exhibition, Somerset West, South Africa. September 2000.

Strachan L.J., Trois C., Robinson H.D. and Olufsen J.S. 2000b. Appropriate Biological Treatment of Landfill Leachates with Full Nitrification and Denitrification. Proceedings of the WISA Biennial Conference and Exhibition, May-June 2000.

Surface, J.M., Peverly T.H., Steenhuis T.S., and Sandford W.E. 1993. Effect of Season, Substrate Composition and Plant Growth on Landfill Leachate Treatment in Constructed Wetlands. *In:* G.A. Moshiri (ed), *Constructed Wetlands for Water Quality Improvement*. Lewis Publishers. pp 461-472.

Tchobanoglous G. 1993. Constructed Wetlands and Aquatic Plant Systems: Research, Design, Operational and Monitoring Issues. *In:* G.A. Moshiri (ed), *Constructed Wetlands for Water Quality Improvement.* Lewis Publishers. pp23-34.

Tchobanoglous G., Stowell R., Ludwig R., Colt J., and Knight A. 1979. The Use of Aquatic Plants and Animals for the Treatment of Wastewater: An Overview. *In:* F.E.H, Rogers K.H. and Buze J.S. (eds), *Wetlands for Wastewater Treatment – With Special Reference to Municipal Wastewaters*. South Africa. pp 30.

Tchobanoglous G. and Burton F.L. 1991. *Wastewater Engineering. Treatment, Disposal and Reuse*. Third edition. Metcalf and Eddy Inc. MacGraw-Hill.

Tortora G.J., Funke B.R. and Case C.L. 1989. *Microbiology: An Introduction*. Third Edition.

The Benjamin/Cummings Publishing Company, Inc. California pp. 664-691.

Trois C., Bowers A.J., Strachan L.J., and Schreiner H.D. 2000. An Investigation in to Landfill Emissions Under a Sub-Tropical Climate: Outlook on the "Cellular" Method. Proceedings from Wastecon2000: the International Waste Management Biennial Conference and Exhibition. Somerset West, South Africa. September 2000.

Trois C., Strachan L.J. and Olufsen J.S. 2002. Constructed Wetlands for the Treatment of Landfill Leachates: Meeting South African Discharge Directives. Proceedings from WasteCon2002: the International Waste Management Biennial Conference and Exhibition Durban, South Africa Vol II pp276.

Turner R.K. 1993. Sustainability: Principles and Practice. *In:* R.K. Turner (ed*), Sustainable Environmental Economics and Management, Principles and Practice.* John Wiley and Sons. Chichester, UK.

United Kingdom Department of Environment (UK DoE) and Welsh Office. 1995. Making Waste Work: A Strategy for Sustainable Waste Management in England and Wales. HMSO ISBN 010130402-1, London.

United Kingdom Department of the Environment (UK DoE). 1995. Landfill Design, Construction and Operational Practice. Waste management Paper 26G, HMSO, London.

United States Environmental Protection Agency (US EPA). 1989. Decision Makers Guide to Solid Waste Management. Washington, D.C. EPA/530-SW-89-072 (November).

US EPA website. 2003. Referenced on July 1, 2003. http://www.epa.gov/epaoswer/non-hw/muncpl/facts.htm

United States of America (USA) Code of Federal Regulations (CFR). 1977. 33 CFR – Chapter 26 - Part 1342 as of 1977.

United States of America (USA) Code of Federal Regulations (CFR). 2003. 40 CFR – Chapter1 - Part 258 as of June 27, 2003.

Ushikoshi K., Kobayashi T., Toji A., Kojima D, and Kobayashi P. 2001. Operating Results and Dioxins Removal Technology for Leachate Treatment by Reverse Osmosis System. Proceedings from Sardinia 2001: the Eighth International Waste Management and Landfill Symposium. Vol 1. pp 383-390.

Vasel J.L. 2002. Are Constructed Wetlands Sustainable Systems for Leachate Treatment? Proceedings from WasteCon2002 the International Waste Management Biennial Congress and Exhibition. Durban, South Africa. Vol 2. pp 277-289.

Van Oostrom J.J. 1995 Nitrogen Removal in Wetlands Treating Nitrified Processing Effluent. *Water Science and Technology.* Vol 32. N3 pp137-147.

Water Research Centre (WRC). 1990. Methane Recovery from the Anaerobic Digestion of

Landfill Leachate. Report ETSU B 1223, Prepared by the WRC for the Department of Energy's Renewable Energy Research and Development Programme, managed by the Energy Technology Support Unit (ETSU), report prepared by N.C. Blakey and P. Maris.

Wetzel R.G. 1993. Constructed Wetlands: Scientific Foundations Are Critical. *In:* G.A. Moshiri (ed), *Constructed Wetlands for Water Quality Improvement.* Lewis Publishers. pp 3-8.

Whitlow R. 1995. Basic Soil Mechanics. 3rd edition. Addeson Wesley Longman Limited.

Williams P.T. 1998. *Waste Treatment and Disposal.* John Wiley & Sons Ltd. Chichester, West Sussex, England.

Wood A. 1990. Constructed Wetlands for Wastewater Treatment – Engineering and Designing Considerations. Proceedings from the International Conference for the Use of Constructed Wetlands in Water Pollution Control. Cambridge, England. 24-28 September.

Wood A. 1996. Investigation into the Application and Performance of Constructed Wetlands for Wastewater Treatment in South Africa. Paper presented at WISA Biennial Conference and Exhibition, Port Elizabeth, May. Paper No. 78.

Wood A. 1999. Investigation into the Application and Performance of Constructed Wetlands for Wastewater Treatment in South Africa. Water Research Commission (WRC) Report No 416/1/99. pg 2-4.

World Commission on Environment and Development (WCED). 1987. Our Common Future. Headed by G.H. Brundtland. Oxford University Press.

World Health Organization (WHO) 1995. Landfill. WHO – Regional Office for Europe, Environmental health planning pamphlet series – 9.

APPENDIX A

NOTE: Information regarding the drainage regions referred to in Table A1 and A2 can be obtained from DWAF, upon written request.

Table A1: Areas excluded from General Authorization for discharges to water resources

Primary drainage region	Tertiary drainage region	Description of main river in drainage region
B	B11, B12 B20 B31, B32 B41, B42 B60	Olifants River Wilge River Olifants River Steelpoort River Blyde River
W	W51, W52, W53, W54, W55, W56, W57	Usutu River
X	X11, X12, X13, X14 X21, X22, X23, X24 X31, X32, X33 X40	Nkomati River

Table A2: Listed Water Resources.

	WATER RESOURCE
1	Hout Bay River to tidal water
2	Eerste River to tidal water
3	Lourens River to tidal water
4	Steenbras River to tidal water
5	Berg and Dwars Rivers to their confluence
6	Little Berg River to Vogelvlei weir
7	Sonderend, Du Toits and Elandskloof Rivers upstream and inclusive of The Waterskloof Dam
8	Witte River to confluence with Breede River
9	Dwars River to Ceres divisional boundary
10	Olifants River to the Ceres divisional boundary
11	Helsloot and Smalblaar (or Molenaars) Rivers to their confluence with Breede River
12	Hex River to its confluence with Breede River
13	Van Stadens River to tidal water
14	Buffalo River from its source to where it enters the King Williams Town municipal area
15	Klipplaat River from its source to Waterdown Dam
16	Swart Kei River to its confluence with the Klipplaat River
17	Great Brak River
18	Bongola River to Bongola Dam
19	Kubusi River to the Stutterheim municipal boundary
20	Langkloof River from its source to Barkly East municipal boundary
21	Kraai River to its confluence with the Langkloof River
22	Little Tsomo River
23	Xuka River to the Elliot district boundary
24	Tsitsa and Inxu Rivers to their confluence
25	Mvenyane and Mzimvubu Rivers from sources to their confluence
26	Mzintlava River to its confluence with the Mvalweni River

	WATER RESOURCE
27	Ingwangwana River to its confluence with Umzimkulu River
28	Umzimkulu and Polela Rivers to their confluence
29	Elands River to the Pietermaritzburg-Bulwer main road
30	Umtamvuma and Weza Rivers to their confluence
31	Umkomaas and Isinga Rivers to their confluence
32	Lurane River to its confluence with the Umkomaas River
33	Sitnundjwana Spruit to its confluence with the Umkomaas River
34	Inudwini River to the Polela district boundary
35	Inkonza River to the bridge on the Donnybrook-Creighton road
36	Umlaas to the bridge on District Road 334 on the farm Maybole
37	Umgeni and Lions River to their confluence
38	Mooi River to the road bridge at Rosetta
39	Little Mooi and Hlatikula Rivers to their confluence
40	Bushmans River to Wagendrift Dam
41	Little Tugela River and Sterkspruit to their confluence
42	M'Lambonjwa and Mhlawazeni Rivers to their confluence
43	Mnweni and Sandhlwana Rivers to their confluence
44	Tugela River to its confluence with the Kombe Spruit
45	Inyamvubu (or Mnyamvubu) River to Craigie Burn Dam
46	Umvoti River to the bridge on the Seven Oaks-Rietvlei road
47	Yarrow River to its confluence with the Karkloof River
48	Incandu and Ncibidwane Rivers to their confluence
49	Ingogo River to its confluence with the Harte River
50	Pivaan River to its confluence with Soetmelkspruit
51	Slang River and the Wakkerstroom to their confluence
52	Elands and Swartkoppie Spruit to their confluence
53	All tributaries of the Komati River between Nooitgedacht Dam and its confluence with and including Zevenfontein Spruit
54	Seekoeispruit to its confluence with Buffelspruit
55	Crocodile River and Buffelskloofspruit to their confluence
56	All tributaries of the Steelpoort River down to its confluence with and including the Dwars River
57	Potspruit to its confluence with the Waterval River
58	Dorps River (or Spekboom River) to its confluence with the Marambanspruit
59	Ohrigstad River to the Ohrigstad Dam
60	Klein-Spekboom River to its confluence with the Spekboom River
61	Blyde River to the Pilgrim's Rest municipal boundary
62	Sabie River to the Sabie municipal boundary .
63	Nels River to the Pilgrim's Rest district boundary
64	Houtbosloop River to the Lydenburg district boundary
65	Blinkwaterspruit to Longmere Dam
66	Assegaai River upstream and inclusive of the Heyshope Dam
67	Komati River upstream and inclusive of the Nooitgedacht Dam and the Vygeboom Dam
68	Ngwempisi River upstream and inclusive of Jericho Dam and Morgenstond Dam
69	Slang River upstream and inclusive of Zaaihoek Dam
70	All streams flowing into the Olifants River upstream and inclusive of Loskop Dam, Witbank Dam and Middelburg Dam
71	All streams flowing into Ebenezer Dam on the Great Letaba River
72	Dokolewa River to its confluence with the Politzi River
73	Ramadiepa River to the Merensky Dam on the farm Westfalia 223, Letaba
74	Pienaars River and tributaries as far as Klipvoor Dam

	WATER RESOURCE		
	RAMSAR LISTED WETLANDS:	PROVINCE	LOCATION
75	Barberspan	North-West	26°33' S 25°37' E
76	Blesbokspruit	Gauteng	26°17' S 28°30' E
77	De Hoop Vlei	Western Cape	34°27' S 20°20' E
78	De Mond (Heuningnes Estuary)	Western Cape	34°43' S 20°07' E
79	Kosi Bay	Kwazulu-Natal	27°01' S 32°48' E
80	Lake Sibaya	Kwazulu-Natal	27°20' S 32°38' E
81	Langebaan	Western Cape	33°06' S 18°01' E
82	Orange River Mouth	Northern Cape	28°40' S 16°30' E
83	St Lucia System	Kwazulu-Natal	28°00' S 32°28' E
84	Seekoeivlei Nature Reserve	Free State	27°34' S 29°35' E
85	Verlorenvlei	Western Cape	32°24' S 18°26' E
86	Verloren Valei	Mpumalanga	25°14' S 30°4' E
87	Nylsvlei	Northern	24°39' S 28°42' E
88	Wilderness Lakes	Western Cape	33°59' S 22°39' E

APPENDIX B

Table B1: Class-A-Pan Data (courtesy of Courtesy of the South African Weather Bureau)

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
1957	86.9	80.5	136.4	132.1	170.2	155.7	209	*	*	*	*	*
1958	179.6	169.7	178.6	136.9	113	95.5	91.7	128.3	142.2	206.8	184.4	194.3
1959	205	178.3	212.3	160.8	93.5	85.9	96	122.9	152.7	162.1	170.2	191.8
1960	214.9	159	169.4	106.9	92.7	77.2	88.4	102.4	149.9	144	176.3	186.7
1961	211.3	176	164.8	102.9	91.7	74.4	86.4	126.5	140.2	170.4	162.3	205.2
1962	187.7	169.7	171.2	141.5	111.3	89.9	107.2	133.6	146.3	146.6	160.5	193
1963	184.7	189.5	132.8	125.7	114.6	67.6	70.6	119.9	120.9	125	175.5	206
1964	179.3	184.7	175.8	128.5	94.5	83.6	90.9	122.9	121.7	89.2	155.4	191
1965	180.6	173.5	179.6	142.5	110.2	89.2	76.2	106.2	122.4	150.9	129.8	208.5
1966	183.9	164.8	223	116.1	95.2	75.9	109.5	121.4	154.4	175.3	177	206
1967	191.8	164.8	160.5	109	99.6	88.1	104.4	146.6	147.8	210.3	180.3	232.9
1968	195.5	196.6	143.6	134	111.7	110.4	89.9	104.7	135.3	148.6	166.6	210.3
1969	254.5	171.1	151.1	126.3	99.5	89.6	103.5	145.5	120.1	147.2	151.6	187
1970	205.8	190	235	152.4	96.5	102.2	106.6	119.2	139.4	156.9	142.2	213.2
1971	163.6	181.5	156.5	120.1	92.3	92.2	87.5	127.8	95	157.6	182.9	183.2
1972	243.3	171.4	196.6	143	80.6	80.3	95.9	112.1	139.4	171	196.6	205.9
1973	191.2	183.6	169.9	131	112.7	93.3	108.6	103.2	123.6	167.4	190.7	230.6
1974	198.2	164.2	178.9	139.4	128.3	86.6	116.2	146.9	171.4	172.6	177.7	187.5
1975	198.2	150.1	147.3	111	85.2	78.6	96.3	123.9	103.8	164.6	168.6	214.1
1976	249.7	208.4	214.7	144.8	125.7	132.7	149	164.1	165.3	200.3	208.6	*
1977	190.4	166.1	158.4	146.7	136.1	100.5	120.6	129.9	143.7	160.3	188.3	222.1
1978	200.9	157.2	168.2	133.5	109.6	91.1	88.1	84	128.9	159.2	166.4	228.5
1979	217.8	219.6	207.3	141.8	112.5	113.7	107.1	86.8	131.7	165.4	184.1	196.3
1980	265	209.1	197.6	165.5	127.2	104	103.7	131.4	130.5	191.1	196.6	276
1981	182.6	162.1	205	176.8	107.7	104.9	112.3	130.5	144.3	222.8	185.1	219.2
1982	193.9	210.3	196.2	141.9	124.2	99.2	121.1	153.2	174.3	192	225.9	258.4
1983	234.6	215.9	212.7	172.6	152.1	100.3	130.8	160.2	163.3	165.7	154.2	200
1984	192.7	183.7	155	156.4	117.9	85.4	87.5	171.7	148.8	153.6	182	221.7
1985	197.8	160.7	183.5	172	136	103.4	144	168.8	173.9	204.5	222	238.7
1986	228.2	234.4	282.4	128.3	121.8	107.4	112	124.2	139.1	168.8	169.2	203.5
1987	166.1	202.1	177.4	149.6	129.5	99.2	113	145	104.1	137.8	138.8	173.1
AVE	203	182.3	183.5	138.6	110.8	91.9	102.6	128.5	138.9	165.3	174.8	209.8
YEAR	AVE	:	1830									

APPENDIX C

Table C1: Durban, South Africa rainfall data during the treatability trials (courtesy of the South African Weather Bureau)

Date	Day in trial	Total weekly rainfall (mm/week)	Mean daily rainfall (mm/day)
20-Feb	3	6	0.86
27-Feb	10	2.4	0.34
6-Mar	17	16.8	0.89
13-Mar	24	0	0.00
20-Mar	31	63.8	9.11
27-Mar	38	19.2	2.74
3-Apr	45	0	0.00
10-Apr	52	19.4	2.77
17-Apr	59	87.8	12.54
24-Apr	66	6.6	0.94
1-May	73	1.8	0.26
8-May	80	0	0.00
15-May	87	21	3.00
22-May	94	1.8	0.26
29-May	101	0	0.00
5-Jun	108	0	0.83
12-Jun	115	0	0.00
19-Jun	122	28.6	4.09
26-Jun	129	11.6	1.66
3-Jul	136	10.2	1.46
10-Jul	143	0.4	0.06
17-Jul	150	0	0.00
24-Jul	157	1.4	0.20
31-Jul	164	0	0.00

Table C2 : *Phragmites australis* evapotranspiration data (modified from Oulfsen 2003)

Date	Day in trial	Mean ET loss (mm/day)	Mean ET loss (liter/day)	Estimated effluent (liter/day)
20-Feb	3	3.26	10.43	29.57
27-Feb	10	3.83	12.27	27.73
6-Mar	17	4.61	14.76	25.24
13-Mar	24	5.02	16.05	23.95
20-Mar	31	3.35	10.73	29.27
27-Mar	38	3.88	12.43	27.57
3-Apr	45	4.62	14.77	25.23
10-Apr	52	3.76	12.03	27.97
17-Apr	59	4.86	15.55	24.45
24-Apr	66	5.85	18.71	21.29
1-May	73	3.99	12.78	27.22
8-May	80	5.20	16.65	23.35
15-May	87	5.45	17.45	22.55
22-May	94	5.13	16.4	23.6
29-May	101	5.97	19.09	20.91
5-Jun	108	5.79	18.53	21.47
12-Jun	115	5.43	17.37	22.63
19-Jun	122	5.57	17.83	22.17
26-Jun	129	5.80	18.57	21.43
3-Jul	136	4.96	15.87	24.13
10-Jul	143	5.56	17.79	22.21
17-Jul	150	2.62	8.37	31.63
24-Jul	157	2.68	8.57	31.43
31-Jul	164	2.48	7.93	32.07

APPENDIX D

Table D1: Results of the analysis of the chemical concentrations in the influent.

Day in Trial	Alkalinity ($mgCaCO_3/l$)	Ammonia ($mgNH_3/l$)	BOD (mgO_2/l)	Calcium (mgCa/l)	Chloride ($mgCl^-/l$)	COD (mgO_2/l)	Conductivity (µS/m)	Iron (mgFe/l)	Lead (mgPb/l)	Magnesium (mgMn/l)	Nitrate/Nitrite (mg NO_3+NO_2/l	Ortho phosphate ($mgPO_4/l$)	pH
3	1075	0.5	-	5	600	463	687	<0.1	<0.05	7.1	0.1	4.6	8.4
10	1205	0.62	-	49	293	323	742	<0.1	<0.05	130	0.53	1.8	8.1
17	1200	0.5	-	66	351	271	698	<0.1	<0.05	149	0.1	0.84	8.4
38	2308	245	-	85	327	384	677	1.3	<0.05	113	0.1	1.3	8.3
45	1700	115	100	95	790	230	552	<0.1	0.23	99	3	4.6	8
52	1621	116	140	88	783	225	522	0.3	0.17	90	4.8	3.3	7.8
59	1400	82	120	92	857	215	504	0.12	<0.05	95	24	3.9	8
66	1280	67	130	89	878	244	492	<0.1	<0.05	95	57	3.9	7.6
73	970	62	110	99	860	234	484	<0.1	<0.05	100	38	4	8.1
80	1147	20	140	110	818	193	481	0.13	<0.05	114	57	5	7.5
87	970	18	230	88	787	532	442	0.11	<0.05	95	23	5.4	7.7
94	1961	236	240	57	225	540	731	0.9	<0.05	86	0.1	0.23	8
101	1536	285	220	56	992	494	731	0.4	<0.05	98	1.2	0.97	8.1
108	1708	221	240	58	1417	558	669	0.51	<0.05	92	3.3	2.1	8
115	1270	138	160	58	1148	399	700	0.29	0.06	91	10	24	8.2
122	1508	116	170	55	1086	374	682	0.026	<0.05	92	23	2.8	8.1
129	2000	162	210	56	921	630	735	0.6	<0.05	98	7.3	2.2	8.5
136	1540	81	180	130	1227	422	730	0.8	<0.05	185	11	2.3	8.1
143	1706	187	210	58	1183	480	739	0.51	<0.05	96	37	2.2	8.3
150	1624	188	230	66	1079	500	721	0.65	<0.05	96	13	2.4	8.4
157	1803	274	230	70	1182	476	723	0.6	<0.05	120	8.7	2.5	8.3
164	2045	295	210	51	1305	402	784	0.8	<0.05	81	2.9	2	7.9

Table D2: Results of the analysis of the chemical concentrations in the effluent

Day in Trial	Alkalinity ($mgCaCO_3/l$)	Ammonia ($mgNH_3/l$)	BOD (mgO_2/l)	Calcium (mgCa/l)	Chloride ($mgCl^-/l$)	COD (mgO_2/l)	Conductivity (µS/m)	Iron (mgFe/l)	Lead (mgPb/l)	Magnesium (mgMn/l)	Nitrate/Nitrite (mg NO_3+NO_2/l	Ortho phosphate ($mgPO_4/l$)	pH
3	(only sampled from inlet)												
10*	480	2.2	-	71	1660	629	227	0.19	0.05	48	0.1	4.3	7.3
17*	550	39	-	88	473	140	243	0.1	0.05	51	51	0.93	7.6
38*	1150	63	-	62	248	157	353	0.18	0.05	61	16	0.32	7.7
45	970	0.5	30	107	617	86	358	0.17	0.08	75	0.1	1.3	7.5
52	990	5.4	70	100	792	141	394	0.11	0.09	78	0.1	0.93	7.6
59	840	9.2	80	97	814	147	408	0.1	0.05	82	0.18	0.63	7.4
66	750	0.5	80	87	824	179	416	0.1	0.05	94	1.8	1.5	7.9
73	1000	0.5	120	89	778	262	430	0.1	0.05	102	0.39	1.2	8.5
80	911	0.5	110	84	994	243	494	0.28	0.05	89	0.19	0.22	7.9
87	940	0.5	140	94	1325	331	423	0.1	0.05	98	0.35	0.24	7.8
94	930	3.3	110	87	1076	246	479	0.1	0.05	87	0.3	0.3	7.9
101	737	1.9	130	135	772	343	480	0.91	0.05	86	2	0.53	8.4
108	1190	33	190	106	1240	453	600	0.1	0.05	132	0.18	3.4	7.5
115	1361	9.3	190	113	93	456	670	0.89	0.05	144	0.25	0.88	7.8
122	1413	15	160	102	1039	384	680	0.68	0.05	138	10	0.26	7.6
129	1314	37	70	85	1099	282	598	1.3	0.05	113	8.7	0.22	7.8
136	1290	11	140	113	1207	338	653	0.64	0.05	132	9	0.1	7.6
143	1000	17	220	90	676	366	666	0.16	0.05	118	19	0.1	7.6
150	1175	30	190	111	992	425	719	0.14	0.05	131	23	0.91	7.8
157	1350	55	190	143	1459	430	748	0.21	0.05	178	43	0.23	7.8
164	1383	47	290	66	1397	638	814	0.17	0.05	80	32	0.16	7.5

Note: *Samples from Days 10, 17 and 38 were taken from Pipe #2.
The breaks in the data represent a new batch of leachate