FRASER RIVER
ACTION PLAN

Sewage Lagoon Design Using Wetlands & Other Upgrading Technologies to Achieve Non-Acutely Toxic Effluent

–

2nd Revised Edition

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SEWAGE LAGOON DESIGN USING WETLANDS AND OTHER UPGRADING TECHNOLOGIES TO ACHIEVE NON-ACUTELY TOXIC EFFLUENT

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DOE FRAP 1994 - 34

Prepared for:

Environment Canada
Environmental Protection
Fraser Pollution Abatement Office
224 West Esplanade
North Vancouver, B.C.
V7M 3H7

and

British Columbia Ministry of Environment, Lands & Parks
Environmental Protection Division
777 Broughton Street
Victoria, B.C.
V8V 1X5

Prepared by:

NovaTec Consultants Inc.
224 West 8th Avenue
Vancouver, B.C.
V5Y 1N5

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DISCLAIMER

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Any comments regarding this report should be forwarded to:

Fraser Pollution Abatement Office
Environment Canada
224 West Esplanade
North Vancouver, B.C.
V7M 3H7

EXECUTIVE SUMMARY

The Federal Green Plan pollution abatement component of the Fraser River Action Plan directs that action be taken to identify and control contaminants entering the Fraser River Basin from both point and non-point sources. One of the key issues of concern to both the Federal and Provincial governments is to identify whether such sources are acutely toxic to fish and to identify remedial actions which can be taken reduce any identified fish toxicity as identified using the LC_{50} test. Sewage lagoons which discharge into the Fraser River Basin are expected to consistently (i.e. on a year-round basis) achieve sufficiently low ammonia concentrations to ensure the effluent is not acutely toxic to fish.

The toxicity of lagoon effluents treating primarily domestic wastewater is believed to be most often the result of elevated concentrations of total ammonia and/or free ammonia, chlorine residuals (where chlorination is practiced), or hydrogen sulphide resulting from anaerobic sludge decomposition. The ability of lagoon systems to remove ammonia depends primarily on pH, detention time, season, and liquid temperatures. The major removal pathway is volatilization at the water surface during warm weather, which is greatly diminished during cold weather periods, particularly where there are conditions of ice cover.

This document reviews historical lagoon performance data, obtained for various locations in Canada, with respect to effluent ammonia concentrations and fish toxicity. The data suggests that a total ammonia concentration of 10 to 11 mg/L-N is a critical level, above which effluents are typically toxic, and below which effluents are typically non-toxic. Using an arbitrary factor of safety of two, it is recommended that a total ammonia concentration of 5 mg/L-N be used as a design criteria.

In addition to presenting design guidelines for lagoon systems, the document presents guidelines for upgrading technologies to reduce effluent ammonia levels. Wetland systems are one of the upgrading technologies described which has received considerable attention in recent years in terms of their application in further treating lagoon effluents prior to discharge, and reducing fish toxicity. The other technologies described include: 1) storage basins; 2) rapid infiltration; 3) overland flow; and 4) fixed film processes. Upgrading technologies have limited abilities to remove ammonia during cold weather, and should be considered in conjunction with winter storage basins.

The guideline document focuses on factors to enhance the removal of ammonia, but also includes recommendations which will result in overall operational and design improvements (i.e. pre-treatment, size, configuration, baffles, aeration, diffused inlet/outlets structures, liners, operating depth, hydraulic retention time, sludge management, maintenance, etc.). Capital and operating costs are also presented for use in comparing the alternative treatment technologies described.

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NovaTec Consultants Inc.
224 West 8th Avenue
Vancouver, British Columbia
V5Y 1N5

The following individuals contributed to the authorship of this document:

Dr. Troy D. Vassos, P.Eng.
Mr. Sherwood Reed, P.E.
Mr. Dean Reil, M.Eng., P.Eng.
Ms. Zorica Knezevic, M.A.Sc., P.Eng.
Dr. O. (Sam) Turk, P.Eng.
Mr. Greg J. Bull, M.Eng., P.Eng.
Ms. Catherine Ponsford, E.I.T.
Mr. Bill Shu, P.Eng.

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- Dr. R. (Mano) Manoharan, P.Eng. of the Ontario Ministry of Environment and Energy;
- Mr. Terry Underwood, P.Eng., of T.R. Underwood Engineering;
- Mr. S.N. Lawrence, P.Eng. of L&M Engineering Ltd.;

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GLOSSARY OF ABBREVIATIONS

NH_3	Ammonia (un-ionized)
NH_4^+	Ammonia (ionized)
$NH_3 + NH_4^+$	Total Ammonia
H_2S	Hydrogen Sulphide
TKN	Total Kjeldahl Nitrogen
BOD_5	5 day Biochemical Oxygen Demand
TSS	Total Suspended Solids
COD	Chemical Oxygen Demand
TOC	Total Organic Carbon
LC_{50}	50th percentile lethal concentration
USGPD	US gallons per day
mg	milligrams
L	litres
mg/L	milligrams per litre
pH	negative log of the hydrogen ion concentration
µg	microgram
µg/L	microgram per litre
TN	total nitrogen
mg/L-N	milligrams per litre as nitrogen (N)
m^3	cubic metres
m^2	square metres
m^3/d	cubic metres per day
mil	1/1000 inch = .025 mm

1 INTRODUCTION

1.1 BACKGROUND

The Federal Green Plan pollution abatement component of the Fraser River Action Plan directs that action be taken to identify and control contaminants entering the Fraser River Basin from both point and non-point sources. One of the key issues of concern to both the Federal and Provincial governments is to identify whether such sources are acutely toxic to fish (as defined by the Environment Canada (1990) 96-hr LC_{50} rainbow trout bioassay criteria), and to identify remedial actions which can be taken to reduce any identified toxicity.

In the absence of a high proportion of industrial discharges to the wastewater entering a lagoon system (pond), effluent toxicity is often the result of elevated concentrations of total ammonia and/or free ammonia, chlorine residuals (where chlorination is practiced), or hydrogen sulphide resulting from anaerobic sludge decomposition. Of the three, ammonia is the most common potentially toxic component of domestic sewage. Ammonia removal in lagoon systems is related to several factors such as pH, detention time, season, and liquid temperatures. The pH fluctuates as a result of algal-carbonate interactions, mitigated by the amount of alkalinity in the water. The major removal pathway is volatilization at the water surface during warm weather. In cold weather climates, particularly where there is ice cover on the lagoons during the winter months, ammonia removal is not consistent throughout the year. The Province's proposed Municipal Sewage Discharge Criteria allows for ammonia to be removed from the sample tested for acute toxicity, or alternatively allows the pH to be lowered to reduce the proportion of free ammonia, to reduce ammonia toxicity.

Even if this criteria is applied there are other sources of toxicity, such as hydrogen sulphide toxicity, which have to be taken into consideration in designing lagoon systems to produce non-acutely toxic effluent.

Existing sewage lagoons which discharge into the Fraser River Basin do not consistently produce (i.e. on a year-round basis) a non-acutely toxic effluent. Where it is desirable to achieve a non-acutely toxic effluent on a year round basis, additional upgrading technologies must be considered. Before such technologies can be implemented, it is necessary to determine to what degree toxicity can be reduced in lagoon systems. Consequently, there is a need for design guidelines for use in assessing the capability of existing and proposed lagoon system to reduce effluent toxicity, under the temperate and cold weather conditions found in the Province of British Columbia.

Wetland systems have received considerable attention in recent years in terms of their application in further treating lagoon effluents prior to discharge, and reducing effluent toxicity. However, similar to lagoon systems, there is limited information available to evaluate such systems for use under the climatic conditions within British Columbia, and design guidelines are required. Generally, given the cold climates in most of British Columbia, wetlands can only be used during the summer in conjunction with winter storage ponds.

1.2 DOCUMENT PURPOSE AND SCOPE

The purpose of this document is to present design guidelines for lagoon, and constructed wetland systems to achieve a non-acutely toxic effluent in temperate and cold climate conditions. The document also presents information on alternative upgrading technologies for consideration in reducing lagoon effluent toxicity including: 1) storage basins; 2) rapid infiltration; 3) overland flow and; 4) fixed film processes.

Recommendations are provided for minimum standards for the design of lagoon and constructed wetland systems, and key design considerations are presented for the alternative upgrading technologies. The guideline focuses on factors to enhance the removal of ammonia, but also includes recommendations which will result in overall operational and design improvements (i.e. pre-treatment, size, configuration, baffles, aeration, diffused inlet/outlets structures, liners, operating depth, hydraulic retention time, sludge management, maintenance, etc.). Capital and operating costs are also presented for use in comparing the alternative treatment technologies described.

1.3 DOCUMENT STRUCTURE

The guideline document consists of seven sections. The first section introduces the background to the regulatory concerns, with respect to lagoon effluent toxicity, which have resulted in the need to prepare a design guideline manual. The second section describes general categories of potential effluent contaminants, and describes how these contaminants can influence acute lagoon effluent toxicity. Section three introduces lagoon systems, and reviews expected performance data from the literature and data obtained from operating systems in British Columbia. A recommendation is also made regarding the use of target effluent ammonia criteria to minimize effluent toxicity. Section four describes facultative and aerobic lagoon design and construction criteria considerations are presented which will minimize effluent ammonia concentrations. Section five presents design criteria for

constructed wetland systems to further remove ammonia from lagoon effluents. Section six presents general design considerations for five alternative upgrading technologies to wetland systems for reducing effluent ammonia levels. Comparative capital and operating costs are presented for the lagoon, wetland, and upgrading technologies in section seven. Finally, example calculations are presented in section eight.

REFERENCES

Environment Canada. 1990. Biological Test Method: Acute Lethality Test Using Rainbow Trout, EPS 1/RM/9, July 1990.

2 LAGOON WASTEWATER CHARACTERISTICS

2.1 CONTAMINANTS

Lagoon effluents can contain a variety of contaminants related to the proportion of industrial, commercial, residential, and storm water sources entering the lagoon system. Although certain pollutants are highly soluble, and reside in the liquid phase, many contaminants can become adsorbed to suspended particles which are removed from the effluent. These contaminants include phosphorus, ammonia, biodegradable organic matter, bacteria, pesticides, halogenated and non-halogenated hydrocarbons, metals and some organic toxic contaminants. The degree to which toxic contaminants are removed through sedimentation or solids removal (i.e. filtering) depends on the chemical behaviour of the contaminant (i.e. solubility and surface chemistry).

Pollution occurs where waste materials are discharged to the environment affecting a change in that environment. This change can be caused by the adaptation of organisms in the receiving environment to the waste material (i.e. excess algal growth due to elevated nutrient levels), by indirect effects (i.e. depletion of dissolved oxygen by bacteria as a result of organic discharges), or by direct effects (i.e. benthic smothering due to discharged solids, or organism toxicity due to toxic contaminants in the discharge). Although toxicity may be caused by a single elevated toxic contaminant, typically toxicity data is difficult to interpret as to the cause of toxicity, due to the complex nature of the contaminants present in the wastewater. There are five general groups of potential effluent contaminants:

- conventional;
- toxic organic contaminants;
- toxic metals;
- pathogenic organisms;
- floatable materials.

2.1.1 Conventional Parameters

This group consists of parameters which are associated with overall effects on aquatic organisms. They typically include: biochemical oxygen demand (BOD_5), chemical oxygen demand (COD), various forms of nitrogen and phosphorus, and total suspended solids (TSS). Other physical/chemical characteristics often included in this group include temperature, pH, and alkalinity. With respect to effluent toxicity, ammonia nitrogen and pH are the key conventional parameters of interest. Ammonia nitrogen is discussed in Section 2.2. Hydrogen sulphide, a byproduct of anaerobic

biodegradation of organic material, can also be considered a conventional parameter of interest for lagoon effluents.

2.1.2 Toxic Organic Contaminants

This group consists of organic compounds which can be toxic to aquatic organisms. These compounds can either be present in a dissolved form, or can be adsorbed (attached) to sewage solids. Compounds present in a dissolved form at potentially toxic concentrations in lagoon effluents are generally rapidly diluted to non-toxic concentrations once discharged. Of greater concern are those associated with solid particles if they can be shown to accumulate near the discharge site, resulting in acute or chronic toxicity levels to benthic (bottom dwelling) organisms.

Toxic organic contaminants of particular interest in the Fraser River Basin include (Swain & Walton, 1988):

- Chlorinated phenols and polychlorinated biphenyls (PCBs);
- Polynuclear Aromatic Hydrocarbons;
- Organochlorine Pesticides;
- Volatile Organic Compounds (VOC).

Unless there is a significant industrial effluent contribution containing the above contaminants, their concentration in effluents originating from lagoon systems treating primarily domestic wastewater is usually near or below the analytical detection limit, and well below the acute aquatic toxicity levels.

2.1.3 Toxic Inorganic Contaminants

This group primarily consists of specific metals which can be toxic to aquatic organisms. Similar to organic contaminants, these metals can either be present in a dissolved form, or can be adsorbed (attached) to particulates in the effluent discharge. With the exception of lagoon systems receiving a high proportion of industrial wastewater, most metals are unlikely to be present in a dissolved form at potentially toxic concentrations in the lagoon effluents, and would be expected to be rapidly diluted to even lower concentrations within the receiving environment. Again, of possibly greater concern are those metals associated with suspended solids, which may accumulate near the discharge resulting in acute or chronic toxicity levels to benthic (bottom dwelling) organisms.

Metals of particular concern include the following:

- arsenic (As)
- copper (Cu)
- chromium (Cr)
- lead (Pb)
- zinc (Zn)
- cadmium (Cd)
- molybdenum (Mo)
- nickel (Ni)
- mercury (Hg)

2.1.4 Pathogenic Organisms

This group primarily consists of disease causing viruses, helminths, bacteria and protozoa which can present a significant risk to human health as a result of either incidental ingestion or even body contact. These micro-organisms are present either in suspension, or in association with suspended solids in the discharge, and their low concentrations make them extremely difficult and costly to detect. Consequently, other non-pathogenic sewage related bacteria, which are present in substantially higher numbers and are easier to detect, are used to indicate the level of health risk of either the wastewater effluent or the receiving water. Solid particles contained in non-disinfected effluents can be expected to contain high levels of pathogenic organisms, which can potentially accumulate in the sediments near the discharge and concentrate in shellfish and other benthic organisms.

2.1.5 Floatables

This group typically consists of waste plastic materials, oil and grease, and other materials which float on the surface and can accumulate along the shoreline if released into the receiving environment. These materials are an aesthetic problem, and can potentially pose a health risk with direct human contact. Floatables may be removed using pretreatment facilities, but is not typically recognized as a concern with lagoon systems. Typically the problem manifests itself in the form of public complaints regarding sewage related materials found along bathing beach areas.

2.2 TOXICITY

There is a growing thrust by regulatory agencies in Canada to adopt the 100% LC_{50} criteria to all effluent (including municipal) discharging to receiving waters. For example, a non-toxic effluent is now required for all municipal discharges in the Yukon. Ontario is in the process of making it compulsory for all municipal discharges not to be acutely toxic to fish in that Province.

The draft B.C. Municipal Pollution Control Objectives (1993) requires that municipal effluents discharged to the environment be non-acutely toxic to fish, without consideration for dilution. The requirement that a municipal effluent, discharging to a surface water, to successfully pass the fish bioassay test is becoming a standard permit requirement for the B.C. Ministry of Environment Land and Parks. For example, it is currently included in the Discharge Permit for the Cities of Chilliwack and Matsqui.

Through precedent law, the courts have adopted the LC_{50} test (Environment Canada, 1990) as a measure of deleterious and toxic substances to fish in discharges to receiving waters. This test involves subjecting a number of rainbow trout to various effluent dilutions over a period of 96 hours, and monitoring their appearance, behaviour, and mortality during that period. Failure occurs when more than 50 percent of the fish die in 100 percent effluent over the test period.

Ammonia is recognized as a major chemical constituent in treated municipal effluents that is responsible for failure of the 96 hr LC_{50} fish bioassay test. Elevated hydrogen sulphide concentrations can also be acutely toxic to aquatic organisms. Although toxicity effects can also result from the chlorination/dechlorination of lagoon effluents, this is not discussed in this report as chlorination is a supplementary step in effluent treatment prior to discharge into the receiving environment.

2.2.1 Ammonia Toxicity

Two forms of ammonia are common in lagoon wastewaters:

- Ionized ammonia (NH_4^+) and
- Un-ionized ammonia (NH_3)

The balance between the two forms is predominantly determined by pH and temperature according to the equation:

$$NH_3 + H_2O \rightleftharpoons NH_4^+ + OH^-$$

At higher pH and temperature values, the equilibrium balance is driven to a higher concentration of un-ionized ammonia. For example, the un-ionized ammonia concentration increases from 0.0621% of the total ammonia at a pH of 6.5 and temperature of 0°C, to 28.4% of the total ammonia at a pH of 9.0 and a temperature of 20°C (Emerson et al., 1975). *Total ammonia* refers to both the ionized and the un-ionized forms of ammonia. This is sometimes referred to in the literature simply as *ammonia*.

Of the two ammonia forms, un-ionized ammonia is the more toxic to trout as it is lipid soluble, therefore able to pass through fish membranes into the blood stream. Toxicity usually manifests itself through neurological disorders (acute toxicity) and/or the thickening of gill membranes, effectively reducing oxygen diffusion capacity (chronic toxicity), both effects eventually resulting in death of the organism (Haywood, 1983). Ionized ammonia is also toxic to fish, but in much higher concentrations (Lewis, 1988).

2.2.2 Hydrogen Sulphide

Hydrogen sulphide gas (H_2S) is a byproduct of anaerobic biodegradation of organic material by bacteria at the bottom of sewage lagoons. Similar to ammonia, it is present in either an unionized or ionized form, depending on the pH of the water. The unionized form is the more toxic of the two forms. An equilibrium is reached in the water column according to the following equation:

$$H_2S \rightleftharpoons HS^- + H^+$$

Low pH shifts the equilibrium to the left, yielding a greater concentration of un-ionized hydrogen sulphide and increasing toxicity. The presence of significant H_2S concentrations in lagoon effluent is relevant to fish toxicity in two ways; one, H_2S is toxic itself to fish and two, the presence of H_2S is indicative of lower dissolved oxygen concentrations resulting from excess organic demand (see Section 2.2.3). Hydrogen sulphide is reported to be acutely toxic to rainbow trout at concentrations as low as 8.7 µg/L (Ontario, MOE, 1990).

In lagoons with no ice cover, the hydrogen sulphide is usually released as a gas at the surface, or it can be trapped in the upper layers under alkaline conditions. In the winter, ice cover can prevent release, and the gas remains dissolved in the water column or trapped in the sediment. Lagoon effluents discharged during periods of ice cover can have elevated hydrogen sulphide concentrations. In the spring, rising temperatures cause the ice cover to melt and can create an inverted thermal gradient to occur (where the surface water is colder than the bottom water), resulting in sudden mixing and the release of trapped hydrogen sulphide from the sediments and elevated effluent concentrations. Similar mixing conditions can also occur during the fall.

Upgrading technologies which reduce ammonia concentrations concurrently reduce hydrogen sulphide concentrations. Hydrogen sulphide toxicity is typically a minor secondary form of toxicity in comparison to ammonia toxicity.

2.2.3 Other Effluent Characteristics

The lagoon effluent components described below can have a effect on effluent toxicity, but are generally a minor consideration with respect to lagoon systems treating primarily domestic wastewater.

1. Oxygen depleting substances, the organic component of which is measured by BOD_5, will lower the dissolved oxygen levels during the bioassay test. Lower dissolved oxygen levels make the fish more susceptible to toxic constituents.

2. The presence of more than one toxic compound in lagoon effluents will generally have a synergistic effect (i.e. the individual toxicity threshold concentrations of each parameter are effectively lowered) (Haywood, 1983). This effect may be partially responsible for the wide range of lagoon effluent ammonia concentrations which are correlated with fish toxicity.

3. Elevated levels of organic and inorganic toxic contaminants as a result of a high proportion of industrial wastewater entering the lagoon system can contribute to effluent acute toxicity.

2.2.4 Effluent Toxicity Criteria

Factors affecting effluent toxicity, such as those noted in Section 2.2.3, cannot be readily incorporated into a lagoon design criteria. For example, it is not possible to predict the synergistic effects of low levels of toxic contaminants on acute toxicity, nor is it possible to accurately predict

the effluent organic and inorganic toxic contaminant levels in lagoon effluent resulting from mixed industrial wastewater contributions. However, there appears to be a reasonable correlation between total ammonia concentration and acute toxicity for lagoon effluents which can be used to set a reasonable design criteria objective. Consequently, for the purposes of this document, it is recommended that effluent toxicity reduction design considerations be focused on ammonia removal. The actual design criteria objectives are discussed in Section 3.2.4.

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3 LAGOON SYSTEMS

3.1 GENERAL

Lagoon systems are the most commonly used wastewater treatment process in Canada. They are widely used to treat wastewaters from small communities, primarily due to their low capital and operating costs, and taking advantage of the large and inexpensive land areas generally available around such communities. The natural processes, and cycles, which lagoon systems use to treat the wastewater are described in this section.

3.1.1 Limnology

A lagoon system contains two main types of microbial cells which are responsible for the stabilization of wastes:

- procaryotes (do not have nucleus);
- eucaryotes (have a true nucleus).

Bacteria and cyanobacteria (known as blue-green algae) are procaryotes and algae, fungi, and protozoa are eucaryotes. Lagoon systems also contain helminths and higher animals such as crustaceans and rotifers, aquatic plants and insects.

The aerobic, facultative and anaerobic bacterial populations present in lagoons change in response to environmental conditions. The aerobic (i.e. require oxygen) bacteria are mostly heterotrophic, gram-negative bacteria, but also include autotrophic organisms in fewer numbers. These organisms decompose the organic materials present in the near-surface aerobic (oxygenated) zone of the lagoon, where the dissolved oxygen can be readily replenished from the atmosphere. Bacteria in the lower water depths along the bottom of the lagoon, and within the sediments, are usually anaerobic (i.e. do not require oxygen). The anaerobic benthic zone of the lagoon contains two main groups of obligate anaerobes:

- acid formers
- methane producers

Acid forming bacteria are heterotrophs that convert complex organic matter into simple acids and alcohols. The activity of these bacteria is important since they provide substrates for the final

reduction of the organic material by methane producing methanogens. A population of facultative anaerobes is found in the transition zone between aerobic and anaerobic conditions.

Other groups of bacteria (aerobic or anaerobic) such as purple sulphur bacteria are involved in oxidation and reduction of sulphur. Purple sulphur bacteria occupy the anaerobic layer below the algae, cyanobacteria, and other aerobic bacteria in the pond. Conversion of odorous sulphide compounds to elemental sulphur or sulphate by sulphur bacteria is a significant factor for odour control in lagoons.

Cyanobacteria are aerobes. Species found in lagoons are nonmotile photoautotrophs unable to grow in the dark and on organic substrates. Cyanobacteria assimilate simple inorganic compounds while utilizing carbon dioxide as the major source of carbon, producing oxygen as a by-product of photosynthesis.

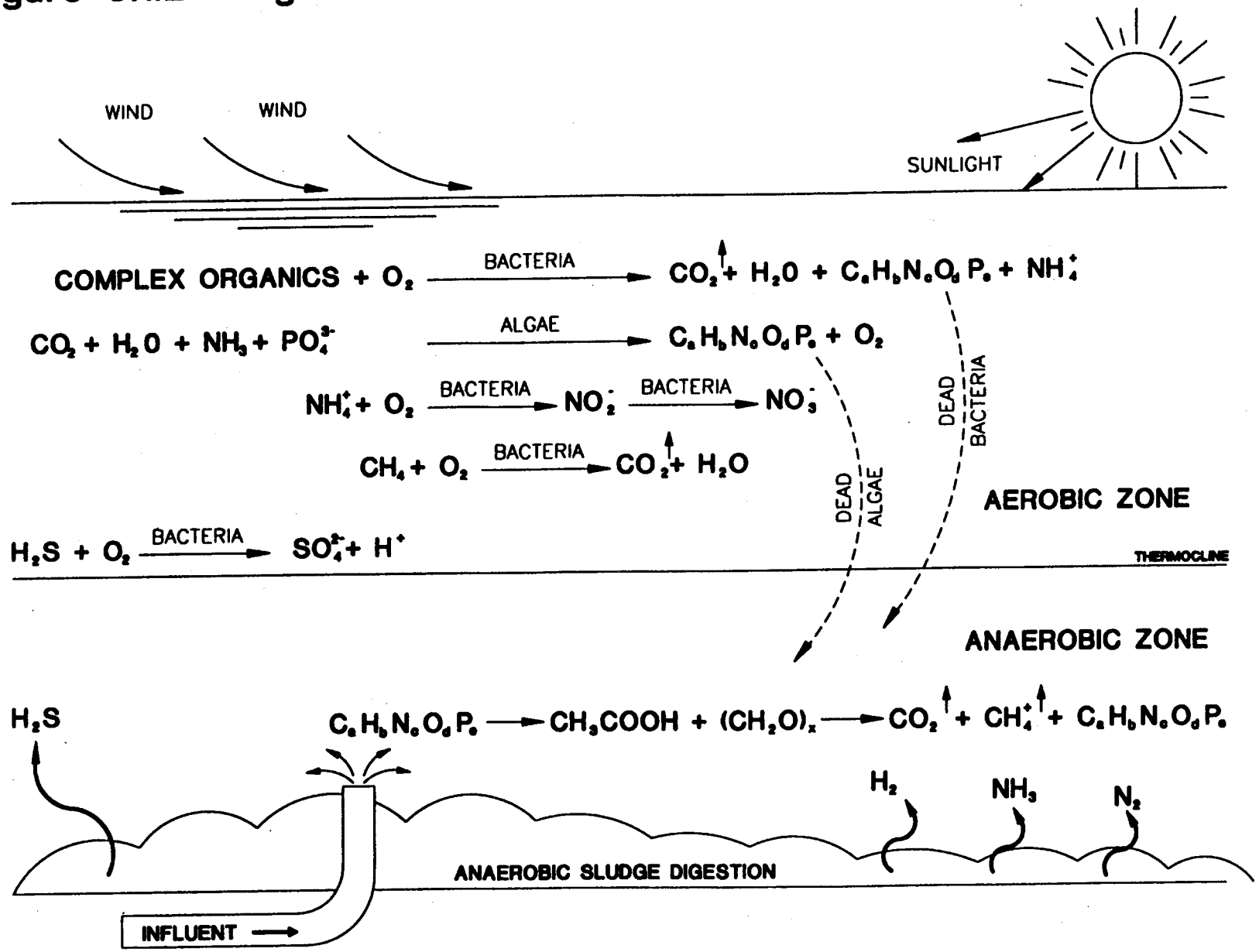
The symbiosis of aerobic and facultative bacteria with algae result in aerobic oxidation of soluble organic matter. Lagoon systems contain different types of algae capable of adjusting their depth as a function of changes in environmental conditions, especially light. Algae may be unicellular or multicellular, motile or nonmotile. Algae utilize inorganic nutrients such as phosphate, carbon dioxide, and nitrogen. Algae are generally divided into three major groups based on colour. Green and brown algae are common to lagoons with the red algae occurring infrequently.

3.1.2 Stoichiometry

The mechanics of organic waste stabilization in a lagoon system can be visualized by the schematic shown in Figure 3.1.2. The suspended and settleable solids, dead bacteria, algae and other organisms settle at the bottom of the lagoon and undergo anaerobic decomposition. Carbon dioxide, methane, nitrogen and hydrogen sulphide as well as anaerobic bacterial cells are formed. The gasses move upward towards the surface where the pH is in the alkaline range. Methane and nitrogen are released into the atmosphere. Due to the alkaline conditions at the top layers of the lagoon hydrogen sulphide and carbon dioxide gases are trapped in the upper layer of the lagoon in the form of carbonates and sulphides.

Algae and bacteria together comprise the essential elements of successful stabilization of organics in the lagoon systems. Bacteria break down the complex organic waste components into simple products which are then available for use by the algae. Algae, in turn, produce the oxygen necessary for maintaining the aerobic environment for the bacteria during the process known as photosynthesis.

Figure 3.1.2 Organic Waste Stabilization in a Lagoon System



I:\1231\08\FIG_312.DWG

3.1.2.1 Carbon Cycle

Bacteria degrade organic matter according to the following simplified equations (Gloyna, 1971, Smith et al, 1983, and Environment Canada, 1985):

· aerobic conditions: $C_aH_bN_cO_dP_e + O_2 \rightarrow C_aH_bN_cO_dP_e + CO_2 + H_2O + NO_3^- + PO_4^{\equiv}$

· anaerobic conditions: $C_aH_bN_cO_dP_e \rightarrow CH_3COOH + C_aH_bN_cO_dP_e$

$CH_3COOH \rightarrow C_aH_bN_cO_dP_e + CO_2 + CH_4$

Algae, in turn, reuse the carbon (as carbon dioxide) to form algal biomass according to the following equation:

$$CO_2 + H_2O + NH_3 + PO_4^{\equiv} \rightarrow C_aH_bN_cO_dP_e + O_2$$

The simplified carbon cycle is shown in Figure 3.1.2.1.

3.1.2.2 Sulphur Cycle

Of particular interest in lagoons is the sulphur cycle due to the potential release of malodorous and toxic hydrogen sulphide. The simplified sulphur cycle is presented in Figure 3.1.2.2. Organic matter containing sulphur is decomposed by microorganisms to hydrogen sulphide which is in turn used by some bacteria (i.e. purple and green sulphur bacteria) as an energy source in the process known as anoxygenic photosynthesis (it occurs in the complete absence of oxygen). As a product, elemental sulphur is formed which is aerobically oxidized to produce sulphate. Sulphate can be further utilized by bacteria to form hydrogen sulphide or by plants, animals, and bacteria for production of organic compounds.

Simplified equations representing sulphur cycle are as follows:

$$C_aH_bN_cO_dP_e + CO_2 + H_2S \rightarrow C_aH_bN_cO_dP_e + H_2 + S$$

$$S + O_2 \rightarrow SO_4^{\equiv}$$

$$CH_3COOH + SO_4^{\equiv} + NH_3 + PO_4^{\equiv} \rightarrow C_aH_bN_cO_dP_e + H_2S + CO_2 + OH^-$$

Figure 3.1.2.1 Carbon Cycle in a Lagoon System

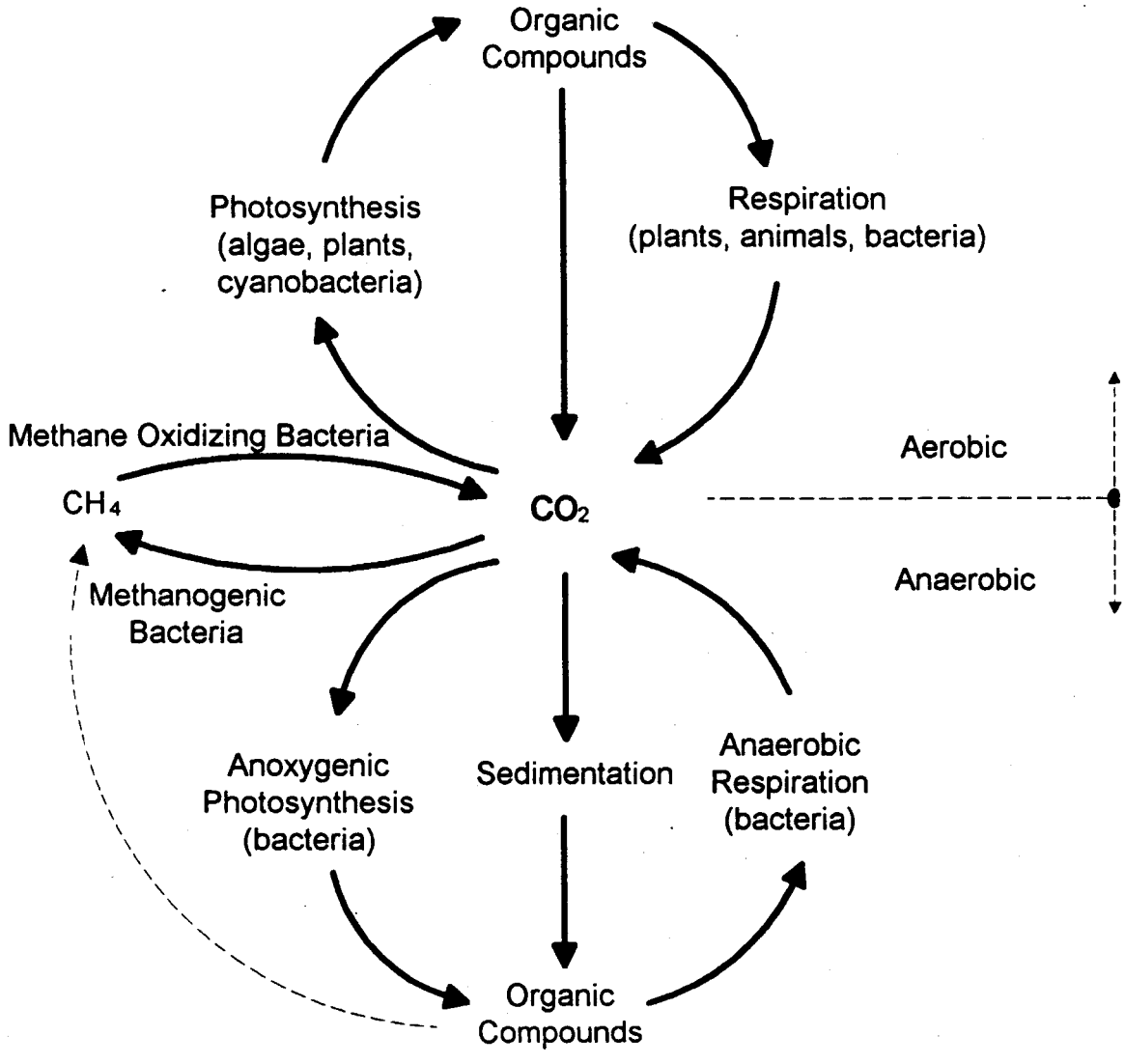
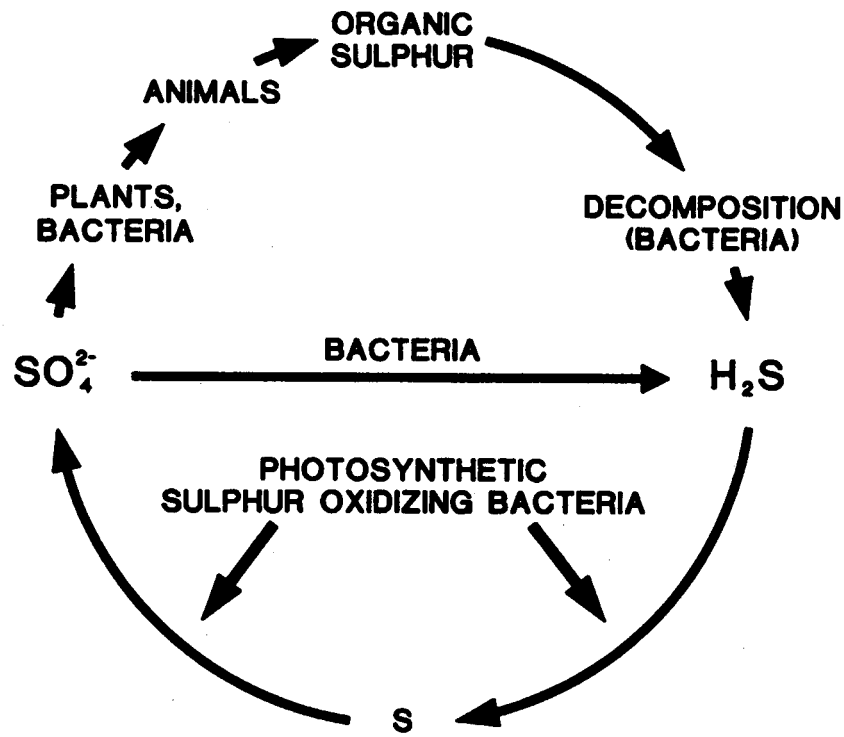


Figure 3.1.2.2 Sulphur Cycle in a Lagoon System



3.1.2.3 Nitrogen Cycle

Organic nitrogen and ammonia nitrogen enter the lagoon with the influent wastewater. Organic nitrogen undergoes conversion to ammonia by microbial activity in a process known as ammonification. The ammonia is present in both ionized and un-ionized forms (the proportion depends on pH and temperature). Ammonia produced from ammonification can be assimilated by organisms as a nutrient to produce cell protein and other nitrogen-containing compounds. Excess ammonia is nitrified to nitrite and nitrate by autotrophic bacteria. The nitrate is utilized in denitrification to form nitrite and then nitrogen gas. A simplified nitrogen cycle is shown in Figure 3.1.2.3. Nitrification can occur in lagoons under aerobic conditions whereas denitrification occurs under anaerobic conditions. Reactions are defined by the following equations:

· nitrification (aerobic):

$$NH_3 + O_2 \rightarrow NO_2^- + H^+ + H_2O$$
$$NO_2^- + O_2 \rightarrow NO_3^-$$

· denitrification (anaerobic)

$$NO_3^- + CH_3OH \rightarrow N_2 + CO_2 + H_2O + OH^-$$

Nitrogen gas can subsequently be "fixed" to form organic nitrogen by cyanobacteria, thus completing the nitrogen cycle.

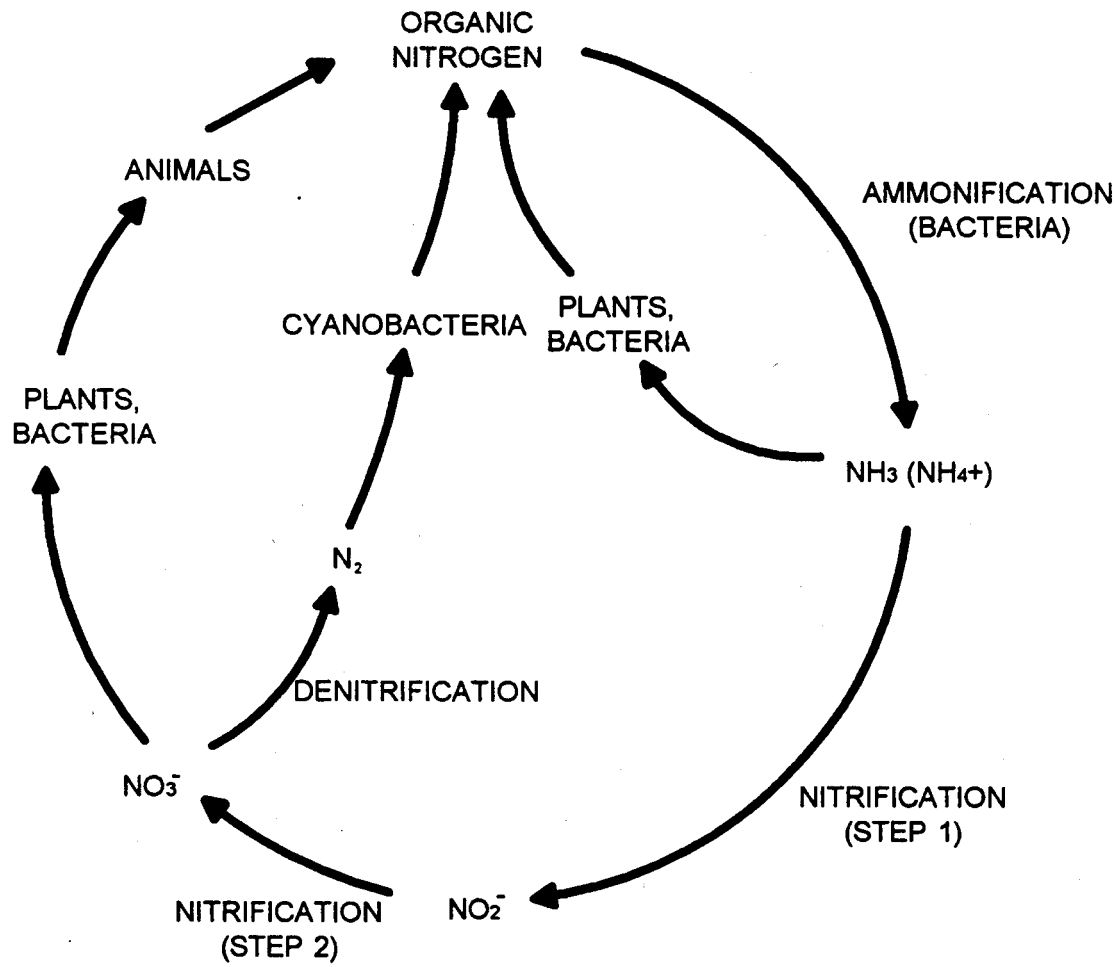
3.1.3 Effect of Temperature

Temperature is a very important factor in the lagoon treatment system. Temperature determines the predominant species of algae, bacteria, and other aquatic organisms at and near the surface.

Algae can survive at temperatures of 5°C to 40°C. However, the most efficient growth of green algae occurs at temperatures near 30°C to 35°C. Aerobic bacteria are viable within the temperature range of 10°C to 40°C, and 35°C is optimum for cyanobacteria and many bacteria.

The major sources of heat in lagoon systems are influent temperature and solar radiation. Generally lagoon liquid temperature decreases with increasing depth. This can influence the anaerobic decomposition at the bottom since anaerobic bacteria prefer temperatures above 15°C.

Figure 3.1.2.3 Nitrogen Cycle in a Lagoon System



Seasonal temperature changes result in vertical water stratification in the lagoons. During the summer, the upper water layers are warmed up and the density decreases, whereas at greater depths the water is colder resulting in higher water density. The result of this is stratification. Temperature changes rapidly in the thin layer known as the thermocline. As temperature decreases during the fall, stratification decreases and, facilitated by wind action, the lagoon becomes completely mixed. This phenomenon is referred to as the fall overturn. The density of water decreases as the temperature falls below 4°C and winter stratification occurs. As ice cover breaks up and the water warms, a spring overturn can also occur.

During both spring and fall overturns significant odours, caused by hydrogen sulphide release from the bottom anaerobic layer, may occur. Release of hydrogen sulphide can also occur after the melting of ice cover. During the ice cover period oxygen production by algae can completely cease and oxygen supply by wind action is also eliminated. This can result in anaerobic conditions in the lagoon and increased production of hydrogen sulphide which is trapped under the ice cover and released after the ice cover melts.

The overall effects of temperature and light intensity are reflected in improved lagoon performance during summer months and decreased efficiency during cold periods. The effects of temperature on biological process reaction rates can be predicted by the following relationship:

$$K_T = K_{20}\Theta^{(T-20)}$$

where K_{20} is known reaction rate at 20°C, and Θ is the temperature coefficient. Values of Θ vary widely depending on the type of process considered.

3.1.4 Sludge Accumulation and Bottom Sediments

The sludge zone contains the settled solids from the water column. This zone is mostly anaerobic and contains a sensitive population of obligate anaerobes which ferment the settled solids. The two groups of microorganisms active in this zone include: 1) acid formers and; 2) methanogens. The first group ferment complex organic compounds and produce a variety of fermentation products while the second group, consisting mostly of methanogens, utilize some of the organic acids produced by acid formers to yield methane. These two types of bacteria are very sensitive to changes in environmental conditions, particulary temperature and their activity is greatly reduced below 10°C.

The accumulation of sludge in lagoons has been investigated by a number of authors. Middlebrooks et al (1982) indicated that a large portion of the accumulated sediments is inorganic material and silt washed into the lagoon. The same authors reported that sludge accumulation rates in Canada and Alaska ranged from 0.25 to 0.40 m^3 per 1000 people per day. Assuming an average per capita flow of 265 L per person per day, this range is equivalent to 0.94 - 1.5 m^3 sludge accumulated per 1000 m^3 sewage, similar to the recommendations made by EPA in Section 4.1.4. In a summary of cold climate sewage lagoon performances (A.R. Townshend and H.Knoll, 1987) sludge accumulation rates ranging from 0.17 to 0.50 m^3 per 1000 people per day were reported.

Sludge accumulation reduces lagoon volume, potentially leading to reduced detention times and shorter operational life.

3.1.5 Ammonia Removal

Ammonia removal in lagoons can occur by gaseous ammonia stripping to the atmosphere, ammonia assimilation in bacterial and algal biomass, biological nitrification followed by nitrate uptake by plants and algae, or by denitrification.

Conversion of organic matter containing nitrogen is carried out by numerous chemo-heterotrophic bacteria in a process known as ammonification, converting the organic nitrogen to ammonia (i.e. NH_3 and NH_4^+). The ratio of NH_3 to NH_4^+ is controlled by lagoon water pH and temperature. As the pH of the water decreases the more NH_4^+ is present, and as the pH increases, the more NH_3 is present. As mentioned above, some of the ammonia is lost due to stripping to the atmosphere. A small portion of the ammonia can be utilized by some microorganisms and plants in production of organic compounds (NH_4^+ is directly added to a pre-existing organic compound). However, most of the ammonia is further oxidized by nitrification.

Nitrification is a two step process in which ammonia is first converted to nitrite by autotrophic bacteria consisting largely of *Nitrosomonas* species and then to nitrate by another group of autotrophic bacteria composed largely of *Nitrobacter* species. Nitrate is assimilated by algae, plants and microorganisms for production of organic compounds needed for reproduction and growth. At the end of a life cycle when these organisms die, organic nitrogen is returned to the cycle by decomposition of organic matter.

Some of the nitrate can undergo anaerobic denitrification and is converted to nitrogen gas. Nitrogen gas can escape to the atmosphere or it can be utilized by cyanobacteria to form organic matter.

If the lagoon effluent contains significant concentrations of algae and bacteria when released to the receiving water, nitrogen bound as organic matter will be eventually released through endogenous respiration. This may cause accelerated eutrophication of surface waters. The same can be expected if nitrate is released to surface waters since nitrate is a readily available source of nitrogen for algae. Abundant development and decomposition of algae may cause dissolved oxygen depletion. If the nitrate is released to groundwater can also adversely affect the use of the groundwater as a source of potable water.

3.1.6 Hydrogen Sulphide Generation and Removal

The rate of hydrogen sulphide removal in a lagoon system is controlled by anoxygenic photosynthetic bacteria (i.e. purple and green sulphur bacteria) which produce elemental sulphur as an end product. Hydrogen sulphide rises to the upper layers of the water column where, generally, it is trapped due to the alkaline conditions. However, during the spring and fall overturns (see Section 3.1.3) a significant amount of hydrogen sulphide can be released to the atmosphere causing odour problems. A significant release of hydrogen sulphide may also occur after melting of the ice cover due to prevailing anaerobic conditions under the ice cover and increased production of hydrogen sulphide.

The primary removal mechanism for hydrogen sulphide gas is through release to the atmosphere at the surface of the lagoon. Consequently, conditions which trap the gas or result in an increase in concentration in the water column (alkaline conditions) will contribute to effluent toxicity.

3.1.7 Types Of Lagoon Systems

Two of the most commonly used lagoon systems are:

1. facultative lagoons, and
2. aerated lagoons (partial mix aerated lagoons).

A facultative lagoon system may be comprised of anaerobic fermentation cells, facultative treatment cells and/or facultative/storage cells. Anaerobic fermentation cells provide pre-treatment by sedimentation and fermentation while facultative treatment cells provide the main secondary treatment. A facultative/storage cell provision is optional and depends on winter climatic conditions and the receiving environment. During severe winter conditions, ice cover eliminates surface aeration and subsequent snow cover reduces the production of photosynthetic oxygen to zero, thereby creating anaerobic conditions throughout the water column. The facultative/storage cell thus provides

storage and sedimentation in the winter and allows for effective treatment once the ice cover melts and the liquid temperature increases.

Aerated lagoons, on the other hand, are composed of a series of aerated cells generally followed by a quiescent zone or separate polishing cell which allows settling of suspended solids.

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3.2 EVALUATION OF EXISTING LAGOON SYSTEMS

The performance of a number of facultative and aerated lagoon systems with regard to fish toxicity was evaluated for this report. The following subsections present the performance data for both facultative and aerated lagoons in terms of effluent ammonia, BOD_5, hydrogen sulphide levels, and bioassay test results.

3.2.1 Facultative Lagoons

Data for several facultative lagoons collected from BC Environment files, EPS reports, Ontario Ministry of Environment files and reports, and the authors' files were examined. The physical characteristics of these lagoon systems are presented in Table 3.2.1.1.

3.2.1.1 Ammonia

Facultative lagoon effluent ammonia concentrations, and toxicity data (where available), are presented in Table 3.2.1.2 and graphically in Figures 3.2.1.1 through 3.2.1.6. Facultative lagoons generally produce a non-toxic effluent during the summer months; effluent toxicity is more common during the cooler winter months. Ammonia levels for the McBride and Vanderhoof lagoons illustrate this year-round cycle: low levels in the summer and fall, rising levels in the winter, high levels in spring, and decreasing levels into the summer. This springtime correlation is also clearly illustrated by the Tavistock, Mitchell, Markdale and Wingham lagoons. The high ammonia levels during the spring is attributed to poor wastewater treatment due to colder temperatures, and possible ice cover during the winter months. The spring discharges reflect the flushing-out of untreated ammonia accumulated during the winter.

Examination of the data suggests that lagoon effluents with total ammonia concentrations less than 11 mg/L-N and/or un-ionized ammonia concentrations less than 1.0 mg/L-N, are generally not acutely toxic. However, there are a number of data points which illustrate that these values cannot be used as effluent criteria limits. Effluents have been reported to be non-toxic even when one or both of the values are exceeded and, similarly, they have been reported to be toxic even when the total and un-ionized ammonia levels are below the values noted above.

Table 3.2.1.1 Facultative Lagoons Physical Characteristics

Lagoon	Description of System	Surface Area (ha)	Depth (m)	Total Retention Time (d)
Hagwilget, BC	two facultative lagoons in series	0.3 0.35	2.3 2.3	279
Kamloops. BC	three facultative lagoons in series	9.8 16.7 37.6	1.83 3.35 3.35	max 120
Markdale, Ont.	single facultative lagoon	5.7	-	-
McBride, BC	single facultative lagoon	2	1.5	92
Mitchell, Ont.	two facultative cells in parallel, both discharge to a common third cell	8.5 8.5 10.3	- - -	-
Moricetown Band, BC	single facultative lagoon	0.4	1.65	117
Strathroy, Ont.	single facultative lagoon	15	1.52	86-110
Tavistock, Ont.	two facultative lagoons in parallel	6.9 6.1	- -	-
Vanderhoof, BC	two facultative lagoons in series	4.6 4.6	1.5 1.5	90
Whitehorse, Yukon	four anaerobic lagoons series/parallel	3.7	6.1	18
Porter Creek, Yukon	two anaerobic lagoons series/parallel	0.5	4.6	6
Wingham, Ont.	two facultative lagoons in parallel	6.1 6.1	- -	-

Table 3.2.1.2 Annual Effluent Ammonia and Toxicity of Facultative Lagoons

	McBride, BC (1991-1992)				Vanderhoof, BC (1991-1992)				Wingham, Ontario (1977-1978)		Markdale, Ontario (1977-1978)		Mitchell, Ontario (1977-1978)		Tavistock, Ontario (1977-1978)	
Month	pH	Total Ammonia mg/L-N	NH3 mg/L-N	% Mort.	pH	Total Ammonia mg/L-N	NH3 mg/L-N	% Mort.	Total Ammonia mg/L-N	24 hour % Mort.	Total Ammonia mg/L-N	24 hour % Mort.	Total Ammonia mg/L-N	24 hour % Mort.	Total Ammonia mg/L-N	24 hour % Mort.
Jan																
Feb																
Mar																
Apr	8.2	7.19	0.295	100	7.8	14	0.234	100								
May	7.9	4.6	0.096	0	8.3	17	0.868	100								
Jun	8.2	0.366	0.015	0	9	3.67	0.779	100								
Jul	8.2	0.564	0.023	0	8.6	7.34	0.712	0								
Aug	8.2	0.561	0.023	10	8.6	7.41	0.718	10	4*	0	4*	0				
Sep	8.1	1.56	0.051	0	8.6	15.5	1.503	0	4*	0	4*	0				
Oct	8	5.91	0.155	0	8.5	5.85	0.460	0	4*	0	4*	0				
Nov	8.1	6.41	0.210	0	8.5	11.6	0.911	0								
Dec	8.2	0.024	0.001	0	8.2	17.7	0.725	100	3	0	10.5	0	1.2	0	5.5	0
Jan	8.3	14.8	0.756	90	8.4	20.4	1.294	100	6	0	12	0	5.5	0	17	100
									8.5	0	13.5	90				
Feb	7.5	9.35	0.079	0	8.3	19.5	0.996	100	9.5	20	16	100	6	100	16	100
									12	60						
Mar	8.2	10.9	0.447	0	8.3	16.7	0.853	100	13	60	21	100	11.2	100	9	100
									16	90						
Apr	7.9	7.36	0.154	0	8.3	19.2	0.960	100	9.5	60	16	100	8.5	0		
									5.5	0	14	80				
May	8.1	5.16	0.169	0	8.4	16.9	1.072	100	4.2	40	12	60			11	0
Jun	8	3.24	0.085	0	8.5	7.84	0.616	30								

Taken from Metikosh et al. 1981, Shepherd et al. 1982, Ontario MoE 1990, Rupke 1977, BC Ministry of Environment files and NovaTec Files

% Mort. : percentage mortalities
* : denotes average value

If blank, information not available.

Toxicity tests conducted for 96 hours except where noted.
NH3 = un-ionized ammonia, calculated as a percentage of total ammonia using equation by Emerson et al.:
Fraction of total ammonia in unionized form: 1/(1+10^(pKa-pH))
Where pKa = 0.09018 + 2729.92/(273+T), T in degrees Celcius
T = 15 for all samples (laboratory water temperature)

H:\1231\06\CHP3\CHP32\FACLAGN.WB1_B

Table 3.2.1.2 Annual Effluent Ammonia and Toxicity of Facultative Lagoons (con't)

	Kamloops, BC (1981)				Strathroy, ON (1976)	Moricetown Band, BC (1993)				Hagwilget, BC (1993)				Whitehorse, Yukon (1989)				Porter Creek, Yukon (1989)			
Month	pH	Total Ammonia mg/L-N	NH3 mg/L-N	% Mort.	Total Ammonia mg/L-N	pH	Total Ammonia mg/L-N	NH3 mg/L-N	% Mort.	pH	Total Ammonia mg/L-N	NH3 mg/L-N	% Mort.	pH	Total Ammonia mg/L-N	NH3 mg/L-N	% Mort.	pH	Total Ammonia mg/L-N	NH3 mg/L-N	% Mort.
Jan																					
Feb					7																
Mar					11																
Apr					8																
May					3.4					7.1	16.2	0.05	30	8.2	13.7	0.56	40				
Jun					3.3																
Jul					4.2																
Aug					4.3																
Sep																					
Oct														7.3	8	0.04	100	7.2	13.4	0.08	100
Nov	7.7	23.5	0.313	0		7.4	20.2	0.14	100												
Dec																					
Jan																					
Feb																					
Mar																					
Apr																					
May																					
Jun																					

Taken from Metikosh et al. 1981, Shepherd et al. 1982, Ontario MoE 1990, Rupke 1977, BC Ministry of Environment files, NovaTec files

% Mort. : percentage mortalities
* : denotes average value

If blank, information not available.

Toxicity tests conducted for 96 hours except where noted.
NH3 = un-ionized ammonia, calculated as a percentage of total ammonia using equation by Emerson et al.:
Fraction of total a Fraction of total ammonia in unionized form: 1/(1+10^(pKa-pH))
Where pKa = 0.0 Where pKa = 0.09018 + 2729.92/(273+T), T in degrees Celcius
T = 15 for all sam T = 15 for all samples (laboratory water temperature)

H:\123108\CHP3\CHP32\FACLAGN.WB1_B

Figure 3.2.1.1

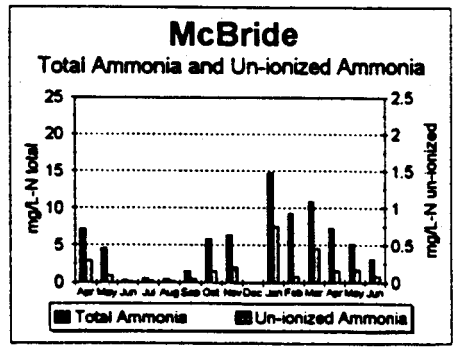
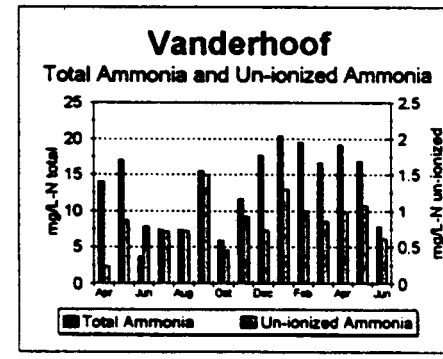
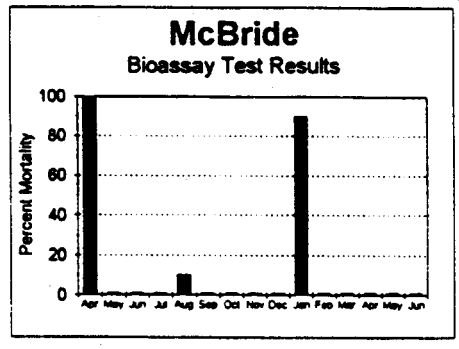


Figure 3.2.1.2





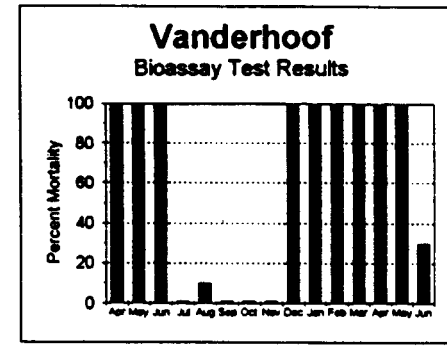
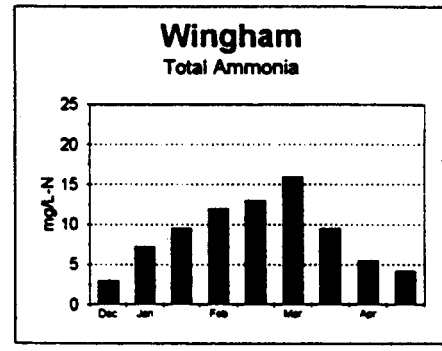
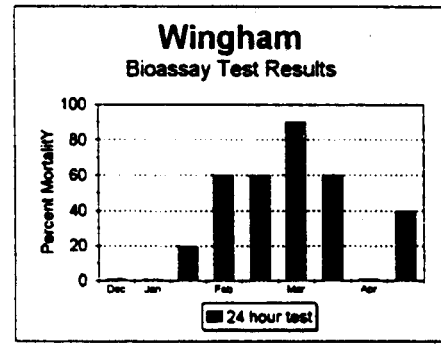


Figure 3.2.1.3





Percent mortality incremental is 10% (total of 10 fish per test)
For Wingham, two data points are shown for some months.

Figure 3.2.1.4

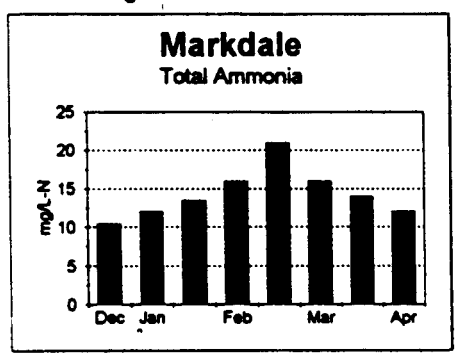
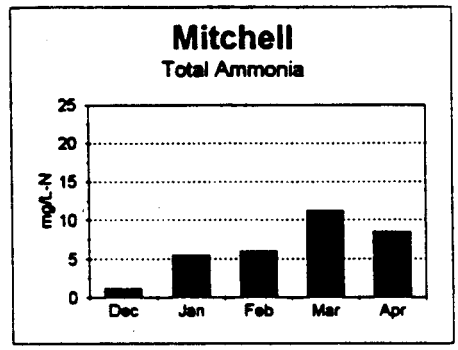
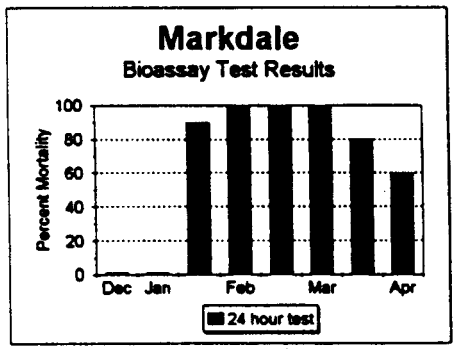


Figure 3.2.1.5





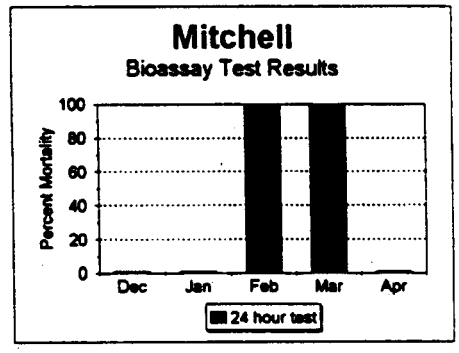
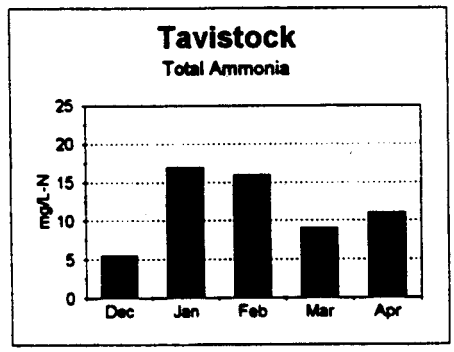
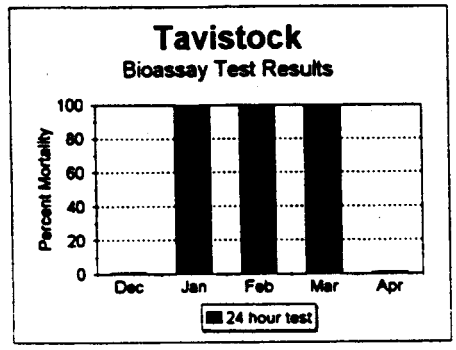


Figure 3.2.1.6





Percent mortality incremental is 10% (total of 10 fish per test)
For Markdale, two data points are shown for some months.

An example of the toxicity of un-ionized ammonia alone is found in Vanderhoof lagoon's June effluent with a total ammonia concentration of 3.67 mg/L-N and un-ionized ammonia concentration of 0.779 mg/L-N resulting in 100% mortalities. At the opposite extreme, the Moricetown Band lagoon's November effluent had a total ammonia concentration of 20.2 mg/L-N and an un-ionized ammonia concentration of 0.14 mg/L-N, also resulting in 100% mortalities.

An exception to the toxicity limits defined above occurred in April for the McBride lagoon (total ammonia 7.19 mg/L-N, un-ionized ammonia 0.295 mg/L-N) resulting in 100% mortality. Two noticeable anomalies at the other end of the range are Vanderhoof's September effluent data with a total and un-ionized ammonia concentration of 15.5 and 1.503 mg/L-N, respectively, and the Kamloops lagoons November effluent with 22.5-23.5 mg/L-N total ammonia and 0.313 mg/L-N un-ionized ammonia, neither of which resulted in any mortalities. The wide range in ionized and un-ionized ammonia concentrations resulting in toxicity may be explained by the presence of other toxic contaminants not analyzed for, or by the synergistic effects of toxic components.

The recorded LC_{50} values and ammonia concentrations for Whitehorse and Porter Creek lagoon systems are given in Table 3.2.1.3. This data illustrates that effluents with total ammonia concentrations less than 11.0 mg/L-N (previously noted limit) are not consistently non-toxic.

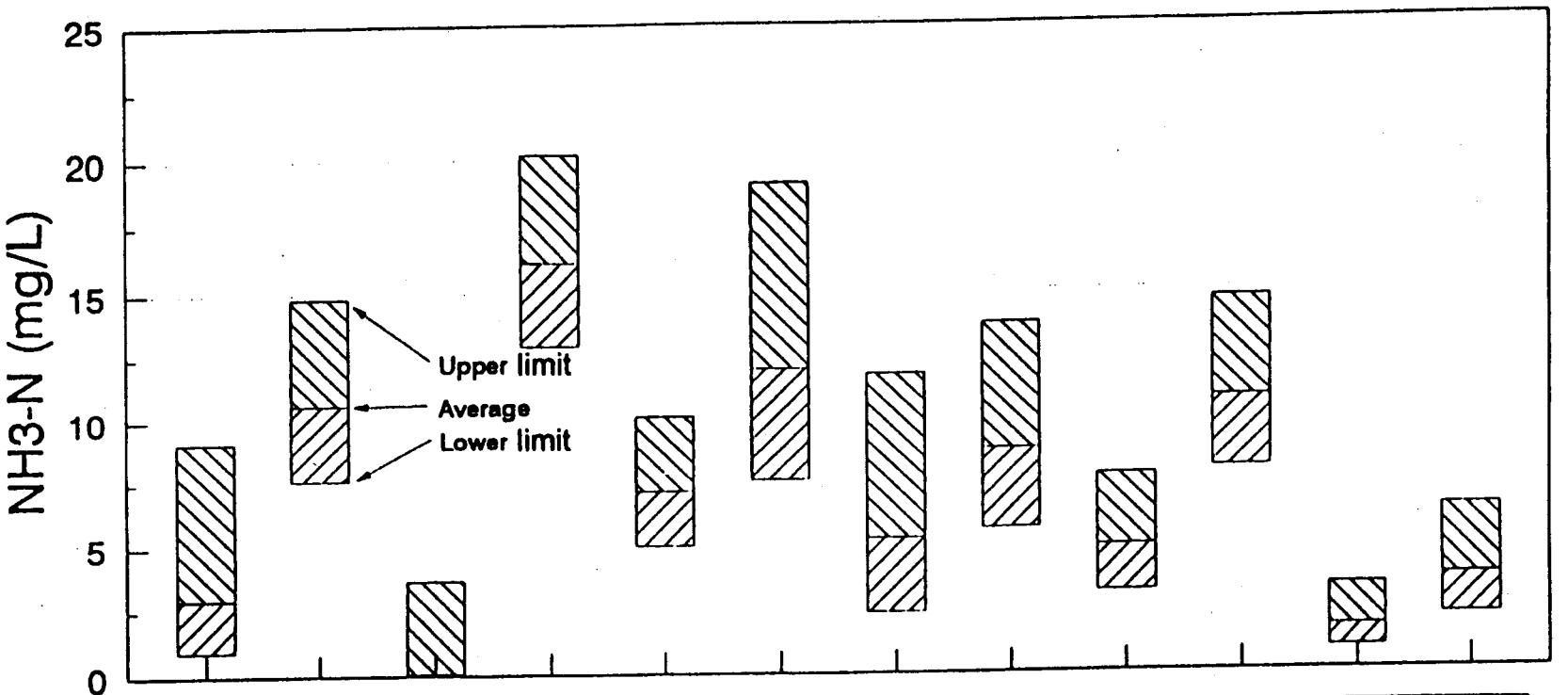
Table 3.2.1.3 Whitehorse and Porter Creek Lagoon Systems LC_{50} Values and Ammonia Concentrations

Date	System	LC_{50} value	Ammonia (mg/L-N)
March 1985	WH	67.4%	9.2
May 1986	WH	100.0%	6.8
September 1986	WH	100.0%	8.6
May 1989	PC	40.0%	10.9
May 1989	WH	100.0%	13.7
October 1989	WH	79.0%	8.0
October 1989	PC	46.0%	13.4

Taken from NovaTec files. WH: Whitehorse; PC: Porter Creek

Figure 3.2.1.7 illustrates a summary of research carried out at the University of Alberta (Smith, 1993) on effluent ammonia concentrations for spring and fall conditions for various lagoon configurations.

Figure 3.2.1.7 NH3-N Effluent Concentrations for Various Treatment Technologies



	A. Sludge	Aerated Lagoon	Ext. Aeration	Lagoon 0S-1L SPR.	Lagoon 0S-1L FALL	Lagoon 0S-2L SPR.	Lagoon 0S-2L FALL	Lagoon 2S-2L SPR.	Lagoon 2S-2L FALL	Lagoon 4S-2L SPR.	Lagoon 4S-2L FALL	RBC
Upper 95%	9.12	14.80	3.67	20.15	10.08	19.06	11.73	13.68	7.61	14.61	3.24	6.26
Average	2.95	10.61	0.06	16.11	7.06	11.96	5.17	8.70	4.82	10.72	1.63	3.56
Lower 95%	0.95	7.60	0.00	12.88	4.95	7.50	2.28	5.53	3.05	7.87	0.82	2.03
# of Sites	9	25	10	11	11	9	5	16	15	22	23	10

(Note: 4S-2L FALL denotes four short detention cells and two long detention cells with a fall discharge). Of the facultative lagoons, the 4S-2L configuration yielded the lowest overall (fall and spring) ammonia concentrations. The year-round effluent for the same lagoon configuration is presented in Figure 3.2.1.8. Note that the upper 95 percentile effluent ammonia concentration during the month of April was in excess of 18 mg/L-N.

3.2.1.2 BOD_5 Removal

A review of the data from Tavistock, Markdale, Mitchell and Wingham lagoons was done by the Ontario Ministry of Environment. It was found that in cases where effluent total ammonia concentrations exceeded 10 mg/L-N, and BOD_5 removal efficiencies were greater than 80%, effluents were rarely found to be acutely toxic. With the same ammonia concentration but less than 80% BOD_5 removal, effluents were almost consistently acutely toxic. In cases where effluent ammonia concentrations were less than 10 mg/L-N and BOD_5 removal was less than 80%, half of the samples were acutely toxic. With BOD_5 removals greater than 80% and the same ammonia concentration, almost all of the samples were acutely non-toxic. The conclusion formed was that effluent toxicity due to ammonia increased with increased concentrations of BOD_5. The data from this report is summarized in Table 3.2.1.4.

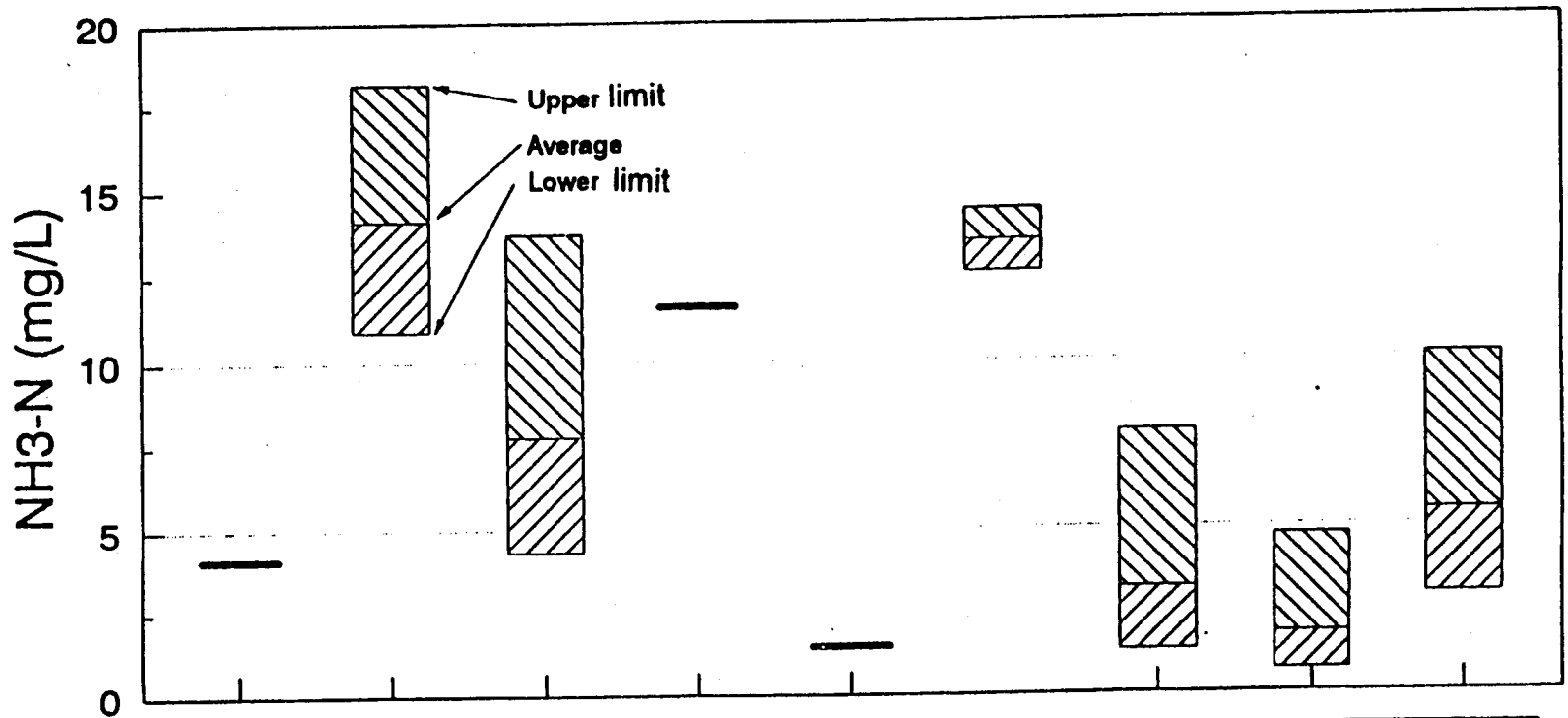
3.2.1.3 Hydrogen Sulphide

After a 34 hour discharge from Mitchell lagoon in February, low dissolved oxygen and high H_2S concentrations in the receiving environment were concurrent with observed fish kills (Slaughter et al., 1978). A toxicity test completed on the undiluted effluent that same month resulted in 100% mortality. As the total ammonia concentration was only 6.0 mg/L-N, hydrogen sulphide would appear responsible or to have contributed to the observed toxicity in this case.

3.2.1.4 Short Circuiting

The Vanderhoof lagoon (BC) and McBride lagoon (BC) are both facultative systems with similar retention times (90 and 92 days respectively) and operating depths (both 1.5 m) performing under approximately the same climatic conditions. Sketches of the lagoons are given in Figure 3.2.1.9.

Figure 3.2.1.8 NH3-N Effluent Concentrations of 4S-2L Lagoons for Various Months of the Year



	Mar.	Apr.	May	Jun.	Jul.	Aug	Sep.	Oct.	Nov.
Upper 95%		18.18	13.58			14.37	7.86	4.72	10.06
Average	4.10	14.08	7.66	11.60	1.43	13.43	3.16	1.76	5.41
Lower 95%		10.90	4.32			12.55	1.27	0.65	2.91
# of Sites	1	15	11	1	1	2	6	15	8

Table 3.2.1.4 Categorization and Toxicity of Lagoon Effluents on Process Efficiency and Industrial Flow Contribution Taken From Ontario Ministry of Environment, 1990

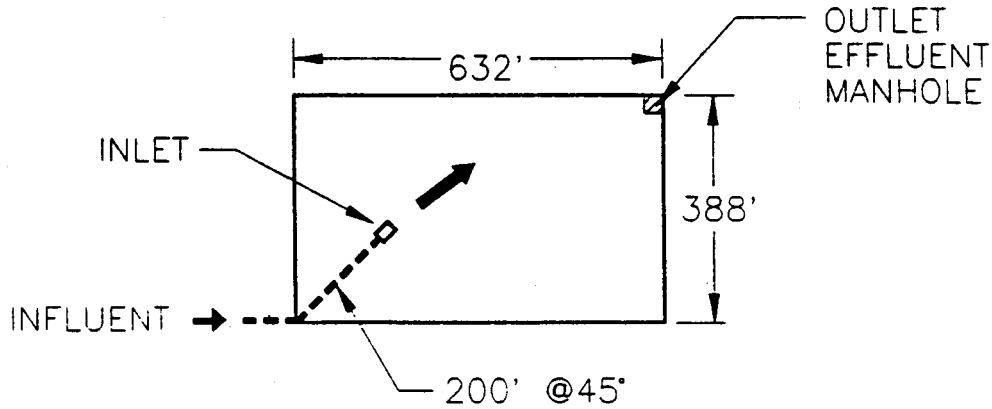
Effluent Total Ammonia-N (mg/L)	BOD5 Removal Efficiency (%)	Domestic Input	Month.Year	Major Industrial Input	Month.Year
NH3 > 10	> 80%	Markdale	12.77	Listowel	02.78.01
			01.78.01		02.78.02
			01.78.02 **		03.78.02
			05.78 **		04.78
		Tavistock	05.78		
	< 80%	Markdale	01.78.02	Listowel	03.78.01
			01.78.03 **		
			02.78 **		
			03.78.02 **		
			04.78.01 **		
		Wingham	02.78.02 **		
			03.78.01 **		
			03.78.02 **		
		Mitchell	03.78 **		
		Tavistock	01.78 **		
			02.78 **		
NH3 < 10	> 80%	Wingham	01.78.01	Listowel	12.77
			01.78.02		01.78.01
			02.78.01 *		01.78.02
			05.78 *		
		Mitchell	12.77		
			01.78		
		Tavistock	12.77		
	< 80%	Wingham	12.77		
			04.78.01 **		
			04.78.02		
		Mitchell	02.78 **	-	-
			04.78		
		Tavistock	03.78 **		

* acute mortality in >0 to 49% of samples.

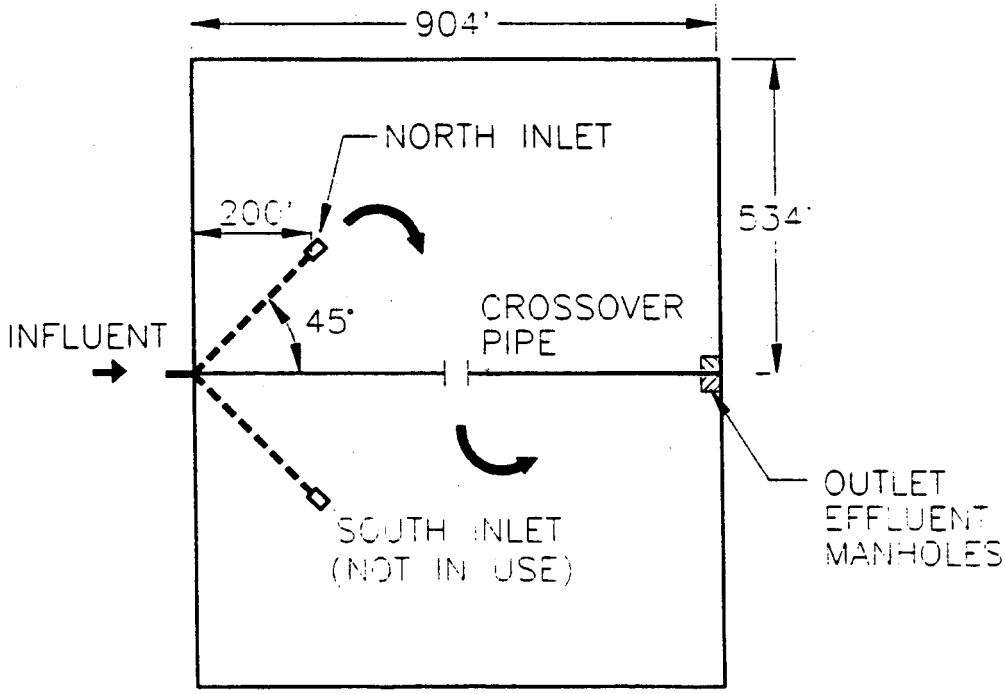
** acute mortality in >50% of samples.

Figure 3.2.1.9 McBride and Vanderhoof Lagoon Inlet / Outlet Configuration

McBRIDE



VANDERHOOF



The McBride lagoon system consists of a single stabilization cell, whereas the Vanderhoof lagoon system consists of two cells operating in series. Multicell lagoon systems operating in series should outperform single cell lagoon systems. The bioassay results collected from both these lagoons, however, indicates the contrary as illustrated in Figures 3.2.1.1 and 3.2.1.2. Overall, the Vanderhoof effluent has a higher ammonia concentration and is toxic for a longer period of the year.

An examination of the relative locations of the inlet and outlet structures, and the likely flow patterns within the ponds provides a possible explanation for the difference in performance. The inlet of the McBride lagoon is opposite from the outlet, ensuring that the majority of the lagoon volume is utilized in treatment. In comparison, the Vanderhoof lagoon inlet is relatively close to the cross connecting pipe which in turn is relatively close to the outlet, creating the potential for short circuiting. Short circuiting would not maximize use of the lagoon volume thereby reducing the 90 day design retention time.

It is clear from this example that lagoon configuration is extremely important in attaining design retention times, maximizing wastewater treatment and minimizing effluent toxicity.

3.2.2 Aerated Lagoons

Recorded data for several aerated lagoon systems treating predominantly municipal wastewater was collected for this report. Data was retrieved from BC Environment files, Ontario Ministry of Environment files, EPS reports, and NovaTec's own records. The physical characteristics of these systems are given in Table 3.2.2.1.

Table 3.2.2.1 Aerated Lagoons Physical Characteristics

Lagoon	Description of System	Surface Area (ha)	Depth (m)	Retention Time (d)
Blackburn (Prince George), BC	two aerated lagoons in series	0.5 1.3	3.7 3.7	8.5 25
Fraser Lake, BC	two aerated lagoons in series	1.2 1.2	2.4 2.4	78 80
Listowel, Ontario	one aerated cell in series with two facultative cells	0.6 17.2, 10.8		
Tumbler Ridge, BC	single aerated lagoon followed by a stabilization pond	0.5 2.6	6.0 2.6	126 10
Anahim, BC	single aerated lagoon followed by a facultative lagoon	0.31 0.31	2 2	200
Blueberry, BC	single aerated lagoon	0.064	2	
Cape Mudge, BC	single aerated lagoon	0.24	3.66 (max)	131 (winter flows)
Mt. Currie, BC	two aerated lagoons in series	0.36	3.65 (max)	131 (winter flows)
Lake Cowichan, BC	two aerated lagoons in series	0.26 0.785	3.1 3.1	7.7 23
District of Houston, BC	two aerated lagoons in series followed by an exfiltration basin	0.48 0.48 1.4	1.4 1.4 0.9	14
City of Terrace, BC	two aerated lagoons in series	1.6	4.5	24
Williams Lake, BC (sizes as per Russ Lewis, City of Williams Lake)	two anaerobic cells aerated lagoon polishing cell	0.6 (each) 2.0 1.10	4.6 4.6 4.6	7 23 13
Clinton, BC	two anaerobic lagoons in parallel followed in series by a facultative lagoon and an aerated lagoon	- - -	- - -	10 43 84
Aldergrove, BC	single aerated lagoon followed by a stabilization pond	0.5 2.6	6.0 2.6	126 10

Table 3.2.2.2 presents aerated lagoons effluent ammonia concentrations and toxicity data (where available). These values are illustrated in Figures 3.2.2.1 through 3.2.2.4. Aerated lagoons generally produce a non-toxic effluent through the summer, fall and early winter months (June-January). Fish mortality was observed in some lagoon effluents in late winter and spring (January-June).

3.2.2.1 Ammonia

As stated earlier, the toxicity of the treated effluent seems related to ammonia concentrations; higher mortalities correspond to higher ammonia concentrations. In most cases reviewed for aerated lagoons where total ammonia concentrations exceed 11 mg/L-N and/or un-ionized ammonia concentrations were greater than 1.0 mg/L-N, at least 50% mortality occurred. Where neither limit was exceeded, mortalities were usually less than 10%. In most failed bioassays, both limits were exceeded, making it again difficult to determine which component plays a more significant role in toxicity. Cases where total ammonia concentration alone accounts for toxicity can be found in the District of Houston data where a total ammonia concentration of 14.7 mg/L-N, accompanied by an un-ionized ammonia concentration of 0.03 mg/L-N resulted in 100% mortalities. There are no cases where an unionized ammonia concentration greater than 1.0 mg/L-N is not accompanied by a total ammonia level of less than 11 mg/L-N. This does not indicate however that un-ionized ammonia is not an important toxicity component. The partitioning of total ammonia between un-ionized and ionized forms, as determined by pH and temperature, plays a key role in effluent toxicity.

Two notable anomalies to these generalizations are to be found in Fraser Lake's February effluent sample with a total ammonia of 17.1 mg/L-N an un-ionized ammonia 0.7 mg/L-N and 0% mortality, and the District of Houston's December effluent sample with a total ammonia concentration of 3.8 mg/L-N, an un-ionized ammonia concentration of 0.12 mg/L-N and 100% mortality. As previously discussed in Section 3.2.1, other factors contributing to toxicity other than ammonia may have resulted in the failed bioassays.

Figure 3.2.1.7 also illustrates a summary of the research of aerated lagoon effluent ammonia concentrations carried out at the University of Alberta (Smith, 1993). The range of effluent concentrations (based on 25 aerated lagoons) results in a yearly average lagoon effluent total ammonia concentration greater than 10 mg/L-N.

Table 3.2.2.2 Annual Effluent Ammonia and Toxicity of Aerated Lagoons

	Blackburn (Prince George), BC (1991-1992)				Fraser Lake, BC (1991-1992)				Listowel, Ont. (1977-1978)		Tumbler Ridge, BC (1992-1993)	Anahim, BC (1993)		Blueberry, BC (1994)		Cape Mudge, BC (1994)		Mt Currie, BC (1993)	
Month	pH	Total Ammonia mg/L-N	NH3 mg/L-N	% Mort.	pH	Total Ammonia mg/L-N	NH3 mg/L-N	% Mort.	Total Ammonia mg/L-N	24 hour % Mort.	Total Ammonia mg/L-N	Total Ammonia mg/L-N	% Mort.	Total Ammonia mg/L-N	% Mort.	Total Ammonia mg/L-N	% Mort.	Total Ammonia mg/L-N	% Mort.
Jan																			
Feb											16*								
Mar														12.5	100	11.3	70		
Apr	8.5	9.22	0.723	10	8.3	10.5	0.535	10			15.2*								
May	8.4	8.21	0.520	0	8.3	2.81	0.143	0											
Jun	8.5	9.23	0.724	0	8.2	0.99	0.041	0			1.6*								
Jul	8.6	0.006	0.001	0	8.6	0.038	0.003	0										0.23	0
Aug	8.4	0.127	0.008	0	8.6	0.026	0.003	0	3*	0	2.3*								
Sep		0.701		0	8.6	0.005	0.000	0	3*	0	0.4	2.9	60						
Oct	8.3	2.88	0.147	0	8	1.36	0.036	0	3*	0									
Nov	8.6	8.46	0.819	0	8.1	0.016	0.001	0			3.7*								
Dec	8.5	3.45	0.271	0	7.8	9.88	0.165	0	7.5	0									
Jan	8.7	12.9	1.533	100	8.2	15	0.614	0	8.7*	0									
Feb	8.6	13	1.258	100	8.2	17.1	0.700	0	10.5*	0	17.6								
Mar	8.6	13.6	1.316	100	8.3	18	0.917	0	12*	0									
Apr	8.2	12.3	0.503	50	8.7	11.4	1.355	80			14.7								
May	8.4	9.04	0.572	0	8.5	0.251	0.020	10											
Jun	8.6	11.6	1.123	90	8.2	3.82	0.156	0			0.8								
Jul											3.2								
Aug																			
Sep																			
Oct											3.6								
Nov																			
Dec																			
Jan																			
Feb																			
Mar																			

Taken from BC Ministry of Environment files, Ontario Ministry of Environment 1990, NovaTec files, Ferguson et al. 1977, Metikosh et al. 1980, Higgs 1977.

% Mort. : percentage mortalities
* : average value
** : chlorinated effluent
If blank, information not available.

All toxicity tests conducted for 96 hours except where noted.
NH3 = un-ionized ammonia, calculated as a percentage of total ammonia using equation by Emerson et al.:
Fraction of total ammonia in unionized form: 1/(1+10^(pKa-pH))
where pKa = 0.09018+2729.92/(273+T), T in degrees Celcius
T = 15 for all samples (laboratory water temperature)

Table 3.2.2.2 Annual Effluent Ammonia and Toxicity of Aerated Lagoons (con't)

	Lake Cowichan, BC (1978)				Aldergrove Lagoon, BC (1990)				District of Houston, BC (1992-1994)				City of Terrace, BC (1993-1994)				Williams Lake, BC (1978)		Clinton, BC (1978)	
Month	pH	Total Ammonia mg/L-N	NH3 mg/L-N	% Mort.	pH	Total Ammonia mg/L-N	NH3 mg/L-N	% Mort.	pH	Total Ammonia mg/L-N	NH3 mg/L-N	% Mort.	pH	Total Ammonia mg/L-N	NH3 mg/L-N	% Mort.	Total Ammonia mg/L-N	% Mort.	Total Ammonia mg/L-N	% Mort.
Jan																				
Feb	7.7**	15.9	0.000	0	8.4	28.3	1.79	100												
Mar													7.4	21.3	0.14	100				
Apr									8.3	9.68	0.49	20								
May									8.0	6.02	0.16	10								
Jun																				
Jul																	0.15	0	1.65	0
Aug																				
Sep																				
Oct																				
Nov																				
Dec									8.0	11.9	0.31	10								
Jan																				
Feb																				
Mar									8.2	11.9	0.49	60	8	19.3	0.51	100				
Apr									8.1			10								
May									8.2	11.3	0.46	0								
Jun									8.2	11.3	0.46	0								
Jul									8.0	2.78	0.07	0								
Aug									8.0	5.59	0.15	0								
Sep									8.4	9.7	0.61	10								
Oct																				
Nov																				
Dec									8.1	3.8	0.12	100								
Jan									6.9	14.7	0.03	100								
Feb									7.4	14.5	0.10	100								
Mar																				

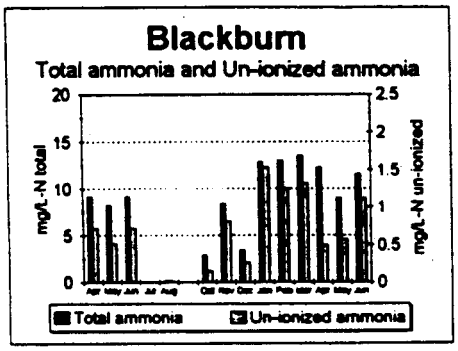
Taken from BC Ministry of Environment files, Ontario Ministry of Environment 1990, NovaTec files, Ferguson et al. 1977, Metikosh et al. 1980, Higgs 1977.

% Mort. : percentage mortalities
* : average value
** : chlorinated effluent
If blank, information not available.

All toxicity tests conducted for 96 hours except where noted.
NH3 = un-ionized ammonia, calculated as a percentage of total ammonia using equation by Emerson et al.:
Fraction of total ammonia in unionized form: 1/(1+10^(pKa-pH))
where pKa = 0.09018+2729.92/(273+T), T in degrees Celcius

Aerated Lagoons

Figure 3.2.2.1



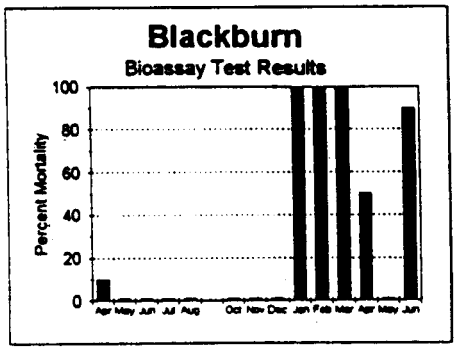
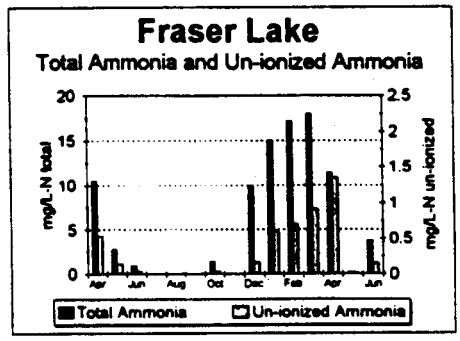


Figure 3.2.2.2



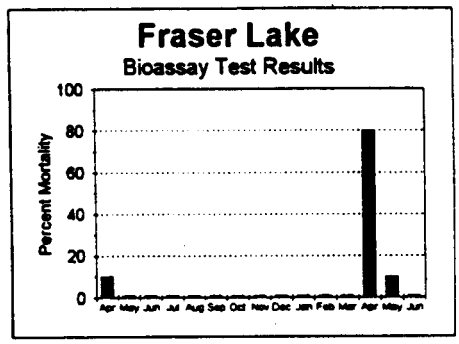
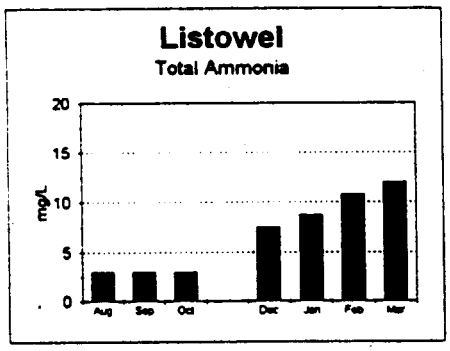


Figure 3.2.2.3



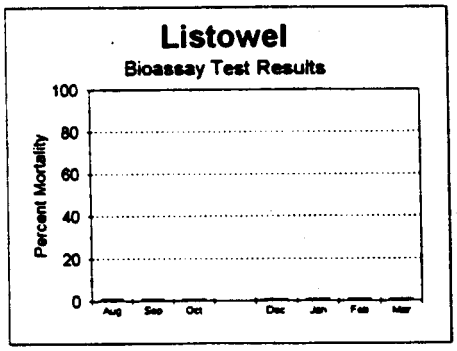
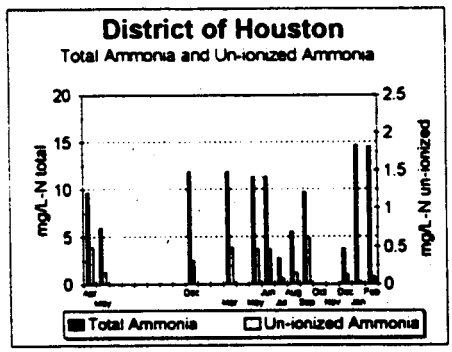
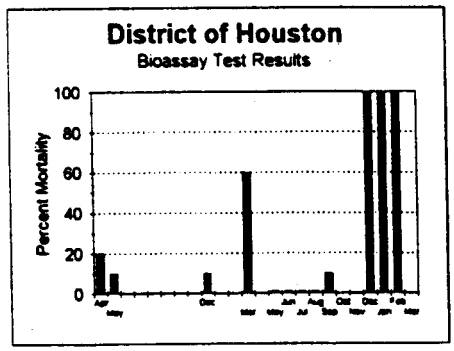


Figure 3.2.2.4





Percent Mortality incremental is 10% (total of 10 fish per test)

H:\1231\08\CHP3\CHP32\AERLAGN.WB1_C

The total and un-ionized ammonia concentrations in facultative and aerated lagoon effluents that resulted in a failed LC_{50} are presented in Figure 3.2.2.5. Data for failed bioassays where the pH of the effluent was not known was not plotted. The results shown in Figure 3.2.2.5 closely resemble those cited in the literature, as shown in Figure 3.2.2.6.

3.2.2.2 Hydrogen Sulphide

Laboratory tests of the Listowel lagoon effluent indicated a consistently non-acutely toxic effluent (24 hour test, sample preaerated). As determined by a Hach kit, the effluent contained less than detectable levels of H_2S. However, Slaughter et al., 1978, measured potentially lethal levels of H_2S in the receiving waters close to the lagoon outfall during that same month of February, 1978 (EPA, 1980). Fish were observed upstream in the receiving waters, but not for some distance downstream. Metikosh et al., 1980, concluded that this absence of fish downstream was due to high levels of H_2S in the receiving water resulting from the discharge of poor quality effluent from the Listowel lagoon during the winter months. This discrepancy may possibly be accounted for by differing sampling times or a low dissolved oxygen effluent undetected in pre-aerated lab samples. It was suggested that more work should be done to determine the aeration requirements of waste stabilization ponds during winter months to ensure non-toxic H_2S concentrations in the effluent (EPA, 1980).

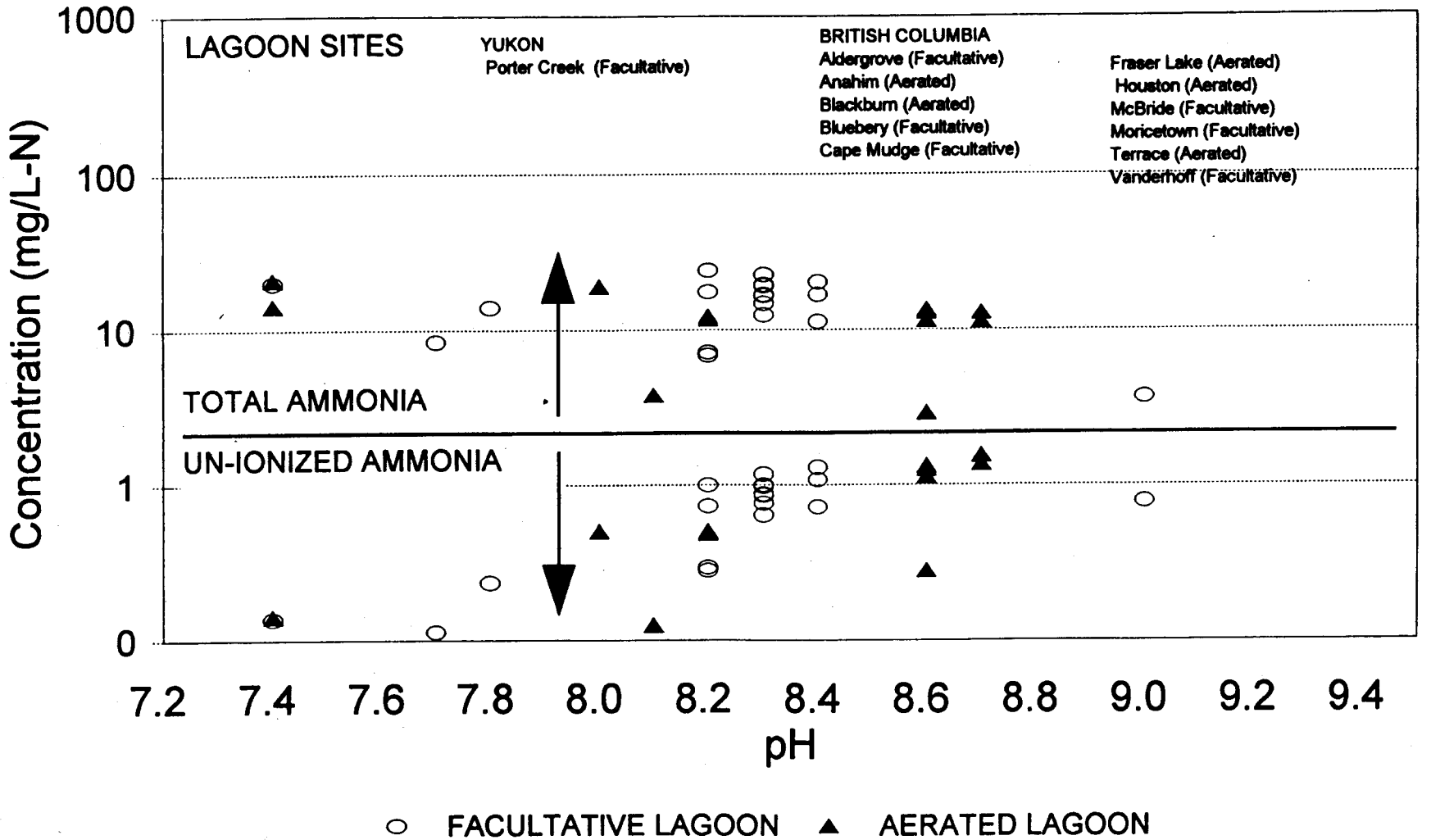
3.2.2.3 Retention Time

Comparing the toxicities of Fraser Lake and Blackburn lagoons over the year, Fraser Lake effluent was highly toxic for the month of April, 1992, whereas the Blackburn effluent was highly toxic for four months of the same year (January, February, March, June). The one significant design difference between these two lagoons is hydraulic retention time. The Fraser Lake lagoon has a total retention time of 158 days, whereas Blackburn's is only 34 days. The longer retention time correlates with an improved effluent quality. It should be noted, however, that ammonia levels are significantly high at Tumbler Ridge even with a hydraulic retention time of 136 days. Unfortunately, no corresponding toxicity data is available.

In Lake Cowichan (hydraulic retention time of 30 days), the influent ammonia concentration of 17.0 mg/L is only reduced to 15.9 mg/L during the month of February. This is accompanied by a slight increase in nitrate concentration (0.010 to 0.240 mg/L).

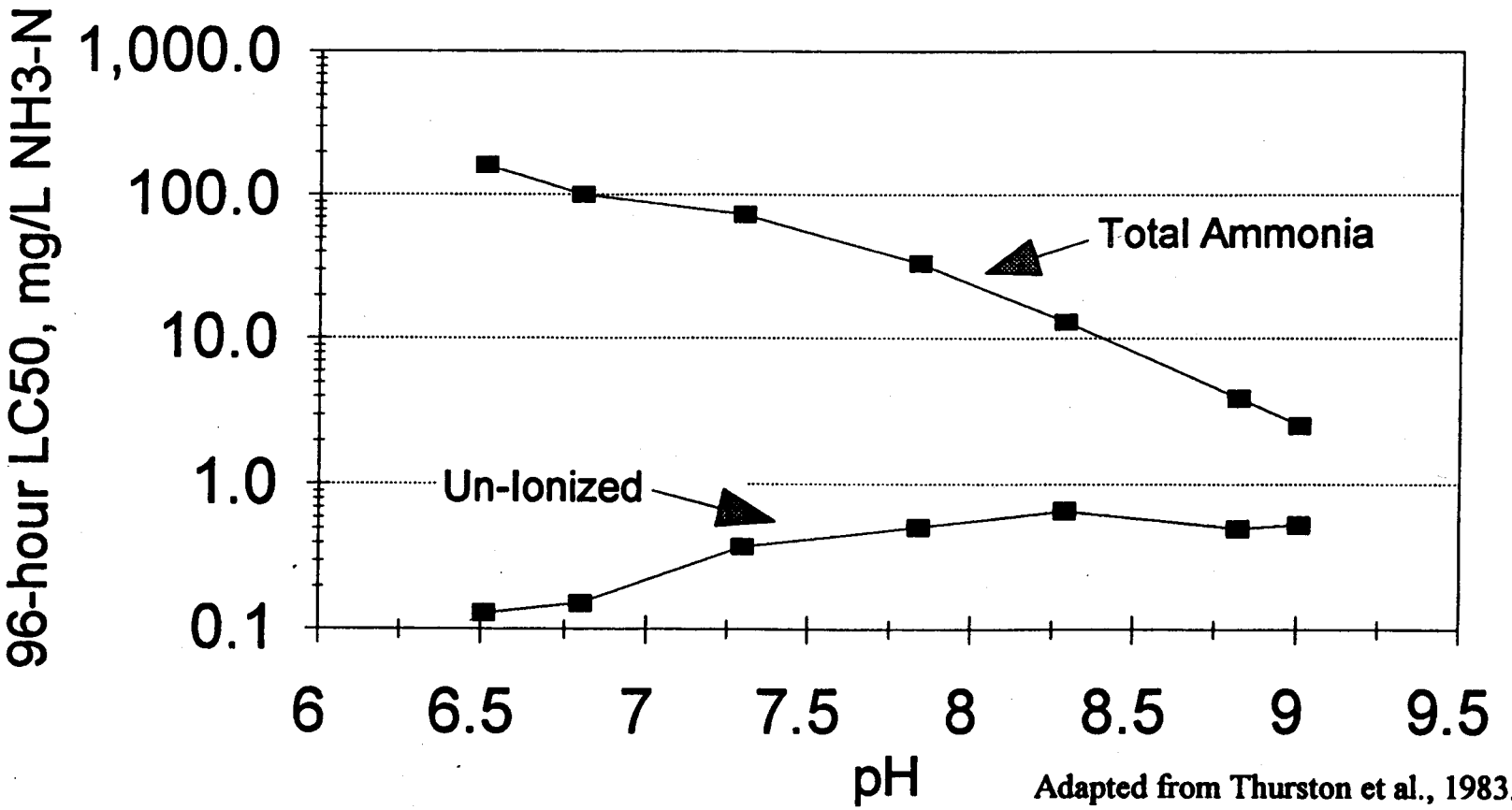
Figure 3.2.2.5

Total Ammonia & Un-Ionized Ammonia Data Resulting in Failed LC50



H:\ 1231 \ 08 \ CHP3 \ CHP32 \ NH3_TOX.WB1:REF_DATA

Figure 3.2.2.6
Acute Toxicity of Ammonia to Rainbow Trout



Adapted from Thurston et al., 1983.

In contrast, ammonia removal during the summer months appears to be very effective. The Williams Lake lagoon (total hydraulic retention time of 43 days) reduced the total ammonia concentration from 16.5 mg/L-N to 0.16 mg/L-N during the month of August. Similarly, the Clinton lagoon (hydraulic retention time of 137 days) reduced the total ammonia concentration from 9.1 mg/L-N to 1.6 mg/L-N during the month of August.

3.2.3 Previous Study

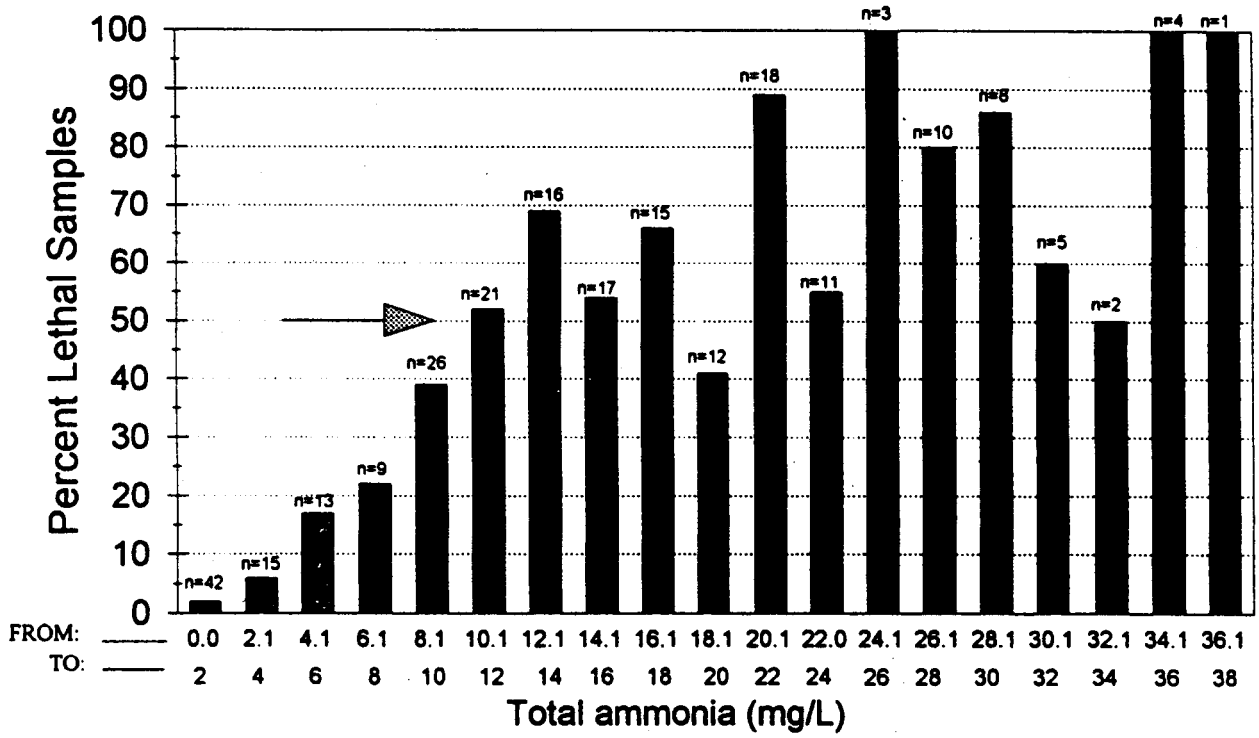
A study carried out by the Ontario Ministry of Environment (Ont. MOE, 1990) found effluent total ammonia toxicity levels to be around 10-12 mg/L-N. Figure 3.2.3 presents a summary of toxicity level data as accumulated by the Ontario Ministry of Environment. For each range of total ammonia concentrations, the percent of samples that proved to be acutely lethal are graphed. A sample is deemed acutely lethal if it results in a greater than 50% mortality of the test organisms. The number of samples taken for each range is given as n=#. With total ammonia concentrations in the range of 10-12 mg-N/L, 50% of the effluent samples were lethal. 10 mg-N/L was taken to represent the best estimated threshold for lethal ammonia concentration (Ontario MOE, 1990). In a separate report, also by the Ontario Ministry of Environment, an un-ionized ammonia concentration of 0.1 mg-N/L was identified as a lethal threshold value for trout (Ontario MOE, 1993).

3.2.4 Conclusions

The data reviewed in this section (3.2) suggest that an effluent total ammonia concentration less than 11 mg/L-N is a reasonable toxicity criteria for lagoon performance. However, the data also indicate that total ammonia levels less than this value can also be acutely toxic. Therefore, for the purpose of establishing a design criteria for upgrading technologies, a factor of safety is suggested with a corresponding total ammonia criteria of 5 mg/L-N. The ammonia toxicity issue with regard to lagoon systems is then whether lagoon effluents can be designed to consistently meet this criteria or not.

Of the 24 lagoons just examined, both aerated and facultative, in BC, the Yukon and Ontario, none were able to meet this criteria on a year-round basis. Of the lagoons researched at the University of Alberta (Smith, 1993), neither facultative nor aerobic lagoons were capable of meeting this criteria.

Figure 3.2.3 Total Ammonia-Nitrogen vs. Relative Frequency of Acutely Lethal Effluent Samples



Taken from Ontario Ministry of Environment, 1990

Based on the experience of the authors, the literature, and data review presented, it is not possible to consistently produce (ie. guarantee) a non-acutely toxic effluent on a year-round basis with either a facultative or aerated lagoon system, particularly where there is seasonal ice cover. Some form of upgrading technology is required to ensure adequate ammonia removal has been achieved (ie. storage ponds, wetlands, rapid infiltration basins, overland flow, fixed film systems, etc.). The only exception to this would be a lagoon with a one year retention time (i.e. long term storage), where ammonia accumulated in winter and spring could be treated in summer and fall, with a fall discharge.

As recommended in the WPCF Manual, Natural Systems for Wastewater Treatment (1990), new lagoon systems should base detention times on BOD_5 removal requirements. Estimates of the ammonia removal for the designed lagoon can then be made using model equations, and the resulting effluent ammonia concentration used for designing upgrading technologies. Two model equations are given in Table 3.2.4.

The literature suggest that it is possible to have effluent toxicity even at ammonia levels of 10 mg/L-N if there is poor BOD_5 removal efficiency. However, in adapting a 5 mg/L-N effluent ammonia criteria, low BOD_5 concentrations would be expected, alleviating the concern of BOD_5 influence on toxicity.

It is not practical to design a lagoon system for temperate or cold weather climates which can reliably maintain the effluent ammonia concentration at less than 5 mg/L-N. To ensure a year-round non-acutely toxic effluent, a lagoon system should be designed based on BOD_5 removal requirements, the effluent ammonia concentration estimated using the equations given, and an upgrading technology applied to further reduce the ammonia discharge concentrations.

Table 3.2.4 Nitrogen Removal Design

Model 1 (Reed et al., 1994)

$$N_e = N_o \, e^{-k_T[t \cdot 60.6\,(pH - 6.6)]}$$

Model 2 (Reed et al., 1994)

$$N_e = N_o \frac{1}{1 + t(0.000576\,T - 0.00028)\, e^{(1.080 - 0.042T)(pH - 6.6)}}$$

N_e=effluent total nitrogen, mg/L-N

N_o=influent total nitrogen, mg/L-N

k_T=temperature dependent rate constant

$=k_{20}\,(\Theta)^{(T-20)}$

k_{20}=rate constant at 20°C = 0.0064

Θ=1.039

T=temperature of lagoon water, °C

t=detention time, days

pH=pH of near surface of bulk liquid; if necessary estimate with:

$$pH = 7.3\, e^{0.0005ALK}$$

ALK=expected influent alkalinity, mg/L.

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4 LAGOON SYSTEM DESIGN AND DISPOSAL

4.1 FACULTATIVE LAGOON SYSTEM DESIGN

4.1.1 General Requirements

Before designing a facultative lagoon system, the designer should determine the following:

- whether the lagoon system can be continuously discharged or should operate on a seasonal discharge basis;
- the period(s) of the year when a discharge will not be permitted (eg. because of low stream flows);
- what discharge rates will be permitted for storage lagoons during the seasonal discharge period and what, if any, provision should be made for controlling effluent discharge rates in proportion to receiving stream flow rates;
- the minimum and maximum duration of discharge (i.e. seasonal) for storage lagoon cell contents.
- receiving water quality

For wastewater flows less than 20 m^3/day, a single cell facultative lagoon system is acceptable. For flows in excess of 20 m^3/day, however, a minimum of 2 cell treatment system should be used.

For multicell systems, permanent in ground piping and control devices shall be incorporated to permit temporary bypass of individual treatment cells. Where possible, piping shall also permit multicell systems to operate either in series or parallel.

4.1.2 Primary Treatment Using Anaerobic Cells

Where practical, primary treatment through the use of anaerobic lagoons is recommended for facultative lagoon systems, but not mandatory. Anaerobic lagoons serve to remove oil and grease, rags, grit, solids, and to reduce the BOD_5.

Peer experience has shown that anaerobic cells represent significant potential for odours. Where odour concerns could become issues with neighbouring property owners, anaerobic cells may not be appropriate. For example, in order to reduce the energy requirements of a complete aerated lagoon system, the City of Kamloops decided to include anaerobic cells into the system when upgrading in

the early 1980s. At the time, it was felt that a 2.0 km separation distance to residential properties was adequate to dissipate odours, particularly with a pulp mill in the same area. However, odours proved to be a major problem in the mid 1980s. Under inversion conditions and relatively calm winds, odour complaints related to the anaerobic cells were received in excess of 3.0 km from the site. In order to reduce odours, the City implemented a program of improvements including surface aerators on the anaerobic cells, screening, etc. at significant cost.

Anaerobic cells should operate at a depth of 3.0 to 5.0 m and retain influent flow for a 2 to 5 day period based on design flow rates. There should be two anaerobic cells operating in series. The piping system should be designed to allow either of the two cells to be taken out of operation for repair or desludging.

Each cell shall have a minimum bottom area of at least 3.0 metres by 3.0 meters to allow for adequate mobility of construction equipment and sufficient bottom area for sludge accumulation. At lower design flows this may result in a total anaerobic cell retention time in excess of the recommended 2 to 5 days.

4.1.3 Facultative Cells

Treatment cell sizing can be based on the plug flow lagoon model (EPA, 1983):

$$\frac{C_e}{C_o} = e^{-k_p t} \qquad (4\text{-}1)$$

Where:

Ce = effluent BOD_5, mg/L

C_o = influent BOD_5, mg/L

k_p = plug flow first order reaction rate, day^{-1}

t = detention time, days

For climatic areas without ice cover, an areal loading of 45 kg BOD_5/ha/day should not be exceeded over the system. The k_p value for a loading of 45 kg BOD_5/ha/day is as follows (EPA, 1983):

$$k_p \text{ (at 45 kg } BOD_5\text{/ha)} = 0.071\ (1.09)^{T-20} \qquad (4\text{-}2)$$

Where:

T = minimum operating lagoon water temperature, °C

With seasonal ice coverage, an areal loading of 22 kg BOD_5/ha/day should not be exceeded over the system, nor should the loading in the first cell be much greater than 40 kg/ha/day. Areal loading of 22 BOD_5/ha/day is also recommended by the Ontario Ministry of the Environment (1993). The k_p value for a loading of 22 kg BOD_5/ha/day is as follows (EPA, 1983):

$$k_p \text{ (at 22 kg } BOD_5\text{/ha)} = 0.045\ (1.09)^{T-20} \tag{4-3}$$

The lagoon water temperature can be estimated using Eq.4-4.

$$T_w = \frac{AfT_a + QT_i}{Af + Q} \tag{4-4}$$

Where:

T_w = lagoon cell water temperature, °C
T_a = ambient air temperature, °C
T_i = influent wastewater temperature, °C
A = surface area of individual cell, m^2
f = proportionality factor = 0.5
Q = wastewater flow rate , m^3/day

In cold climates, this equation sometimes results in a negative lagoon water temperature. In these cases, a minimum lagoon water temperature of 2°C should be used.

The normal operating design liquid depth of facultative lagoons is 1.5 m (under ice cover, 1.2 m) exclusive of sludge storage. Under temporary operating conditions, the maximum and minimum operating depths should be 2.0 m and 0.6 m respectively.

Consistent with Northwest Territory and Alberta Environment guidelines, and EPS Cold Climate recommendations, facultative cells should have a minimum retention time of 60 days.

The procedure for designing facultative lagoons is an iterative process:

1. Assume a water temperature, determine the K_P, and solve Eq.4-1 for detention time. From the given flow rate, determine the lagoon system volume.

2. Dividing the volume by the number of cells to be used and knowing the berm slope and design liquid depth, determine the surface area of each lagoon cell.

3. Calculate the lagoon cell water temperature using Eq.4-4.

4. Compare this calculated T_W and the original assumed value. If they differ by greater than 1°C, change the initial temperature assumption and repeat all calculations until the two temperature converge within 1°C.

If the resulting design does not meet BOD_5 loading requirements, i.e. the BOD_5 loading per area exceeds the recommended limits, the following procedure applies.

1. Increase the lagoon area to the minimum area (A_{min}) required to meet the maximum BOD_5 loading.

2. Calculate the new water depth (d_2) resulting from the new area (A_{min}) and same hydraulic retention time.

3. If this new depth (d_2) is less than the recommended minimum (d_{min}) of 1.2 m, the design depth must be set at the minimum depth (d_{min}). This will effectively increase the hydraulic retention time.

4. If this new depth (d_2) does exceed the recommended minimum, the design depth and area are set as d_2 and A_{min} respectively.

Extra depth should be allowed for sludge storage and, in cold climates, for ice cover.

4.1.4 Sludge Storage Volume

The sludge storage volume can be based on the following (EPA, 1983):

$$\text{Sludge Volume} = (1.5\ m^3/1000\ m^3 \text{ of sewage})\ (Q)\ (P) \qquad (4\text{-}5)$$

Where:

Q = design flow, m^3/d

P = number of days in lagoon life design period, days

Where anaerobic lagoons are included in system design, consideration can be given to reducing the sludge storage volume in the following lagoon(s).

4.1.5 Ice Cover

Ice cover should be assessed on a site specific basis and can be calculated by the Stefan Equation (Heinke et al. 1988) (see Section 5.5.4.2 Free Water Surface Wetlands, Case 3 for alternative version of same equation):

$$h = \epsilon \ (SFI)^{0.5} \qquad (4\text{-}6)$$

Where:

h = ice thickness, m

SFI = freezing index at ground surface, °C-sec

= summation of the degree-days below 0°C based on the mean daily temperature for the year (converted to °C - seconds). (Refer to the Canadian Climate Normals, published by Environment Canada).

ϵ = coefficient of proportionality, see Table 4.1.5.1, $m/(°C^{0.5}sec^{0.5})$

Table 4.1.5.1 "ϵ" Factor for Calculating Ice Cover (Zarling, 1978)

"ϵ" Factor x 10^{-5}	Conditions
10.4 to 11.0	Practical maximum for ice not covered with snow
9.3	Windy lakes with no snow
8.1 to 9.3	Medium-sized lakes with moderate snow cover

If no information is available, 1.0 m thick ice cover should be assumed.

For example, to estimate the ice cover of a lagoon in the Prince George area, the number of degree-days is taken from the Canadian Climate Normals, Degree Days, 1951 -1980 reference (Environment Canada, Atmospheric Environment Service). The number of degree-days below 0.0°C is 1057.9 for the year. Converting this value to degree-seconds, i.e. 1057.9 degree-days * 24 hours/day * 3600 seconds/hour = 9.14E7 degree-seconds. Assuming an ϵ value of 8.5E-5, the estimated depth of ice formation is 0.81 m.

4.1.6 Facultative Storage Cell(s)

In cold climates with ice and snow cover, treatment efficiency drops significantly. Long term storage, with seasonal discharge during appropriate receiving water and effluent quality conditions, may be required under such conditions.

Long term storage with seasonal discharge provides additional wastewater treatment during warmer months (spring, summer and fall) under facultative conditions after the ice melts, and reduces the environmental impact on the receiving waters. Storage pond criteria are further described in Section 6.1.

4.2 AERATED LAGOON SYSTEMS DESIGN

4.2.1 General Requirements

In designing aerated facultative lagoons (partial mix aerated lagoons), consideration should be given to:

- BOD_5 removal;
- Effluent characteristics;
- Oxygen requirements;
- Mixing requirements;
- Wastewater and air temperature;
- Oxygen transfer and mixing equipment;
- Solids separation.

Where practical, primary treatment through the use of anaerobic lagoons is recommended for aerobic lagoon systems. Again, for each site, the potential for anaerobic cell odours and associated neighbour complaints should be anticipated and accounted for (see Section 4.1.2 Primary Treatment Using Anaerobic Cells).

4.2.2 Number of Cells

A minimum of two cells is recommended, and three or four cells are preferred. A polishing zone or cell is desirable.

4.2.3 Sizing of Treatment Cells

If preceded by anaerobic cells, aeration cells should be of equal volume. In cases where there is no primary treatment in anaerobic cells, peer experience has shown it to be beneficial to design an aerobic lagoon system with a relatively short retention initial cell followed by a longer retention aerated cell(s). This approach concentrates sludge in a relatively small area and therefore simplifies subsequent sludge removal.

The aeration cells should be of equal volume.

Consistent with Ontario Ministry of Environment, Alberta Environment and EPS Cold Climate recommendations, the minimum detention time should be 25 days, followed by 5 days in the polishing cell in temperate climates. The minimum detention in cold climates should be 30 days in the aerated cells and 5 days in the polishing cell. When designing strictly for BOD_5 reduction without consideration for effluent ammonia concentrations, the aerated lagoon detention time should be a minimum of 15 days.

Additional volume for sludge storage and ice cover should be provided. (For details refer to Section 4.1.4 - Sludge Storage Volume and Section 4.1.5 - Ice Cover).

A conservative approach in sizing the aerated lagoons is to use the partial mix lagoon model (EPA, 1983). However, experience has shown that aerated lagoons fall somewhere between the complete and partial mix models.

$$t = \frac{n}{k_{pm}} \left[\left(\frac{C_o}{C_n}\right)^{\frac{1}{n}} - 1 \right] \tag{4-7}$$

Where:

t	=	minimum detention time, days
n	=	number of cells
C_o	=	influent BOD_5, mg/L
C_n	=	effluent BOD_5 from the nth cell in an n cell series, mg/L
k_{pm}	=	reaction rate for an aerated lagoon, at design temperature. The reaction rate, k_{pm}, can be adjusted for temperature by the following equation (Townshend & Knoll, 1987; WPCF, 1990):

$$\frac{k_{pm}}{k_{pm20}} = (\theta_w)^{T_w - 20} \tag{4-8}$$

Where:

k_{pm}	=	reaction rate at design temperature, $days^{-1}$
k_{pm20}	=	reaction rate at 20°C, $days^{-1}$: for complete mix $k_{pm20} = 2.5\ day^{-1}$ for partial mix $k_{pm20} = 0.276\ day^{-1}$
Θ_w	=	temperature factor, dimensionless = 1.036
T_w	=	temperature of lagoon water, °C

The reaction rate noted above is for domestic sewage; if the sewage includes some industrial wastes, other wastes and partially treated sewage, the reaction rate should be determined experimentally for various conditions that might be encountered in the aerated lagoons. Conversion of the reaction rate coefficient at other temperatures should be made based on experimental data.

The impact of the ambient air temperature on the lagoon water temperature can be calculated from the following equation (EPA, 1983).

$$T_w = \frac{AfT_a + QT_i}{Af + Q} \tag{4-9}$$

Where:

T_w = lagoon water temperature, °C
T_a = ambient air temperature, °C
T_i = influent wastewater temperature, °C
A = surface area of individual cell, m^2
f = proportionality factor = 0.5
Q = wastewater flow rate , m^3/day

In cold climates, this equation sometimes results in a negative lagoon water temperature. In these cases, a minimum lagoon water temperature of 2°C should be used.

The procedure for designing aerated lagoons is an iterative process:

1. First assume a wastewater temperature and solve Eq. 4-8 for k_{pm}, reaction rate at design temperature.

2. Having already determined the number of cells desired, solve Eq. 4-7 for t, minimum detention time. If this is less than the minimum number of days recommended, then proceed with detention time = minimum.

3. From the detention time and flow rate, calculate the lagoon volume.

4. Choose a design liquid depth (i.e. not including ice, sludge and freeboard allowances). Determine the resulting surface area, taking into account berm slopes (see Section 8 Examples).

5. Calculate the lagoon wastewater temperature using Eq. 4-9. Continue iterations of this process until the wastewater temperature assumed at the beginning matches that determined by Eq. 4-9 to within 1.0°C.

4.2.4 Design Depth

The design water depth of the aerated cell(s) should be 3.0 to 4.5 m. This depth limitation may be altered depending on the aeration equipment used, waste strength and climatic conditions.

4.2.5 Settling Zone/Cell

A quiescent settling area for suspended solids should be provided as the last cell or as part of the last cell. The choice of whether to create a separate polishing cell from the aeration cell(s) will depend on the size of the system and the type of aeration equipment used. For example, for the larger systems with multicells and mechanical or coarse bubble aeration, a separate polishing cell may be appropriate, whereas, for small systems with diffused fine bubble tapered aeration, a settling zone in the last cell may be appropriate. The settling cell or zone should have a minimum hydraulic detention time of 5 days based on the design average daily flow.

4.2.6 Aeration Equipment

4.2.6.1 General

Aeration is used to mix the cell contents and to transfer oxygen to the liquid. Oxygen requirements generally depend on the maximum diurnal BOD_5 loading, the degree of treatment, and the concentration of suspended solids to be maintained in the lagoons.

Aeration equipment should be capable of maintaining a minimum of 2.0 mg/L of dissolved oxygen in the liquid at all times. Dissolved oxygen meters are recommended for checking and possibly controlling this aspect. In the absence of experimentally determined values, the oxygen required to address the BOD_5 loading of the influent wastewater should be estimated using 1.1 kg O_2 / kg peak BOD_5, or 1.5 kg O_2 / kg peak BOD_5 in cold climates.

Peer experience has shown that an additional 50% allowance for benthic oxygen demands prevents the oxygen transfer capacity of the aeration system from being inadequate as the design flow is approached. This also provides a further safety factor should a submerged aerator cease to function shortly after installation. Because of the additional aeration, the process of draining the lagoon to replace an aerator can be avoided.

The recommended design oxygen requirements are then:

1.65 kg O_2 / kg peak BOD_5 in temperate climates
2.25 kg O_2 / kg peak BOD_5 in cold climates

The calculated design oxygen requirement must be adjusted for field conditions (i.e. altitude, dissolved oxygen level expected to be maintained, and the lagoon liquid and temperature). The following formula can be used (EPA, 1983):

$$\frac{N}{N_0} = \frac{E}{E_0} = \left(\frac{\beta C_{ss} P - C_L}{9.17}\right) \alpha \ (1.024)^{T-20} \qquad (4\text{-}10)$$

or (EPA, 1983)

$$E = E_o \left(\frac{\beta C_{ss} P - C_L}{9.17}\right) \alpha \ (1.024)^{T-20} \qquad (4\text{-}11)$$

Where:

N	=	rate of oxygen input into wastewater under actual conditions, kg/hr/kW
N_o	=	rate of oxygen input under standard conditions (20°C, 101.3 kPa), kg/hr/kW
E	=	actual oxygen absorption efficiency
E_o	=	oxygen absorption efficiency under standard conditions, varies from 4 to 12 percent for diffusers and spargers in domestic wastewater. The variation depends on bubble size and the general design and arrangement in the lagoon.
α	=	relative air-water interface diffusion rate
ß	=	the ratio of saturation values of wastewater to clean water at wastewater temperature and actual atmosphere pressure (approximately 0.9 for domestic wastewater)
C_L	=	desired DO in the lagoon, mg/L
C_{ss}	=	oxygen saturation concentration for clean water at design temperature, mg/L

P	=	ratio of barometric pressure at plant site to barometric pressure at sea level
T	=	liquid temperature in lagoon, °C
9.17	=	oxygen saturation concentration at 20°C and 101.3 kPa

In the absence of an experimentally determined alpha value, a factor of 0.85 can be assumed for systems treating primarily (90% or greater) domestic sewage. Where the wastewater contains a higher percentage of industrial wastes, a correspondingly lower value should be used and calculations should be included in the design brief to justify such a value.

Having determined the oxygen absorption efficiency, air requirements for a diffused air system can be calculated from the following:

$$Q_A = \frac{O_r}{E \times 1.2} \qquad (4\text{-}12)$$

Where:

Q_A = air required, Std m^3/hr

O_r = oxygen required, kg/hr (i.e. 1.1 to 1.5 kg/kg BOD_5 as noted previously)

Density of air = 1.2 kg/m^3 at standard conditions (20°C, 101.3 kPa)

A high proportion of the aeration capacity should be provided at the lagoon influent to maintain influent solids in suspension, and to meet the high influent BOD_5. In aerated facultative (partial mix) cells, power input is reduced in each succeeding cell because of the reduction in organic matter to be oxidized as the wastewater flows through the system. Careful consideration should be given to maximizing oxygen utilization per unit of power input. Final sizing of aeration equipment should be based on guaranteed performance by an equipment manufacturer. Certified testing, in accordance with the latest ASCE testing procedures, can be provided to verify the mechanical aerator's performance. Field testing for oxygen transfer can be included in the design specification.

4.2.6.2 Diffused Air System

Non-clog type diffusers are recommended, and the diffuser systems should be capable of providing for the diurnal peak oxygen demand or 200% of the design average oxygen demand, whichever is larger. The air diffusion piping and diffuser system should be capable of delivering normal air requirements with minimal friction losses. Air piping systems should be designed such that the total

head loss from blower outlet (or silencer outlet where used) to the diffuser inlet does not exceed 3.4474 kPa (0.5 psi) at average operating conditions.

Air piping should be sloped from the blowers to the lagoons for draining condensate to the lagoon(s), or a condensate trap with valve cock and drain should be employed.

The spacing of diffusers should be in accordance with the oxygen requirements through the length of the cell, and should be designed to facilitate adjustment of their spacing without major revision to air header piping.

Individual assembly units of diffusers should be equipped with control valves and preferably with air flow indicators for throttling or for complete shutoff. Diffusers in any single assembly should have uniform pressure loss.

The specified capacity of blowers or air compressors, particularly centrifugal blowers, should take into account that the air intake temperature may reach 40 °C or higher and the pressure may be less than normal. The specified capacity of the motor drive should also take into account that the intake air may be -30 °C, or less, and may require oversizing of the motor, or a means of reducing the rate of air delivery to prevent overheating or damage to the motor.

The blowers should be provided in multiple units, so arranged, and in such capacities, as to meet the maximum air demand with the single largest unit out of service. The design should also provide for varying the volume of air delivered in proportion to the load demand of the treatment system.

To prevent damage to blowers and clogging of the diffusers, air filters should be provided in numbers, arrangements, and capacities to furnish at all times an air supply sufficiently free from dust.

4.2.6.3 Mechanical Aeration

The mechanism and drive unit should be designed for the expected conditions in the aerated lagoon in terms of the power performance. The design requirements of a mechanical aeration system should accomplish the following:

- Meet maximum oxygen demand and maintain process performance with the largest unit out of service;

- Provide for varying the amount of oxygen transferred in proportion to the load demand on the lagoon(s);

- Ensure that adequate pumping or mixing is provided to maintain a uniform dissolved oxygen level in the cell and to maintain the proper level of suspended organic solids;

- Provide overlapping zones of mixing with aerators spaced in proportion to the expected oxygen demand; the oxygen demand should decrease in each succeeding cell;

- Ensure the mechanism is protected from freezing where extended cold weather conditions occur, and ensure that the propeller is positioned below the potential ice depth.

4.3 EFFLUENT DISPOSAL

If effluent meets the water quality requirements, it can be discharged directly (i.e. no upgrading technologies employed) to surface waters (streams, rivers, lakes, estuaries, oceans) or to ground water recharge on either a seasonal or continuous discharge basis. Under ideal climatic conditions, effluent total retention with evaporation as the only means of disposal may be possible.

When discharging effluent continuously, full advantage of the dilution capacity of the receiving waters should be taken by ensuring the discharge point is located where natural mixing is optimized. Whenever insufficient dilution in the receiving water is encountered, effluent storage with seasonal discharge may be necessary.

A drainage course receiving a seasonal discharge from a wastewater stabilization lagoon should be capable of transporting the effluent to the ultimate receiving waters (streams, rivers, lakes, wetlands, estuaries, and ocean) without the occurrence of either flooding of adjacent lands or erosion of the drainage course itself.

For seasonal discharges to receiving waters such as streams, rivers and lakes, the time of year and actual duration depends on regulations and receiving water protection requirements. Factors such as dilution ratios (i.e. rate of discharge and stream/river flow conditions) and effluent quality should be considered. When selecting the season(s) of discharge, discharge during the fall is preferred. (Details refer to Section 6.1 - Storage Ponds). The duration and season of discharge should be determined based on individual site characteristics in consultation with Department of Fisheries and Oceans Canada and BC Ministry of the Environment, Fisheries and Wildlife Branch. As an example, Alberta Environment stipulates a maximum discharge duration of three weeks, preferably in the fall (Alberta, 1988).

In all cases, a water mass balance should be done to maintain a satisfactory water level in the lagoon system.

4.3.1 Water Mass Balance

The water balance in a lagoon is dependent upon inflow, outflow, precipitation, evaporation, infiltration and seepage. Special attention should be taken during the initial period, when the sewage flow is much lower than the design flow, to prevent desiccation of the soil liner materials and berm erosion. When designing for storage losses, the designer should ensure that a minimum operating

depth of 0.6 m is maintained in the facultative lagoons. When designing evaporation cells, the depth should be no greater than 1.5 m.

4.3.2 Seepage

Wastewater lagoon seepage considerations are one of the most important aspects of lagoon design, construction, operation and maintenance. The maximum allowable seepage should be 5.2×10^{-9} m/s where seepage is prohibited or undesirable. The use of impermeable membranes as liners to minimize the exfiltration of lagoon liquid may be a necessary requirement as dictated by the soil conditions and potential environmental impact.

The maximum allowable hydraulic conductivity for lagoon liners consisting of in-situ soil materials, compacted clay, bentonite and sand, asphaltic concrete or other porous materials in which seepage is governed by Darcy's Law, can be calculated using the following equation (Alberta, 1988):

$$\textit{Maximum allowable } K_t = \frac{5.2 \cdot 10^{-9}(d)}{2 + d} \qquad (4\text{-}13)$$

Where:

K_t = hydraulic conductivity of liner in the field, being at least one order of magnitude greater than the laboratory value, m/s

d = required or proposed thickness of liner, metres

Peer comment suggests that an improperly applied asphaltic concrete can lose its cohesive strength, simply disintegrating over time. Care should be taken in choosing a liner suitable for the local climate conditions.

When some discharge occurs by means of infiltration into the surrounding ground, ground water movement and its potential contamination must be considered.

4.4 PHYSICAL DESIGN AND CONSTRUCTION CONSIDERATIONS

4.4.1 Setback Distances

Setback distances from wastewater stabilization lagoons are required to buffer the effect of potential odours and to provide a margin of public safety. Buffer zones also serve to protect the integrity of nearby buildings and roads during construction or in case of berm failure. Table 4.4.1 outlines the minimum recommended horizontal setback distances for wastewater stabilization lagoons. Again, the potential for odour issue concerns being raised by nearby neighbours should be considered using site specific criteria.

Table 4.4.1 Recommended Setback Distances From Community Wastewater Stabilization Lagoons Outside Berm Toe

To	Minimum Setback (m) Anaerobic & Facultative Lagoons	Minimum Setback (m) Aerobic Lagoons
The property line of any residential, commercial or industrial property, or the designated right-of-way of a rural road or railway	30	30
The designated right-of-way of primary or secondary highway	100	30
Any existing or potential residential or institutional (e.g. school, church, hospital, community centre) sites	300	100

Note: These are recommended MINIMUM distances. Site dependent criteria should be used for design.

(Adapted from Alberta 1988, Heinke et al. 1988, Townshend and Knoll, 1987.)

4.4.2 Physical Site Constraints

Wastewater stabilization lagoons should not be located in any area which is prone to flooding during infrequent storm events. The top of the berms should be at or above the one in 200-year flood return elevation. On a site-specific basis, it may also be necessary to provide an adequate setback distance to adjacent drainage course.

The topography of the site should be selected to minimize interception of surface run-off, ground water or snow melt. Diversion ditches should be constructed for unavoidable interception of these waters. For facultative lagoons, the site selected should have maximum sunlight reaching the surface. In a hilly area, the south side of hills is preferred.

4.4.3 Prevailing Winds

In relation to inhabited areas, lagoons should not be located upwind in the direction of the local prevailing winds.

To reduce short circuiting due to induced current generated by winds, facultative and storage lagoons should be oriented with their lengths perpendicular to the direction of the local prevailing winds.

4.4.4 Geology and Hydrogeology

A geotechnical investigation and report must be done for all earthen berm construction to determine the suitability of the soils, the limiting site conditions, and the construction procedures. A geotechnical study should address issues such as, but not limited to:

- suitability of soils for embankment construction
- typical berm cross-sections with any zoned construction clearly defined in terms of material types and dimensions
- seepage control requirements
- compaction (density requirements)
- erosion control measures
- seepage calculations for assessing liner requirements
- density requirements (eg. Standard Proctor Density) for berm construction

Site and subsurface investigations in combination with appropriate laboratory analyses should be used to determine the design and construction criteria and to identify any necessary operational requirements.

At least one boring should be a minimum of 7.5 m in depth or into bedrock, whichever is shallower. If bedrock is encountered, rock type, structure and corresponding geological formation data should be provided. The boring should be filled and sealed. The permeability characteristics of the lagoon bottom and lagoon seal materials should also be addressed.

A minimum separation of 3.0 m between the lagoon bottom and any bedrock formation is recommended. This is to maintain a distance between the lagoon bottom and soil water levels, allowing for groundwater mounding and capillary rise beneath the lagoon.

Lagoons should not be located in areas that may be subjected to karstification, (i.e. areas where rainwaters dissolve limestone, forming caves, channels and sinkholes.)

Construction of lagoons in close proximity to water supplies and other water bodies sensitive to contamination should be avoided. In these areas, a minimum separation of 1.2 m between the bottom of the lagoon and the seasonal high water level should be maintained.

4.4.5 Groundwater Monitoring

Groundwater monitoring standpipes should be installed to determine the effectiveness of installed seepage control measures and to identify any existing or potential seepage related problems. Groundwater observation standpipes may be required during the site investigation program, installed in exploration boreholes outside the lagoons.

One monitoring standpipe should be installed for every 2 hectares of site, with a minimum of 5 installations per site. At least one standpipe should be upgradient from the lagoon to provide background water quality data. The number and location of monitoring wells, the frequency of groundwater monitoring, and the water quality analyses vary based on local factors, and should be established during the engineering design.

4.4.6 Shape

The shape of all cells should be such that there are no narrow or elongated portions. Anaerobic lagoons should ideally be square or rectangular. Aerated cells should ideally be square if they are in a multi-celled aerated lagoon system (i.e. more than one aerated cell followed by a polishing cell). Otherwise the aerated cell should have length to width ratios of 2:1 to 4:1 as desirable objectives. Facultative lagoons should ideally have length to width ratio of 2:1 to 4:1. Cell dimensions and configurations are in many cases determined by site topography. No islands, peninsulas or coves should be permitted. Dikes should be rounded at corners to minimize accumulations of floating materials. Common-wall dyke construction, wherever possible, is strongly encouraged.

For very small lagoons with special configuration, installation of aeration equipment should be provided for positive control of short-circuiting through the lagoon.

4.4.7 Liner Design

The following design guidelines are offered, however, liner selection and detailed design should be done by qualified design and geotechnical professional engineers. See Section 4.3.2 Seepage for seepage calculations.

4.4.7.1 Natural In Situ Liners

In situ liners should have a minimum thickness of at least 0.9 m below the entire bottom, be relatively uniform and homogeneous, and completely free of hydrogeologic windows such as sand and silt. Engineered sideslope liners should provide if the horizontal hydraulic conductivity of the in situ liner does not meet the seepage control criterion (see Section 4.3.2 - Seepage).

4.4.7.2 Compacted Clay Liners

Alberta Environment recommends that compacted clay liners have a minimum thickness of 0.6 m on the bottom and 1.2 m on the sideslope (measured perpendicular to the slope). Experience shows this facilitates achieving required compaction densities with conventional compaction equipment. The liner should be constructed in 150-200mm lifts and compacted to the required density, within the required moisture content range, to achieve the required seepage control criterion. When determining whether the seepage control criterion is met, the actual hydraulic conductivity of the liner in the field shall be assumed to be at least one order of magnitude greater than laboratory values.

4.4.7.3 Betonite and Sand Admix Liners

Bentonite should only be considered for lagoon liners where mixing with native sands or silts allows a uniform bentonite and sand/silt admix. Only moderate to high swelling sodium bentonite should be used. The bentonite application rate required to meet the seepage control criterion should be determined by laboratory permeability tests and then increased by 25% to allow for field conditions. The admix liner should be at least 100 mm thick after compaction, and any portion of the liner that is susceptible to weathering when exposed should be covered with suitable soil material.

4.4.7.4 Manufactured Clay Liners

A manufactured clay liner, such as "Claymax", may be used to reduce permeability. The manufactured clay liner consists of a layer of bentonite sandwiched between layers of geotextile. Clay liners are specified in weight (of bentonite) per unit area instead of thickness. The design of a manufactured clay liner system should be completed with assistance from the liner manufacturer and a geotechnical engineer.

4.4.7.5 Asphalt Liners

The only asphalt liner should be spray-on bitumen over asphaltic concrete or soil asphalt. Hydraulic asphaltic concrete liners should be a minimum of 100 mm thick comprising two 50 mm lifts with staggered joints. Soil asphalt liners should be mixed with 150 mm of native sandy soil. In all cases, asphalt should be placed over non-frost susceptible base courses or subgrades. The spray-on bitumen covering the hydraulic asphaltic concrete or soil asphalt surface should provide a uniform 2 mm thick membrane. Peer review's experience indicated concern with liner decay problems in asphalt liners over the long term.

4.4.7.6 Flexible Polymeric Membrane Liners

Alberta Environment recommends that the minimum thickness of membranes should be 20 mils, although 30 mils is recommended here. Membranes less than 60 mils thick should be covered with a 300 mm layer of fine-grained soil on the berm sideslopes to prevent liner damage. PVC and other membrane liner materials that are susceptible to weathering (e.g. UV degradation) should be covered with soil on both the sideslopes and bottom. A stable and well prepared subgrade and proper membrane installation, with particular emphasis on seaming, should be provided. A system for venting gas generation beneath the liner should be considered.

Where synthetic liners are used, peer comments suggest that some means of rescue is provided such as a life ring and 30 metres of rope be mounted in an accessible place inside the fencing of the lagoon as a safety measure should a person fall into the lagoon. Synthetic liners are much more slippery than natural liners.

4.4.8 Piping and Channels

All piping should be either polyethylene, PVC, or ductile iron. Where settlement is a concern (due to freeze/thaw conditions or otherwise), the piping should be located so as not to jeopardize the integrity of the lagoon liner or seal. Pipe should be anchored with adequate erosion control.

Where cells are at or near the same elevation, the interconnecting pipes should be valved. Where cell elevations differ significantly, the cross connection pipe should have a chamber with a weir to control flow from the higher cell. The valve or chamber should be provided with suitable locking devices, and be located off the travelled portion of the top of the berm.

For continuous discharge, the hydraulic capacity of pipes and channels within the lagoon system and for discharge structure should be designed at 2.5 times average flow.

Channels and pipes carrying liquids with solids in suspension should be designed to maintain a self-cleansing velocity of 0.6 m/s within the design limits. Adequate provisions should be made to drain segments of channels which are not being used due to alternate flow patterns. The discharge piping elevation should be set 300 mm above the sludge storage zone to minimize the discharge of algae.

The hydraulic capacity for seasonal discharge systems should permit transfer of water at a minimum rate of 150 mm of lagoon water depth per day at the available head, and should have adequate capacity for the design discharge rate.

Concrete seepage collars should be constructed at a maximum of 5 m on centre wherever any interconnecting piping passes through a berm. This is to minimize the possibility of piping along the outside of these pipes with subsequent erosion and failure of the earthen berm.

4.4.9 Flow Distribution Structures

Control manholes are commonly used for access to inlet piping and to regulate the flow from the influent pipeline to the cells. The invert(s) should be at least 150 mm above the maximum operating level and provide sufficient hydraulic head without surcharging the upstream manhole. Flow distribution structures and piping should be designed to allow:

- effectively splitting of hydraulic and organic loads equally to the cells;
- isolation of the cells;
- temporary bypassing of individual cells;
- and, where possible, operation in either series or parallel.

4.4.10 Surface Runoff Diversion

Adequate site drainage should be provided to divert surface runoff that would otherwise cause damage to the system.

4.4.11 Drain Structures

Drains should be provided to ensure that all cells are capable of being drained by gravity to within 300 mm of the cell bottom. These gravity drain pipes should be connected to a vertical manhole in the lagoon berm into which a pump could be conveniently installed. This reduces the possibility of the operator un-intentionally draining the lagoon; this is more likely to occur in totally gravity drained systems. The main drainage pipe should be equipped with a long stem valve so that it can be operated without entering the manhole. Erosion protection may be required at the location of effluent discharge.

In peer review it was recommended that the number of pipes passing through berms and, in particular, pipes at or near the floor elevation such a drains, be minimized. All pipes represent a potential seepage path for lagoon contents. Seepage in and around a seldom used drain pipe may go unnoticed by operational staff and result in berm instability.

4.4.12 Overflows

Provision for emergency overflow should be made for all cells. The overflow from the final cell should be directed to the outfall.

It was noted in peer review that problems can arise with ice formation around overflow pipes. Specifically, ice would form under or around the overflow pipe and lift the pipe in the berm as the lagoon level rises. The lagoon contents could then exit the cell in the void space under the overflow pipe, leading to berm failure. However, this problem can be designed around by placing the overflow pipe flush with the berm.

4.4.13 Berms

The top of berm elevation should be at or above the one in 200-year flood return elevation.

For multicelled systems, embankment tops should have a minimum width of 4.0 m to permit access of maintenance vehicles. Peer experience has shown that it is very difficult to drive a pickup truck along the top a frozen, snow covered, 3 m wide berm. Where vehicle access is not required, 2.5 m is recommended.

All topsoil should be stripped from the area on which the berms are to be constructed.

Embankment slopes should be as steep as the safe operation of equipment will permit and the local soil condition will allow. Slopes of 3:1 (horizontal:vertical) on outer slopes are recommended to safely allow grass mowing; inside berm slopes should be no steeper than 3:1.

Alberta Environment recommends that the vertical distance between the high water or ice level and the top of the berm should be a minimum of 0.3 m with ice build up, and 0.6 m without ice. However, peer experience recommends a greater freeboard of 1.0 m. Increased 'freeboard' may be required where high winds and steep embankments result in water scouring.

Material used should have seepage rates less than 5.2×10^{-9}m/s under a head of 1.8 m.

Soil conditions may require the berm(s) to be "keyed" into the subsoil to preclude the horizontal seepage across the base of the berm.

4.4.14 Lagoon Embankment Surface

Lagoon embankment surfaces are not normally underwater. Inner berm surfaces down to 1.0 m below the high water level should be protected from erosion using either pit run gravel, rip-rap, or mulch seeding on 100 mm of the topsoil layer.

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Where cells will be empty for a long period before filling, the entire berm surface should be mulch seeded to prevent erosion.

The cell bottom should be level and flat (±150 mm), or to within tolerances required by the aeration equipment manufacturer.

4.4.15 Access

An all weather access road should be provided to the lagoon site to allow year-round maintenance of the facility.

4.4.16 Flow Measurement

Provision for flow measurement should be made at the outlet manhole. Effluent weirs, flumes or other devices combined with their associated depth measurement method should be capable of monitoring flow under all outlet conditions.

4.4.17 Fencing

The lagoons should be enclosed with a fence. The fence should be designed and constructed to prevent the entrance of children, and to discourage trespassing. The fence should also serve to preclude the entrance of livestock.

Fences should be located away from the outside toe of the berms by at least 1.0 m to facilitate mowing and maintenance operations. An access gate should be provided to allow entry of maintenance equipment. The gate should be provided with a lock to preclude entrance of unauthorized persons.

4.4.18 Warning Signs

Warning signs should be provided at appropriate locations along the fenced perimeter of the wastewater stabilization lagoons. Each sign should identify the facility by owner, facility contents, advise against trespassing, and provide emergency contact names, phone numbers and/or addresses.

4.4.19 Winter Protection

Special care should be taken to avoid freezing problems when transferring from one cell to another. The application of adequate insulation and/or addition of heat tracing at the transfer points may be required.

If baffles are designed to extend into the freezing zone, the baffles should be designed against ice damage.

Suitable protection of electrical controls from weather should be provided.

4.4.20 Operator Training

The Ministry of Environment currently requires all municipal wastewater treatment facilities to be classified by the BCWWOCP, and all facilities level II and higher to have a certified operator. The BC Operators Certification Board should be contacted to determine operator requirements for new or existing lagoon systems.

4.4.21 Servicing

Provision should be made to facilitate servicing aeration units in place and to remove aeration units from their location for servicing.

4.4.22 Maintenance Facilities

Hosing and cleaning equipment to maintain sanitary conditions should be provided.

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5 CONSTRUCTED WETLAND SYSTEMS FOR UPGRADING LAGOON EFFLUENTS

5.1 INTRODUCTION

The use of constructed wetlands as a wastewater treatment process is a technology which has been under development since the early 1980's. The improvement in water quality which can occur during passage through a natural marsh was observed and it seemed that this performance could be duplicated and even improved by careful construction and management of a marsh system. These constructed wetland systems can be designed to provide optimum treatment benefits and they can be constructed wherever suitable topography and climate conditions permit. Low temperatures during the winter months reduce the rate at which the biological reactions in the wetland occur. In very cold climates a thick ice cover on the wetland can physically restrict the intended flow through the system. In these cases, a seasonal wetland is used where wastewater is stored in the lagoon over the winter months and discharged to the wetland during the summer and early fall. It is estimated that about 1000 wetland systems are currently in operation, for a variety of purposes, on a world wide basis. At least half of those are in North America, and a large portion of those are utilized for lagoon upgrading. A database of wetland systems in North America was recently prepared for the US EPA (Knight et al, 1993). Rational design procedures for removal of wastewater pollutants in constructed wetlands have been developed from this database and similar sources.

Wetlands are not presented as a primary form of treatment, but rather as an upgrading technology option for wastewater lagoon effluent. The following design information addresses the issue of ammonia reduction, not TTS/BOD_5 reduction, although TSS and BOD_5 are concurrently removed in ammonia removal wetland systems.

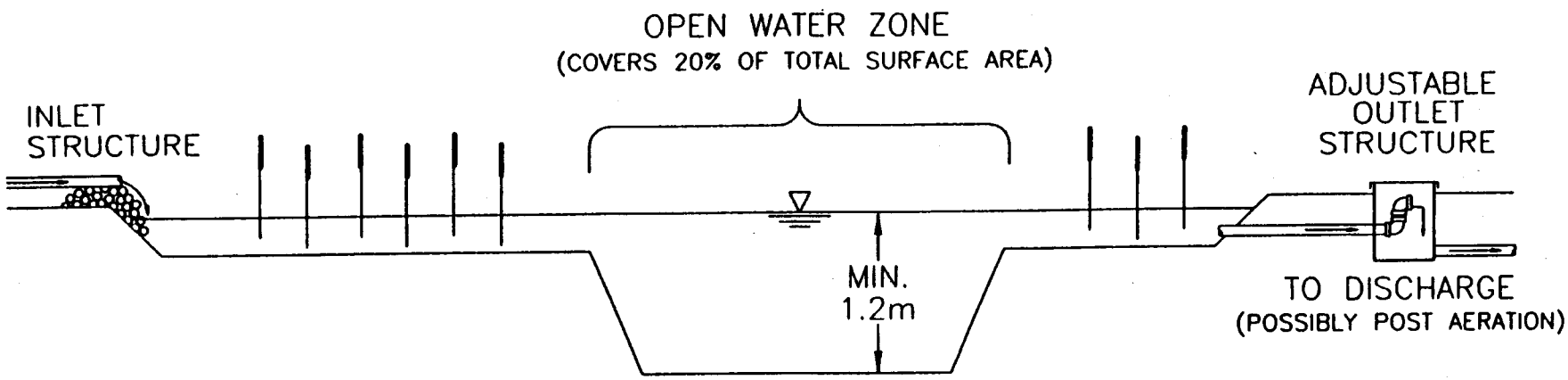
5.2 TYPES OF CONSTRUCTED WETLANDS

There are two types of constructed wetlands in use in North America and in much of the world. The first is called a "free water surface" (FWS) wetland. In this type, the water surface is exposed to the atmosphere, the bed contains emergent aquatic vegetation, a layer of soil to serve as a rooting medium, a liner if necessary to protect the groundwater, and appropriate inlet and outlet structures. The water depth in this type of wetland can range from a few centimetres to 0.5 m or more depending on the purpose of the wetland. A normal operating depth of 0.3 m is typical. In addition, open water zones may be used to enhance atmospheric reaeration and promote flow redistribution. A typical FWS wetland utilizing an open water zone is illustrated in Figure 5.2.1.

The second type of wetland is called a "subsurface flow" (SF) wetland. In this case the excavated basin is filled with a porous media, usually gravel, and the water level is maintained below the top of the gravel. The vegetation is planted in the upper part of the gravel media. A liner is also used, if necessary, to protect groundwater quality. The depth of the media is typically a little over 0.6 m and the depth of water is typically set at 0.6 m.

The use of emergent aquatic vegetation is essential on these wetlands, but the species selected for use is not especially critical and the same type of vegetation can be used on both FWS and SF wetland systems. Although the presence of the vegetation is critical, the removal of pollutants via plant uptake is not a major factor in these systems. These wetland systems can be described as an attached growth biological reactor with functions similar to those occurring in trickling filters, RBC's, and other conventional attached growth systems. The microorganisms attached to the submerged plant parts and litter in the FWS wetland and attached to the plant roots and media surfaces in the SF case provide the major treatment responses in these systems. These biological reactions are dependent on the hydraulic residence time (HRT) in the system and on temperature. Removal, via plant uptake, by the vegetation could be improved with frequent harvesting but that would complicate operations and significantly increase costs. The majority of operational wetland systems do not routinely harvest the vegetation.

FIGURE 5.2.1 - Typical FWS Wetland Utilizing an Open Water Zone



NOTE: OPEN WATER ZONES ALLOW
- FLOW REDISTRIBUTION
- ATMOSPHERIC RE-AERATION

There are several advantages to the SF wetland concept. The biological reactions in both types of wetlands are due to attached growth organisms. Since the gravel media has more surface area than the FWS wetland the gravel bed will have higher reaction rates and therefore can be smaller in area. Since the water surface is below the top of the media and not exposed, the SF type does not have mosquito problems, which can be an issue for FWS wetlands in some locations. Since the water surface is not exposed there are no public access problems and this type of wetland is often used as a component in on-site treatment/disposal systems for schools, parks, public and commercial buildings, etc. This type of wetland can also provide greater thermal protection in cold climates since the water surface is below the top of the gravel and the layer of plant litter. A technology assessment of the SF wetland concept was recently published by the US EPA (Reed, 1993).

All of these potential advantages for the SF concept may be offset by the relatively high cost to procure, deliver, and place the gravel media in the bed. The advantages may be worth the extra expense for relatively small facilities and when the extra thermal protection is essential for operation of the system. It is not possible to predict with certainty which type of wetland will be the most cost effective for a particular situation without a cost analysis because of the variables involved. However, if suitable land is available at reasonable cost the FWS wetland will probably be the more economical system to build at design flow rates higher than 400 m^3/d.

A general summary of wetland characteristics is given in Table 5.2.1.

Table 5.2.1 Wetland Characteristics

	FWS Wetland	SF Wetland
Water Depth	0.3 - 0.45 m (Section 5.5.6)	0.6 (Section 5.5.6)
Water Surface	exposed	not exposed
Length : Width Ratio	1:1 to 4:1 (defined by maximum length) (Section 5.5.5.1)	1:1 to 4:1 (defined by minimum width) (Section 5.5.5.2)
Number of cells	minimum two in parallel (Section 4.7.1)	
Maximum cell size	4-6 ha (Section 4.7.1)	
Major treatment mechanism	Attached growth media (plant, gravel and litter surface area)	
Type of media used	no media used	Coarse sand to coarse rock. Typically medium sized gravel covered with 10-15cm fine pea gravel. (Section 5.5.6)
Vegetation	cattail, bulrush, reed, rush (Section 5.6)	
Liner requirements	dependent on soil type and site investigation (Section 5.7.1)	

5.3 SITE SELECTION

The site selection requirements for a constructed wetland depend on site characteristics, such as topography, soil conditions, groundwater position, etc, as well as proximity to the lagoon being upgraded and access to a receiving water for discharge of the final effluent.

5.3.1 Topography and Soils

In the ideal case, a constructed wetland, designed for upgrading, would be located immediately adjacent to the lagoon to avoid significant pumping and transmission costs, and the effluent from the wetland could follow the same discharge pathway previously employed for the lagoon. In some cases, conflicting land uses may not permit this approach and it will be necessary to identify a more remote site for wetland construction.

A desirable constructed wetland site should have minimal topographic relief to minimize the costs and complexity of extensive cut and fill to prepare the site. A site which permits gravity flow from the lagoon and gravity flow to the point of final discharge is also advantageous. Sites with shallow bedrock or rock outcrops can also significantly increase costs of excavation and site preparation.

A uniform soil type, present over the entire site is the most desirable condition. It is essential that the soils on the bottom of the wetland be sufficiently impermeable to retain a permanent water level in the wetland for the intended treatment and support of the system vegetation. Soils which can be easily compacted to a suitable degree of impermeability are the most desirable. Highly permeable coarse sands and gravels, for example, may require expensive liners.

5.3.2 Hydrogeological Factors

A constructed wetland site location will best serve its intended treatment function and provide for protection of groundwater quality at sites where the groundwater table is permanently sufficiently deep so as not to interfere with the intended flow through the wetland. This can be concern at sites where the seasonal groundwater table approaches the ground surface. In such cases, compaction of the wetland bottom may not be sufficient and the installation of a more positive liner may be necessary.

Sites with karst topography with the potential for sinkholes or solution channels in a limestone bedrock should be avoided if possible or designed with an appropriate liner to avoid potential hydraulic failure of the wetland.

Sites located in the flood plain of rivers or streams face the possibility of periodic flooding. Protective measures, similar to those provided for a lagoon may be necessary. The degree of protection depends in part on the cost and complexity of the wetland system. A system with multiple cells, an expensive liner, and elaborate inlet and outlet structures would require the maximum protection, with exterior dikes of sufficient elevation to resist the design flood. On the other hand, replacement of some wetland components after a flood might be more economical for a small FWS wetland with a simple compacted soil bottom, and minimal flood protection efforts might be justified. Flood protection is more critical for the SF wetland type since the inorganic sediments normally carried in flood waters could clog the gravel bed. A cost comparison of partial wetland replacement versus flood protection is desirable at sites where a significant risk of flooding is present.

5.4 TOXICITY REDUCTION FACTORS

As described previously, because of the relatively long detention time in lagoon systems and the generally effective treatment provided, the wastewater constituents most likely to have toxicity implications for receiving waters are total ammonia and/or unionized ammonia, and sulphides. Sulphides can be effectively managed in a constructed wetland during the hydraulic residence time (HRT) required for ammonia removal. Therefore, ammonia removal is the limiting design parameter for design of a wetland to reduce lagoon effluent toxicity.

Ammonia can be present in lagoon effluents as free ammonia (NH_3) or as ionized ammonia (also called ammonium ions) (NH_4^+). The term ammonia will be used here to refer to total ammonia unless otherwise specified. The equilibrium between these two forms is strongly dependent on pH and temperature. At pH 7 or less, essentially only ammonium ions are present and at pH 12 only dissolved ammonia gas.

5.4.1 Influent Strength

Even though ammonia is the only toxic form of nitrogen present in lagoon effluents, it is necessary to consider all forms of nitrogen present in the lagoon effluent when designing a constructed wetland. Other forms of nitrogen present in lagoon effluents might include organic nitrogen and possibly nitrate (NO_3^-). Organic nitrogen is typically a part of the particulate matter in the lagoon effluent such as organic total suspended solids (TSS) and algae cells. The combination of organic nitrogen and ammonia is expressed as total Kjeldahl nitrogen (TKN).

The ammonia concentration in lagoon effluents can be relatively low (<2 mg/L-N) during the warm summer months due to algal uptake and to volatilization of ammonia gas from the lagoon. However it is not adequate to design the wetland for removal of just the influent ammonia, since the algal cells and other TSS entering the wetland are quickly removed and the organic nitrogen associated with those materials is mineralized and released to the water as additional ammonia. As a result is necessary to design the wetland on the assumption that all of the entering TKN can appear as ammonia.

The wetland influent strength will depend on the season of the year, the temperature of the water, the HRT in the lagoon, and the presence or absence of an ice cover on the lagoon. In the spring of the year, the effluent from long detention time facultative lagoons might contain from 5 to 35 mg/L-N TKN with about 60 percent of that in the ammonia form. In the summer months the TKN might

range from 5 to 25 mg/L-N with 10 to 30 percent of that in the ammonia form. In the fall, after low temperature conditions have eliminated the algae the TKN from long detention time facultative lagoons might range from 5 to 15 mg/L-N with 70 percent of that in the ammonia form. If winter discharge is permitted, the facultative lagoon effluent from an ice covered lagoon may have TKN ranging from 15 to 35 mg/L-N with 60 percent of that in the ammonia form.

5.4.2 Temperature

The removal of nitrogen in both SF and FWS wetland systems is temperature dependent. The biological rate constants decrease with decreasing water temperatures. The presence of an ice cover on the wetland, also the result of low temperatures, restricts oxygen transfer from the atmosphere to the water in a FWS wetland. In many conventional attached growth treatment processes such as trickling filters and RBC's the low water temperature permits an increased concentration of dissolved oxygen and this compensates for the lower biological rate constant for ammonia removal. In the FWS wetland, under an ice cover, there is no supplemental source of oxygen so the rate of nitrogen removal is significantly reduced at low temperatures. In the SF wetland, oxygen is still available on the surfaces of the roots and rhizomes within the root zone during the winter months even though the above ground portion of the plant has died in the late fall. These wetland plants have hollow stems and passageways which permit gas transfer from the above ground parts to the roots and this mechanism persists during the winter months as long as the emergent portion of the plants are exposed to the atmosphere.

5.4.3 Nitrogen Removal Processes in Wetlands

Nitrogen removal in both types of wetland systems typically follows the same three step biological process:

1. Organic nitrogen present in the wetland is decomposed and mineralized resulting in the production of additional ammonia. The decomposition of the plant litter in the wetland can also be a source of such ammonia, but after the first growing season it tends to be in rough equilibrium with plant uptake of nitrogen and is not a major factor. Plant uptake of nitrogen during the first growing season can be significant and result in optimistic conclusions regarding the removal capability of the wetland. Organic wastewater solids and algae entering the wetland are a persistent source of additional ammonia during the operational life of the system and must be considered in the system design. It is prudent to assume that all of the wastewater TKN entering the wetland can be converted to ammonia.

2. The ammonia nitrogen is then biologically oxidized to nitrate. The reaction depends on the availability of oxygen (typically 4.5 kg of oxygen per 1 kg of ammonia to be oxidized), and the reaction rate will depend on temperature. During the first year of operation, there can also be significant removal of ammonia via ion exchange on the soil at the bottom of the FWS wetland. Typically, all of the ion exchange sites are saturated during the first year of operation and this removal pathway ceases to contribute. The ion exchange sites could be restored by a frequent program of draining the wetland cell and exposing the soil surface to the atmosphere. This restores aerobic conditions at the soil surface and allows biological oxidation (nitrification) of the adsorbed ammonia. Such a procedure has not been practised to date at operational wetland systems.

3. The nitrate produced in the previous step is biologically denitrified in the anoxic zones of the wetland resulting in the production of nitrogen gas which escapes to the atmosphere. The denitrification process requires anoxic conditions and depends on the availability of a carbon source and on temperature. Anoxic conditions are generally available below the water surface in both SF and FWS wetland systems. The plant litter and other natural detritus provides the necessary carbon source so water temperature is the limiting factor for denitrification.

Since anoxic conditions are generally present throughout both wetland types (except at the water surface for FWS, and on the surfaces of the plant roots for the SF type) the conversion of ammonia to nitrate (Step 2 above) tends to be the controlling nitrogen removal step for these wetland systems, because of the limited availability of oxygen.

5.4.4 Performance Expectations

As stated previously, the purpose of the constructed wetlands described in this document is to provide for additional ammonia removal and thereby reduce potential toxicity in receiving waters. The lagoon toxicity test data discussed in Section 3.2 suggests that an 11 mg/L-N total ammonia concentration is a reasonable toxicity threshold level. Applying a safety factor of two, the design effluent ammonia concentration of 5 mg/L-N has been suggested.

Attainment of a 5 mg/L-N effluent ammonia concentration is a reasonable expectation for a constructed wetland in combination with an adequately designed lagoon system for much of coastal and interior British Columbia. For example, in a four year pilot study of a FWS constructed wetlands conducted at Listowel, Ontario, by the Ministry of the Environment, the long term average combined ammonia concentration from the wetland cells treating facultative lagoon effluent was about

4 mg/L-N, with 0.01 mg/L-N of that being unionized ammonia. This was achieved despite encountering sever oxygen transfer limitations in both summer and winter (R. Manoharan, pers. comm.).

The feasibility of a constructed wetland for a particular location will require a specific evaluation to determine the size and cost of the wetland required, and the depth of ice formation expected during the winter months. The lagoon effluent entering such a wetland, from an ice covered lagoon, during the winter months can be expected to be about 4°C in temperature. Most FWS constructed wetlands in interior and northern coastal British Columbia can also be expected to be ice covered during the coldest winter months with an average water temperature, beneath the ice, of about 2 - 3°C. Similar water temperatures can be expected in the SF wetland also. The reaction rates for ammonia removal are slow at these temperatures, so a relatively large wetland area will be required for treatment during the winter months. A very large FWS wetland with a long HRT may then be at risk of ice formation to unacceptable depths. A very large constructed wetland of either the FWS or SF type may also be uneconomical compared to other treatment alternatives. A concept in use in North Dakota, and elsewhere provides for storage in the lagoon during the most severe winter months and discharge to a constructed wetland from late spring to early fall.

5.5 DESIGN CRITERIA

This section presents the design models which can be used to estimate the size of a constructed wetland designed for removal of BOD_5 and ammonia nitrogen. Design methods for removal of other wastewater constituents can be found in other references (Reed, S.C., R.W. Crites, E.J. Middlebrooks, 1994. *Natural Systems for Waste Management and Treatment Second Edition*, McGraw Hill Book Co., New York, NY; Reed, S.C., editor, (1990). *Natural Systems for Wastewater Treatment*, MOP FD-16, WEF, Alexandria, VA). In the general case, a constructed wetland designed to produce an effluent ammonia concentration of 5 mg/L-N can be expected to have the following effluent characteristics for other wastewater constituents: BOD_5 2 - 7 mg/L, TSS 2 - 7 mg/L, Total Phosphorus 30 to 50% removal on an annual basis, Fecal Coliforms 2 - 3 log reduction.

Residual BOD_5 must be reduced to less than 15 mg/L in order to produce an environment conducive to nitrification. Therefore in situations where lagoon effluents contain a greater concentration of BOD_5 than 15 mg/L, additional hydraulic retention over and above that needed for ammonia reduction must be provided.

5.5.1 Nitrogen and BOD_5 Removal Objectives

The design procedure for ammonia nitrogen and BOD_5 removal is an iterative process. It is first necessary to assume a water temperature to calculate the size of the wetland required to achieve the desired ammonia removal during the period of concern. Having determined a tentative size it is then possible using heat transfer models to determine the theoretical water temperature in the wetland. If this theoretical water temperature and the original assumed water temperature do not agree it is necessary to repeat all of the calculations until the two values converge to within 1°C.

The same basic procedure is used for both FWS and SF wetlands although the rate constants for nitrogen and BOD_5 removal and the thermal models are different for the two types of wetlands. These design models for both ammonia removal and thermal conditions are derived from a limited database and should be used with some caution. At the present time there are no alternative temperature dependent design models available. Other design approaches, based on mass loading relationships, are available in the literature and can be used to estimate ammonia removal in a constructed wetland during the warm summer months. The ammonia removal models presented in Section 5.5.3 will produce comparable results to these alternatives for the same summer conditions.

The hydraulic design of the wetland must then be considered to ensure selection of an appropriate configuration and maintenance of desired flow conditions in the system. The hydraulics of the FWS type are controlled by Manning's equation and by Darcy's law for the SF type. A large safety factor is necessary at this stage of design because of the uncertainty involved with construction and future operation of the intended system.

5.5.2 BOD_5 Removal

Constructed wetland systems can be considered to be attached growth biological reactors, and their BOD_5 removal can be estimated with first-order plug-flow kinetics. The basic relationship for plug-flow reactors is given by Eq. 5-1.

$$\frac{C_e}{C_o} = \exp^{(-K_T t)} \tag{5-1}$$

Where:

C_e = effluent BOD_5 concentration, mg/L-N
= < 15 mg/L (requirement for nitrification)
C_o = influent BOD_5 concentration, mg/L-N
= (BOD_5 concentration leaving lagoon)
K_T = temperature dependent first order reaction rate constant, $days^{-1}$
t = hydraulic residence time, days

$$k_T = k_{20}\ (1.06)^{(T-20°C)} \tag{5-2}$$

Where:

k_{20} = rate constant at 20°C, (d^{-1})
= 1.104 d^{-1} (SF wetlands)
= 0.678 d^{-1} (FWS wetlands)

The hydraulic residence time in the wetland can be calculated with Eq. 5-3.

$$t = \frac{LWyn}{Q} \tag{5-3}$$

Where:

L = length of the wetland cell, m

W = width of the wetland cell, m

y = depth of water in the wetland cell, m

= 0.15 to 0.45 m for FWS wetlands.

= 0.30 to 0.60 m for SF wetlands.

n = porosity, or the space available for water to flow through the wetland. Vegetation and litter occupy some space in the FWS wetland and the media, roots and other solids do the same in the SF case. Porosity is a percent, expressed as a decimal.

= 0.65 to 0.75 for FWS wetlands, depending on plant density.

= 0.35 to 0.45 for SF wetlands, depending on media size, see Table 5.5.5.2

Q = the average flow through the wetland, m^3/d

$$Q = \frac{Q_{IN} + Q_{OUT}}{2} \qquad (5\text{-}4)$$

It is necessary to determine the average flow with Eq. 5-4 to compensate for water losses or gains via seepage, evapotranspiration, or precipitation as the wastewater flows through the wetland. A conservative design might assume no seepage and adopt reasonable estimates for evapotranspiration losses and rainfall gains from local records for each month of concern. This requires a preliminary assumption regarding the surface area of the wetland so the volume of water lost or added can be calculated. It is usually reasonable for a preliminary design estimate to assume that Q_{IN} equals Q_{OUT}.

It is then possible to determine the surface area of the wetland by combining Eq. 5-1 and 5-3 and rearranging terms:

$$A_s = (LW) = \frac{[Q \ \ln(\frac{C_o}{C_e})]}{K_T y n} \qquad (5\text{-}5)$$

Where:

A_s = surface area of wetland, m^2

The procedure to determine the required wetland surface area for BOD_5 reduction is as follows:

1. Assume a water temperature and solve Eq. 5-2.
2. Using the K_T from Eq. 5-2, solve Eq. 5-5 for the wetland surface area.
3. Using the calculated wetland surface area, solve the appropriate thermal equation for the water temperature. If the calculated water temperature and the original assumed value agree to within 1°C then no further iterations are required. If the two temperatures do not agree, than all of the calculations must be repeated until the two temperature values converge to within 1°C.

In designing mechanical fixed film wastewater treatment processes, the surface area for nitrification is added to that required for BOD_5 removal. This is consistent with the observation that significant nitrification does not occur until the BOD_5 concentration has been reduced to below about 15 mg/L. However, the process equations for wetland systems are not based on kinetics but rather on correlations drawn from filed data (heuristic basis), without discerning between BOD_5 and nitrification removal mechanisms. The amount of BOD_5 removal was not taken into consideration in developing equations for wetlands nitrification from the field data, consequently, the calculated surface area also accounts for BOD_5 removal. The recommended application of these equations is to calculate the required surface area using both the nitrification model (presented in the following sections) and the BOD_5 model, and use the larger value of the two.

5.5.3 Ammonia Removal Models

All constructed wetland systems can be considered to be attached growth biological reactors, and their performance can be estimated with first-order plug-flow kinetics. The basic relationship for plug-flow reactors is given by Eq. 5-6.

$$\frac{C_e}{C_o} = \exp^{(-K_T t)} \qquad (5\text{-}6)$$

Where:

C_e = effluent ammonia concentration, mg/L-N

C_o = influent ammonia concentration, mg/L-N

K_T = temperature dependent first order reaction rate constant, $days^{-1}$

t = hydraulic residence time, days

The wetland surface area can be determined by substituting the ammonia C_e, C_o, and K_T into Eq. 5-5. The value used for the rate constant K_T in Eq. 5-5 or 5-6 depends on the type of wetland (FWS or SF) under consideration and on the temperature of the water in the wetland.

Free water surface wetlands:

at:	0°C,	K_T	$= 0\ d^{-1}$
	1°C+	K_T	$= 0.2187(1.048)^{(T-20°)},\ d^{-1}$

Subsurface flow wetlands:

See Section 5.5.3.2 for derivation of the SF rate constants.

The procedure to determine the required wetland surface area for ammonia reduction is as follows:

1. Assume a water temperature and determine the ammonia K_T.
2. Using the K_T solve Eq. 5-5 for the wetland surface area.
3. Using the calculated wetland surface area, solve the appropriate thermal equation for the water temperature. If the calculated water temperature and the original assumed value agree to within 1°C then no further iterations are required. If the two temperatures do not agree, than all of the calculations must be repeated until the two temperature values converge to within 1°C.

5.5.3.1 Free Water Surface Wetlands

The recommended design model assumes that ammonia removal is entirely due to biological nitrification and no credit is given to plant uptake, since harvesting is not normally practised. The model also assumes that sufficient alkalinity (alkalinity is essential to support the nitrification reactions) is present and that the oxygen concentrations in the water are comparable to those normally present in a FWS wetland. If special measures, such as aeration, are provided to enhance the oxygen supply and nitrification then the basic design model does not apply.

The temperature dependence of the nitrification reactions in FWS wetlands is similar to that observed in conventional attached growth devices such as trickling filters and RBC units. At water temperatures of 10°C, and higher, nitrification is less dependent on temperature than BOD_5 removal;

at temperatures less than 10°C nitrification is strongly temperature dependent, and at 0°C activity ceases completely.

Other ammonia removal design models are also available in the published literature. Eq. 5-7 and 5-8 appear in the WPCF Manual of Practice FD 16 (WPCF, 1990).

$$\ln(C_e) = 0.688\ \ln(C_o) + 0.6551\ln(HLR) - 1.107 \tag{5-7}$$

Where:

HLR = hydraulic loading rate, cm/d

C_o = influent ammonia concentration, mg/L-N

C_e = effluent ammonia concentration, mg/L-N

$$A_s = \frac{(100)(Q)}{\exp^{[1.527\ln(c_e) - 1.050\ \ln(C_o) + 1.69]}} \tag{5-8}$$

Where:

A_s = surface area of wetland, m^2

Q = design flow, m^3/d

Eq. 5-9 was developed by Hammer and Knight with a regression analysis of 17 FWS systems.

$$C_e = \frac{(18.31)(C_o)(Q)}{A_s} - 0.16063 \tag{5-9}$$

Where:

A_s = surface area of wetland, m^2

Q = design flow, m^3/d

C_o = influent ammonia concentration, mg/L-N

C_e = effluent ammonia concentration, mg/L-N

Temperature adjustments are not possible with either Eq. 5-7, 5-8, or 5-9 nor do they take into account the depth of water or the HRT in the wetland. They can be used as an independent check, for warm weather conditions only, of the results from the recommended model (Eq. 5-5 and 5-6).

Eq. 5-6, and 5-7 or 5-9, will predict approximately the same effluent concentration for summer conditions and a water depth of about 0.3 m in the FWS wetland.

5.5.3.2 Subsurface Flow Wetlands

Since the water level is maintained below the media surface in SF wetlands the rate of atmospheric re-aeration is likely to be significantly less than the FWS wetland type. However, as described previously, the roots and rhizomes of the vegetation are believed to have aerobic microsites on their surfaces, and the wastewater as it flows through the bed has repeated opportunities for contact with these aerobic sites in an otherwise anaerobic environment. As a result, conditions for nitrification and denitrification are present in the same reactor. Both of these biological nitrification and denitrification reactions are temperature dependent and the rate of oxygen transfer to the plant roots may vary somewhat with the season.

The major carbon sources supporting denitrification are the dead and decaying roots and rhizomes, the other organic detritus, and the residual wastewater BOD_5. These carbon sources are probably more limited for SF wetlands, during initial operations, as compared to the FWS case since most of the plant litter collects on top of the bed. After a few years of litter build-up and decay both types of wetlands may have comparable carbon sources for support of denitrification.

Since a major source of oxygen in the SF case is the plant roots it is absolutely essential to insure that the root system penetrates to the full design depth of the bed. Any water that flows beneath the root zone is in a completely anaerobic environment and nitrification will not occur except by diffusion into the upper layers. This response is illustrated by the data in Table 5.5.3.2.1 where removal of ammonia can be directly correlated with the depth of penetration by the plant roots. The beds containing *Typha* (root penetration about 40% of the bed depth) only achieved 32 percent ammonia removal as compared to the *Scirpus* beds which achieved 94 percent removal and had complete root penetration.

Most existing SF systems in the world were designed with the assumption that regardless of the plant species selected the roots would somehow automatically grow to the bottom of the bed and supply all of the necessary oxygen. This has not occurred and many of these systems cannot meet their discharge limits for ammonia. This problem can be avoided in the future if proper care is taken during design and operation of the system. The root depths listed in Table 5.5.3.2.1 for Santee, CA probably represent the maximum potential depth for the plant species listed since Santee had a warm climate with a continuous growing season and the applied wastewater contained sufficient nutrients. This

suggests that the design depth of the bed should not be greater than the potential root depth of the plant intended for use if oxygen is needed for ammonia removal.

Table 5.5.3.2.1 Performance Comparison for Vegetated and Unvegetated SF Wetlands at Santee, CA

Bed Condition	Root Penetration, cm	Effluent Quality, mg/L		
		BOD_5	TSS	NH_3
Scirpus	76	5.3	3.7	1.5
Phragmites	> 60	22.3	7.9	5.4
Typha	30	30.4	5.5	17.7
No vegetation	0	36.4	5.6	22.1

Note: Q = 3.04 m^3/d, HRT = 6 d, bed dimensions: L = 18.5 m, W = 3.5 m, y = 0.76 m,primary wastewater influent: BOD_5 = 118 mg/L, TSS = 57 mg/L, NH_3 = 25 mg/L-N.

Operational methods for actually achieving the maximum potential root penetration will still be necessary since the plants can obtain all of the needed moisture and nutrients with the roots in a relatively shallow position. In some European systems, the water level is lowered gradually in the fall of each year to induce deep root penetration. It is claimed that three growing seasons are required to achieve full penetration by *Phragmites* using this method. Another approach, in cool climates, where winter treatment requirements typically require a larger area, is to construct the bed with three parallel cells and only operate two for a month at a time during the warm periods. The roots in the dormant cell should penetrate as the nutrients in the water are consumed. In warm climates, where freezing is not a risk, it is possible to limit the bed depth to ≤ 0.3 m which should allow rapid and complete root penetration. The volume of gravel required will be constant regardless of the bed depth, but the surface area needed to achieve the same level of treatment will increase as the depth decreases.

There is not a final consensus on how much oxygen can be furnished to the root zone in SF wetlands or on the oxygen transfer efficiency of various plant species. There is a consensus that these emergent plants transmit enough oxygen to their roots to keep alive under normal stress levels. The disagreement occurs over how much oxygen is available at the root surfaces to support biological

activity. Published estimates range from 5 to 45 g $O_2/m^2/d$ of wetland surface area. If the ammonia removals observed at Santee are assumed to be due to biological nitrification it is possible to calculate the amount of oxygen which should have been available for that purpose, since it requires about 5 g of oxygen to nitrify 1 g of ammonia. The results of these calculations are shown in Table 5.5.3.2.2.

Table 5.5.3.2.2 Potential Oxygen from Emergent Wetland Vegetation

Plant Type	Root Depth[a]	Available Oxygen	
	m	$g/m^3/d$[b]	$g/m^2/d$[c]
Scirpus (Bullrush)	0.76	7.5	5.7
Phragmites (Reeds)	0.60	8.0	4.8
Typha (Cattails)	0.30	7.0	2.1
	Average:	7.5	

a. Total depth of gravel bed 0.76 m.
b. Available oxygen per unit volume of measured root zone.
c. Available oxygen per unit surface area of 0.76 m deep bed.

The oxygen available for nitrification per unit of wetland surface area ranged from 2.1 to 5.7 $g/m^2/d$ because the depth of root penetration varied with each plant species. These oxygen values are near the low end of the published range (5 to 45 g $O_2/m^2/d$). However, the available oxygen, when expressed in terms of the actual root zone of the various plants is about the same, regardless of the species (average 7.5 g $O_2/m^3/d$). This suggests, that at least for these three species, the oxygen available for nitrification will be about the same so the rate of nitrification is therefore dependent on the depth of the root zone which is present in the SF bed. Eq. 5-10 defines this relationship.

$$K_{NH} = 0.01854 + 0.3922\,(rz)^{2.6077} \qquad (5\text{-}10)$$

Where:

K_{NH} = nitrification rate constant, at 20°C, d^{-1}

rz = percent of SF bed depth occupied by the root zone, a decimal fraction, (0 to 1)

The K_{NH} value would be 0.4107 with a fully developed root zone, and 0.01854 if there were no vegetation on the bed. These values are consistent with performance results observed at several SF sites evaluated for the U.S. EPA. Independent confirmation of this rate constant is provided by the design model published by Bavor. His model takes the same form as Eq. 5-6 with a rate constant at 20°C of 0.107 d^{-1} in a gravel bed system where the plant root zone occupied between 50 and 60 percent of the bed depth. Eq. 5-10 would predict a rate constant of 0.101 for these conditions.

Having defined the basic rate constant K_{NH} it is possible to provide appropriate adjustments for temperature:

at:	0°C,	K_T	=	$0\ d^{-1}$	(5-11)
	1°C+	K_T	=	$K_{NH}(1.048)^{(T-20°)},\ d^{-1}$	(5-12)

Interpolation can be used for temperatures between 0°C and 1°C. The K_T value determined with these equations is then used in either Eq. 5-5 or 5-6 to determine performance or size of an SF constructed wetland.

It is unacceptable to assume that the root zone will automatically occupy the entire bed volume, except for relatively shallow ≤0.3 m systems using small sized gravel (≤ 20 mm). Deep beds ≈ 0.6 m require the special measures discussed previously to induce and maintain full root penetration. If these special measures are not utilized it would be conservative to assume that the root zone occupies not more than 50 percent of the bed depth unless measurements show otherwise. It is also unlikely, based on observations at numerous operational systems that the plant roots will penetrate deeply in the large void spaces occurring when large size rock > 50 mm is selected as the bed media.

5.5.4 Thermal Considerations

As shown in the previous two sections it is necessary to assume a wetland water temperature in order to calculate the wetland surface area required to achieve the desired level of ammonia removal. It is then necessary, using appropriate heat transfer models to calculate the theoretical water temperature in the wetland, based on the previously determined surface area. It may be necessary to repeat several iterations of both these calculations until the assumed and theoretical water temperatures converge on the same value.

A conservative assumption for preliminary calculations is to assume that the water temperature in either FWS or SF wetlands is equal to the average monthly air temperature at the site during ice free

periods, but never less than 3°C. In interior British Columbia it is conservative to assume that a FWS wetland will be ice covered most of the winter and the average water temperature would be between 2°C and 3°C. A 3°C winter water temperature is also a conservative assumption for SF wetlands in interior B.C. Use of these assumptions for preliminary designs will eliminate the need for detailed thermal analysis. It will still be necessary to calculate the maximum potential depth of ice formation during the winter months to insure that a wetland is physically capable of passing the design volume of water during the winter months.

5.5.4.1 Subsurface Flow Wetlands

The actual thermal status of a SF wetland bed can be a very complex situation. There can be heat gains or losses to the underlying soil, to the wastewater flowing through the system, and to the atmosphere. Basic thermal mechanisms involved include conduction to or from the ground, conduction to or from the wastewater, conduction and convection to or from the atmosphere, and radiation to or from the atmosphere. It can be shown that energy gains or losses to the ground are a minor component and can therefore be neglected. It is conservative to ignore any energy gains from solar radiation and this is appropriate at northern sites where the temperature conditions are most critical. In the southwest, where solar radiation can be very significant on a year round basis this factor might be included in the calculations. Convection losses can be significant due to wind action on an open water surface, but this should not be the case for most SF wetlands where a dense stand of vegetation, a litter layer, and a layer of relatively dry gravel is typically present. These damp out the wind effects on the underlying water in the wetland and as a result, convection losses will be relatively minor and can be ignored in the thermal model. The simplified model developed below is therefore based only on conduction losses to the atmosphere, and is conservative.

The temperature at any point in the SF wetland can be predicted by comparing the estimated heat losses to the energy available in the system. The losses are assumed to occur via conduction to the atmosphere and the only energy source available is assumed to be the water flowing through the wetland. As water is cooled it releases energy, this energy is defined as the specific heat. The specific heat of water is the amount of energy that is either stored or released as the temperature is either increased or decreased. The specific heat is dependent on pressure and to a minor degree on temperature. Since atmospheric pressure will prevail at the water surface in these systems and since the temperature influence is minor, the specific heat is assumed to be a constant for practical purposes. For the calculations in this section the specific heat (c_p) is taken as 4215 J/kg/°C. The specific heat relationship applies down to the 0°C freezing point of water. Water at 0°C will still not freeze until the available latent heat is lost. The latent heat is also assumed to be a constant and equal

to 334944 J/kg. The latent heat is in effect the final safety factor protecting the system against freezing. However, when the temperature drops to 0°C, freezing is imminent and the system is on the verge of physical failure. To insure a conservative design, the latent heat is only included as a factor in these calculations when a determination of potential ice depth is made.

The available energy in the water flowing through the SF wetland is defined with Eq. 5-13.

$$q_G = (c_P)(\delta)(A_S)(y)(n) \tag{5-13}$$

Where:

q_G = energy gain from water, J/°C

c_P = specific heat capacity of water, 4215 J/kg/°C

δ = density of water, 1000 kg/m^3

A_S = surface area of wetland, m^2

y = depth of water in wetland, m

n = porosity of wetland (ie: space available for water to flow, the remainder is occupied by the media. See Table 5.5.5.2 for typical values.

If it is desired to calculate the daily temperature change of the water as it flows through the wetland the term A_S/t is substituted for A_S in Eq. 5-14:

$$q_G = (c_P)(\delta)(A_S/t)(y)(n) \tag{5-14}$$

Where:

q_G = energy gain during 1 day of flow, J/°C/d, (BTU/°F/d)

t = hydraulic residence time in the system, days

Other terms defined previously

The heat losses from the entire SF wetland can be defined with Eq. 5-15.

$$q_L = (T_o - T_{AIR})(U)(\sigma)(A_S)(t) \tag{5-15}$$

Where:

q_L = energy lost via conduction to the atmosphere, J

T_o = water temperature entering wetland, °C

T_{AIR} = average air temperature during period of concern, °C

U = heat transfer coefficient at the surface of the wetland bed, $W/m^2/°C$
σ = time conversion, metric: 86400 s/d.
A_S = surface area of wetland, m^2
t = hydraulic residence time in the wetland, d

If it is desired to calculate heat losses and temperatures on a daily basis Eq. 5-15 becomes:

$$q_L = (T_o - T_{AIR})(U)(\sigma)(A_S/t)(1\ \text{day}) \tag{5-16}$$

Where:
q_L = daily energy losses, J/d
T_o = water temperature entering wetland segment of concern, °C
Other terms defined previously

The T_{AIR} values in Eq. 5-15 and 5-16 are obtained from local weather records or from the closest weather station to the proposed wetland site. The year with the lowest winter temperatures during the past 20 or 30 years of record is selected as the "design year" for calculation purposes. It is desirable to use an average air temperature over a time period equal to the design HRT in the wetland for these thermal calculations. If monthly average temperatures for the "design year" are all that is available they will usually give an acceptable first approximation for calculation purposes. If the results of the thermal calculations suggest that marginally acceptable conditions will prevail then further refinements are necessary for a final system design.

The conductance or "U" value in Eq. 5-15 is the heat conducting capacity of the wetland profile. It is a combination of the thermal conductivity of each of the major components divided by it's thickness as shown in Eq. 5-15.

$$U = \frac{1}{\frac{y_1}{k_1} + \frac{y_2}{k_2} + \frac{y_3}{k_3} + \frac{y_n}{k_n}} \tag{5-17}$$

Where:
U = conductance, $W/m^2/°C$
$k_{(1-n)}$ = conductivity of layers 1 to n, W/m/°C
$y_{(1-n)}$ = thickness of layers 1 to n, m

Table 5.5.4.1 presents conductivity values for the materials typically present in SF wetlands.

Table 5.5.4.1 Thermal Conductivity of SF Wetland Components

Material	k	
	W/m/ °C	BTU/ft/ °F/hr
Air (no convection)	0.024	0.014
Snow (new, loose)	0.08	0.046
Snow (long term)	0.23	0.133
Ice (at 0°C)	2.21	1.277
Water (at 0°C)	0.58	0.335
Wetland litter layer	0.05	0.029
Dry (25% moisture) gravel	1.5	0.867
Saturated gravel	2.0	1.156
Dry soil	0.8	0.462

The conductivity values for all of the materials, except the wetland litter layer, are well established and can be found in numerous literature sources. The conductivity for a SF wetland litter layer is believed to be conservative but is less well established and should be used with some caution until further verification is available. The presence of a snow cover on the SF wetland bed can be expected during most winters in interior B.C., and this snow provides very significant thermal insulation. However, it is probably prudent, for design purposes to assume that snow is not present.

The change in temperature T_C due to the heat losses and gains defined by Eq. 5-13 and 5-15 can be found by combining the two equations:

$$T_c = \frac{q_L}{q_G} = \frac{(T_o - T_{AIR})(U)(\sigma)(A_s)(t)}{(c_p)(\delta)(A_s)(\gamma)(n)} \qquad (5\text{-}18)$$

Where:

T_C = temperature change in wetland, °C

Other terms defined previously

The effluent temperature T_e from the wetland is equal to:

$$T_e = T_o - T_c \tag{5-19}$$

Or:

$$T_e = T_o - (T_o - T_{AIR}) \frac{(U)(\sigma)(t)}{(c_p)(\delta)(y)(n)} \tag{5-20}$$

If the calculation is performed on a daily basis, the T_o is the entering water temperature to the wetland segment of concern and the T_e is the effluent temperature from that segment, and T_{AIR} is the average air temperature during the comparable time period.

The average water temperature T_w in the SF wetland is then:

$$T_w = \frac{T_o + T_e}{2} \tag{5-21}$$

This average temperature is compared to the temperature value assumed when the size and the HRT of the wetland was determined with the ammonia removal models. If the two temperatures do not closely correspond then further iterations of these calculations are necessary until the assumed and calculated temperatures converge.

Further refinement of this procedure is possible by including energy gains and losses from solar radiation and conduction to or from the ground. During the winter months conduction from the ground is likely to represent a small net gain of energy since the soil temperature is likely to be higher than the water temperature in the wetland. The energy input from the ground can be calculated with Eq. 5-15, a reasonable U value would be 0.32 W/m^2/°C and a reasonable ground temperature might be 10°C. The solar gain can be estimated by determining the net daily solar gain, for the location of interest, from appropriate records. Eq. 5-22 can then be used to estimate the heat input from this

source. The results from Eq. 5-22 should be used with caution. It is possible that much of this solar energy may not actually reach the water in the SF wetland since the radiation first impacts on the vegetation and litter layer and a possible reflective snow cover, so an adjustment is necessary in Eq. 5-22. As indicated previously, it is conservative to neglect any heat input to the wetland from these sources. If these additional heat gains are calculated they should be added to the results from Eq. 5-13 or 5-14 and this total used in the denominator of Eq. 5-18 to determine the temperature change in the system.

$$q_{SOLAR} = (\Phi)(A_s)(t)(s) \qquad (5\text{-}22)$$

Where:

q_{SOLAR} = energy gain from solar radiation, J

Φ = solar radiation for site $J/m^2/d$

A_s = surface area of wetland, m^2

t = HRT for the wetland, d

s = percentage of solar radiation energy which reaches the water in the SF wetland, typically 0.05 or less.

5.5.4.2 Free Water Surface Wetlands

Since the water surface is exposed to the atmosphere in a FWS wetland, some ice formation, at least on a temporary basis, is likely in northern locations which experience periods of sub freezing air temperatures. The presence of some ice can be a benefit in that the ice layer acts as a thermal barrier and slows the cooling rate of the water beneath. In ponds, lakes, and most rivers, the ice layer floats freely and can increase in thickness without significantly reducing the volume available for flow beneath the ice cover. In the case of the FWS wetland the ice may be held in place by the numerous stems and leaves of the vegetation so the volume available for flow can be significantly reduced as the ice layer thickens. In the extreme case, the ice layer may thicken to the point where flow is constricted, the stresses induced cause cracks in the ice and flow may commence on top of the ice layer. Freezing of that surface flow will occur and the wetland is then in a failure mode until warm weather returns. The biological treatment activity in the wetland will also cease at that point. This situation must be prevented or avoided if a constructed wetland is to be considered. In some locations, which experience very long periods of very low air temperatures (<-20°C) the solution may be to utilize a seasonal wetland component with wastewater stored in a lagoon during the extreme winter months. A number of systems in South Dakota and the Yukon operate in this mode. FWS constructed wetlands have, on the other hand, performed successfully throughout the winter months

in Ontario, Canada, and in several communities in Iowa where extreme winter temperatures are also experienced (Herskowitz, 1986; Knight et al., 1993). It is essential for each project in northern climates to conduct a thermal analysis as described in this section to insure that the wetland will be physically stable during the winter months and can sustain water temperatures which allow the biological reactions to proceed.

The calculation procedure has three parts:

1. Calculate water temperatures in the wetland until conditions (3°C water temperature) allowing ice formation commence. Separate calculations are required for densely vegetated wetland segments and for large area open water zones.

2. Water temperature calculations are then continued for the ice covered case.

3. An estimate is made of the total depth of ice which may form over the period of concern.

The temperatures determined during steps 1 and 2 are also used to determine the basic feasibility of a FWS wetland in the location under consideration and to verify the temperature assumptions made when sizing the wetland with the ammonia removal model. The ammonia removal model is the first step in the design process since these results are necessary for the determination of the wetland size, the HRT, and flow velocity which are used in the subsequent thermal calculations. The total depth of ice estimated in the third step above also provides an indication of the feasibility of a wetland in the location under consideration and is used to determine the necessary operating water depth during the winter months.

Case 1. FWS wetland, prior to ice formation

Eq. 5-23 is used to calculate the water temperature at the point of interest in the wetland. Experience has shown that ice formation commences when the bulk temperature in the liquid approaches 3°C because of density differences and convection losses at the water surface. Eq. 5-23 is therefore repeated until a temperature of 3°C is reached, or until the end of the wetland cell is reached, whichever comes first. If a temperature of 3°C is reached prior to the end of the wetland then Eq. 5-25 is used to calculate the temperatures under an ice cover. If the wetland is composed of vegetated zones interspersed with deeper open water zones, Eq. 5-23 must be sequentially used, with the appropriate heat transfer coefficient (U_s), to calculate the water temperatures.

$$T_W = T_{AIR} + (T_o - T_{AIR})\ e^{-\left[\frac{U_s(x-x_o)}{(\delta)(y)(v)(c_p)}\right]} \qquad (5\text{-}23)$$

Where:

T_W = water temperature at distance x, °C [x in m]

T_{AIR} = average air temperature during period of interest, °C

T_O = water temperature at distance x_o, the entry point for the wetland segment of interest, °C [x_o in m]

U_s = heat transfer coefficient at wetland surface, $W/m^2/°C$

U_s = 1.5 $W/m^2/°C$ for dense marsh vegetation.

U_s = 10 to 25 $W/m^2/°C$ for open water, high value for windy conditions with no snow cover.

δ = density of water, 1000 kg/m^3

y = depth of water, m

v = flow velocity in wetland, m/s

c_P = specific heat, 4215 J/kg/°C

If the first iteration shows a temperature of less than 3°C in the final effluent from the wetland, Eq. 5-23 can be rearranged and solved for the distance "x" at which the temperature becomes 3°C:

$$(x - x_o) = -\frac{(\delta)(y)(v)(c_p)}{U_s}\ \left[\ \ln\frac{(3° - T_{AIR})}{(T_o - T_{AIR}}\right] \qquad (5\text{-}24)$$

Example 5.1

Calculate the water temperatures in a three stage FWS wetland.

Stage 1: 180 m long, depth 0.3 m, densely vegetated, flow velocity 0.00103 m/s.

Stage 2: deep open water zone, 50 m long, 1.2 m deep, velocity 0.00026 m/s.

Stage 3: same as Stage 1.

T_{AIR} = -2°C, influent temperature = 10°C.

Use Eq. 5-23 to calculate temperature at end of stage 1:

$$T_W = (-2°) + (10° - [-2°])\, e^{-[1.5(180)/(1000)(0.3)(0.00103)(4215)]}$$
$$= (-2°) + (12°)(0.813)$$
$$= 7.75°C$$

The water temperature at the end of stage 2 would be:

$$T_W = (-2°) + (7.75° - [-2°])\, e^{-[15(50)/(1000)(1.2)(0.00026)(4215)]}$$
$$= (-2°) + (9.75°)(0.565)$$
$$= 3.5°C$$

The water temperature at the end of the wetland would be:

$$T_W = (-2°) + (3.5° - [-2°])\, e^{-[1.5(180)/(1000)(0.3)(0.00103)(4215)]}$$
$$= (-2°) + (5.5)(0.813)$$
$$= 2.5°C$$

Since the effluent temperature is below 3°C, ice formation would commence on the third segment of this wetland. Eq. 5-24 should be used to determine that this point would occur at about 76 m into the third wetland segment. Eq. 5-25 would then predict a final effluent water temperature of 1.5°C.

Case 2. FWS wetland, flow under an ice cover

Once an ice cover forms, the heat transfer from the underlying water to the ice proceeds at a constant rate which is not influenced by the air temperature or the presence or absence of a snow cover on top of the ice. This is because the ice surface, at the interface with the water remains at 0°C until all of the water is frozen. The rate of ice formation is influenced by the air temperature and the presence or absence of snow but the cooling rate of the underlying water is not. The wetland water temperature under an ice cover can be estimated with Eq. 5-25. This is identical in form to Eq. 5-23, with changes in two of the terms to reflect the presence of the ice cover.

$$T_W = T_M + (T_O - T_M)\, e^{-\left[\frac{U_i(x-x_o)}{(b)(y)(v)(C_p)}\right]} \qquad (5\text{-}26)$$

Where:

T_W = water temperature at distance x, °C
T_M = ice melting point, 0°C
T_o = water temperature at distance x_o, °C
= assume 3°C where an ice cover commences.
U_i = heat transfer coefficient at ice/water interface, $W/m^2/°C$
Other terms defined previously.

The "U_i" value in Eq. 5-25 is dependent on the depth of water beneath the ice and the flow velocity:

$$U_i = (\phi)\frac{(v)^{0.8}}{(y)^{0.2}} \tag{5-26}$$

Where:
U_i = heat transfer coefficient at ice/water interface, $W/m^2/°C$
ϕ = proportionality coefficient
= 1622 $J/m^{2.6}/s^{0.2}/°C$
v = flow velocity, m/s (assume same as no ice condition)
y = depth of water, m

Case 3. FWS wetland, thickness of ice formation

Ice will commence to form on the surface of the FWS wetland when the bulk water temperature reaches 3°C and will continue as long as the temperature remains at or below 0°C. In northern climates where extremely low air temperatures can persist for very long periods the FWS wetland may not be a feasible year-round treatment since extensive ice formation can result in physical failure of the system. The thickness, or depth of ice which will form over a one day period can be estimated with Eq. 5-27. It is necessary to repeat the calculation for each day of interest with appropriate adjustments in the depth of ice and snow in Eq. 5-27. The time period of concern for the previous FWS thermal models is equal to the design HRT for the wetland; in this case the time period of concern may be the entire winter season if significant periods of subfreezing temperatures persist. A reasonable first approximation of potential ice formation can be achieved by using the average monthly air temperatures (in the coldest winter of record) during the period of concern.

$$y = \frac{(t)(\sigma)}{(\delta)(\Omega)} \left[\frac{(T_M - T_{AIR})}{\frac{y_s}{k_s} + \frac{y_i}{k_i} + \frac{1}{U_s}} - U_i (T_W - T_M) \right] \quad (5\text{-}27)$$

Where:

y	=	thickness of ice formation, m
t	=	time period of concern, one day
σ	=	time conversion factor, metric: 86400 s/day.
δ	=	density of ice, 917 kg/m^3
Ω	=	latent heat, 334,944 J/kg
T_M	=	melting point of ice, 0°C
T_{AIR}	=	average air temperature during time period of concern °C
y_s	=	depth of snow cover, m.
k_s	=	conductivity of snow (from Table 5.5.4.1)
y_i	=	depth of daily ice formation, m
k_i	=	conductivity of ice (from Table 5.5.4.1)
U_s	=	heat transfer coefficient at surface,
	=	1.5 W/m^2/°C for dense marsh vegetation.
	=	10 to 25 W/m^2/°C for open water, high value for windy conditions with no snow cover.
U_i	=	Heat transfer coefficient water to ice, from Eq. 5-26
T_W	=	average water temperature during period of concern, from Eq. 5-25

The rate of ice formation will be the highest on the first day of freezing, when neither an ice cover or a snow layer are present to retard heat losses. The final term: $U_i (T_W - T_M)]$ in Eq. 5-27 is usually small and can be neglected for estimation purposes. As a result, Eq. 5-27 reduces to the Stefan formulation (see Section 4.1.5 Ice Cover for alternative version):

$$h = (m)[(T_M - T_{AIR})(t)]^{1/2} \quad (5\text{-}28)$$

Where:

h	=	depth of ice which will form over time period t, m
T_M	=	freezing point of ice, 0°C
T_{AIR}	=	average air temperature during time period t, °C

t = number of days in the period of interest, d

m = proportionality coefficient $m/°C^{½}/d^{½}$

= 0.027 $m/°C^{½}/d^{½}$ for open water zones, no snow

= 0.018 $m/°C^{½}/d^{½}$ for open water zones, with snow

= 0.010 $m/°C^{½}/d^{½}$ for a wetland with dense vegetation and litter

Eq. 5-28 can be used to estimate of total ice formation on FWS wetlands over the entire winter season. This equation can be used to determine the feasibility of winter operations for a wetland in locations with very low winter temperatures. For example, a site with persistent air temperatures at -25°C would result in a wetland 0.46 m deep freezing to the bottom in about 84 days.

The term $(T_M - T_{AIR})(t)$ is known as the freezing index, or degree-days below 0.0°C, and is an environmental characteristic for a particular location, values can be found in published references (Canadian Climate Normals, Volume 4, Degree Days, 1951-1980; Environment Canada, Atmospheric Environment Service).

5.5.4.3 Summary

If the thermal models for either SF or FWS wetlands predict sustained internal water temperature of less than 1°C a wetland may not be physically capable of winter operations at the site under consideration at the design hydraulic residence time (HRT). Ammonia removal will be negligible at those temperatures. Similarly, if Eq. 5-28 predicts a seasonal ice thickness which is greater than about 75 percent of the design depth of a FWS wetland the use of a wetland during the winter months may be questionable. It may be possible to increase the operating depth in these cases as long as the desired treatment results can still be achieved at <3°C and beneath an ice cover which will further impede oxygen transfer for nitrogen removal.

5.5.5 Hydraulic Design Procedures

The hydraulic design of constructed wetland systems is critical to their successful performance. All of the design models in current use assume uniform flow conditions and unrestricted opportunities for contact between the wastewater constituents and the organisms responsible for treatment. In the SF wetland concept it is also necessary to insure that subsurface flow conditions are maintained under normal circumstances for the design life of the system. These assumptions and goals can only be realized through careful attention to the hydraulic design and to proper construction methods.

Flow through wetland systems must overcome the frictional resistance in the system, this resistance is imposed by the vegetation and litter layer in the FWS type, the media, the plant roots and accumulated solids in the SF type. The energy to overcome this resistance is provided by the head differential between the inlet and the outlet of the wetland. Some of this differential can be provided by constructing the wetland with a sloping bottom. However, it is neither cost effective or prudent to depend on just a sloping bottom for the head differential required, since the resistance to flow may increase with time but the bottom slope is fixed for the life of the system. The preferred approach is to construct the bottom with sufficient slope to insure complete drainage when necessary and to provide an outlet which permits adjustment of the water level at the end of the wetland. This adjustment can then be used to set whatever water surface slope is required and in the lowest position used to drain the wetland. Details on these adjustable outlets are shown in Section 5.7.3.

The aspect ratio (L:W) selected for the wetland strongly influences the hydraulic regime and the resistance to flow in the system. In the design of some early FWS systems it was thought that a very high aspect ratio was necessary to insure plug flow conditions in the wetland and to avoid short circuiting and aspect ratios of at least 10:1 were recommended. A major problem with this approach is that the resistance to flow increases as the length of the flow path increases. A FWS system constructed in California with an aspect ratio of about 20:1 experienced overflow at the head of the wetland after a few years because of the increasing flow resistance from the accumulating vegetative litter. Aspect ratios from less than 1:1, up to about 3 or 4 to 1 are acceptable. Short circuiting can be minimized by careful construction and maintenance of the wetland bottom, by use of multiple cells, and with intermediate open water zones for flow redistribution.

5.5.5.1 Free Water Surface Wetlands

The flow of water in a FWS wetland is described by the Manning equation which defines flow in open channels. The flow velocity in the wetland, as described by Eq. 5-29 is dependent on the depth of flow, the slope of the water surface, and on the vegetation density. Other applications of the Manning equation to open channel flow assume that the frictional resistance occurs only at the bottom and the sides of the channel. In these FWS constructed wetlands the resistance is distributed over the entire depth of the water column because the emergent plants and litter are present in that entire space. The Manning equation also assumes turbulent flow, which is not completely valid for wetlands but is an acceptable approximation.

$$v = \frac{1}{n} y^{\frac{2}{3}} s^{\frac{1}{2}} \qquad (5\text{-}29)$$

Where:

v = flow velocity, m/s

n = Manning's coefficient, $s/m^{1/3}$

y = depth of water in the wetland, m

s = hydraulic gradient, or slope of the water surface, m/m

In wetland systems, the Manning's number (n) is a function of the depth of the water because of the resistance imposed by the emergent vegetation. The resistance, in turn depends on the density of the vegetation and the litter which can vary with the location and the season. The relationship is defined by Eq. 5-30.

$$n = \frac{a}{y^{1/2}} \tag{5-30}$$

Where:

a = resistance factor, $s \cdot m^{1/6}$

a = 0.4 $s \cdot m^{1/6}$ for sparse, low standing vegetation, $y > 0.4$ m

= 1.6 $s \cdot m^{1/6}$ for moderately dense vegetation in a wastewater wetland with $y \approx 0.3$ m.

= 6.4 $s \cdot m^{1/6}$ for very dense vegetation and very dense litter layer, in wastewater wetland with $y \leq 0.3$ m.

In most situations, with typical emergent vegetation, it is acceptable to assume for design purposes that the resistance factor value lies between 1 and 4. Substitution of Eq. 5-30 in Eq. 5-31 produces:

$$v = \frac{y^{1/2}}{a} y^{2/3} s^{1/2}$$

or:

$$v = \frac{1}{a} y^{7/6} s^{1/2} \tag{5-31}$$

It is then possible by substitution and rearrangement of terms to develop an equation for determining the absolute maximum length of the wetland cell.

Since:

$$v = \frac{Q}{Wy}, \qquad W = \frac{A_s}{L}, \qquad and \quad s = \frac{(m)(y)}{L}$$

Where:

Q = flow, m^3/d

W = width of the wetland cell, m

A_s = surface area of wetland, m^2

L = length of wetland cell, m

m = increment of depth serving as head differential, % as a decimal

v = apparent Manning velocity, m/s. Does not include porosity.

Substitution of these terms in Eq. 5-31, and rearrangement of terms produces:

$$L = [\frac{(A_s)(y)^{8/3}(m)^{1/2}(86400)}{(a)(Q)}]^{2/3}$$

or:

$$L = [\frac{(A_s)(y)^{2.667}(m)^{0.5}(86400)}{(a)(Q)}]^{0.6667} \qquad (5\text{-}32)$$

The surface area of the wetland (A_s) is first determined with the ammonia removal design model presented in an earlier section. Eq. 5-32 then allows the direct calculation of the maximum acceptable cell length which is compatible with the selected hydraulic gradient. It is desirable to use the smallest possible hydraulic gradient to provide a reserve for future adjustments if necessary. An aspect ratio (L:W) of ≤3:1 will usually be the most cost effective selection, but other combinations of length and hydraulic gradient are also feasible, in order to accommodate the wetland to the constraints of site topography. The m value used for design in Eq. 5-32 will typically be between 10 and 30 percent of the potential head available. The maximum potential head is equal to the full water depth (y) of the wetland when m = 100 percent. This is not a reasonable design configuration since the wetland would be dry at the effluent end, and there would be no reserve capacity if the flow resistance in the wetland were to increase further.

The Q value in Eq. 5-32 is the average flow through the wetland [$(Q_{IN} + Q_{OUT})/2$] to take into account water losses or gains due to evapotranspiration, seepage, and precipitation. It is usually acceptable for a preliminary design to assume that $Q_{IN} = Q_{OUT}$. The final system design should take into account water losses and gains due to evapotranspiration, seepage, and precipitation.

5.5.5.2 Subsurface Flow Wetlands

Darcy's Law as defined by Eq. 5-33 describes the flow regime in a porous media and is generally accepted for design of SF wetlands using soils and gravels as the bed media. A higher level of turbulent flow may occur in beds using very coarse rock and Ergun's equation is more appropriate for that case. Darcy's law is not strictly applicable to subsurface flow wetlands because of physical limitations in the actual system. It assumes laminar flow conditions, but turbulent flow may occur in very coarse gravels when the design utilizes a high hydraulic gradient. Darcy's Law also assumes that the flow in the system is constant and uniform, but in the actual case the flow may vary due to precipitation, evaporation and seepage; and local short circuiting of flow may occur due to unequal porosity or poor construction. If small to moderate sized gravel is used as the media, if the system is properly constructed to minimize short circuiting, if the system is designed to depend on a minimal hydraulic gradient and the gains and losses of water are recognized, then Darcy's Law can provide a reasonable approximation of the hydraulic conditions in a SF wetland.

$$v = k_s s$$

Since:

$$v = \frac{Q}{Wy}$$

Then:

$$Q = k_s A_c s \qquad (5\text{-}33)$$

Where:

Q = average flow through the wetland, m^3/d
= $[(Q_{IN} + Q_{OUT})/2]$

k_s = hydraulic conductivity of a unit area of the wetland perpendicular to the flow direction, $m^3/m^2/d$

A_c = total cross-sectional area, perpendicular to flow, m^2

s = hydraulic gradient, or "slope" of the water surface in the flow system, m/m

v = "Darcy's" velocity, the apparent flow velocity through the entire cross-sectional area of the bed, m/d. Does not include porosity.

It is possible by substitution and rearrangement of terms to develop an equation for determining the acceptable minimum width of the SF wetland cell which is compatible with the hydraulic gradient selected for design.

Since:

$$s = \frac{(m)(y)}{L}, \qquad L = \frac{A_s}{W}, \qquad and \quad A_c = (W)(y)$$

Where:

W = width of the wetland cell, m

A_s = surface area of wetland, m^2

L = length of wetland cell, m

m = increment of depth serving as head differential, % as a decimal

y = depth of water in the wetland, m

$$W = \frac{1}{y} \left[\frac{(Q)(A_s)}{(m)(k_s)} \right]^{0.5} \tag{5-34}$$

The surface area of the wetland (A_s) is first determined with the ammonia removal design model. Eq. 5-34 then allows the direct calculation of the absolute minimum acceptable cell width which is compatible with the selected hydraulic gradient. Other combinations of width and hydraulic gradient are also possible if topographic constraints exist at the proposed site. The m value in Eq. 5-34 will typically be between 5 and 20 percent of the potential head available The maximum potential head is equal to the full water depth (y) of the wetland when m = 100 percent. That is not a reasonable design configuration since the wetland would be dry at the effluent end, and there would be no reserve capacity if the flow resistance in the wetland were to increase further. It is strongly recommended that a value ≤ 1/3 of the effective hydraulic conductivity (k_s) be selected and that m not be more than 20 percent to provide a large safety factor against potential clogging, viscosity effects, and other contingencies unknown at the time of design.

Eq. 5-33 and 5-34 are valid as long as the flow through the void spaces in the media is laminar, which is the case when the Reynold's number is less than 10. The Reynold's number is a function of the flow velocity, the size of the void spaces and the kinematic viscosity of water as shown by Eq. 5-35. For most cases N_R will be much less than 1 so laminar flow will prevail and Darcy's law is valid. If turbulent flow conditions exist then the effective hydraulic conductivity will be significantly less than predicted by Darcy's law.

$$N_R = (v)(D)/\tau \tag{5-35}$$

Where:

N_R = Reynold's number, dimensionless

v = Darcy's velocity (from Eq. 5-33), m/s

D = diameter of void spaces in media, taken as equal to average size of the media, m

τ = kinematic viscosity of water, m^2/s (See published references)

The hydraulic conductivity (k_s) in Eq. 5-33 and 5-34 varies directly with the viscosity of the water, which in turn is a function of water temperature.

$$\frac{k_{sT}}{k_{s20}} = \frac{\mu_{20}}{\mu_T} \qquad (5\text{-}36)$$

Where:

k_s = hydraulic conductivity at temperatures T and 20°C

μ = viscosity of water at temperatures T and 20°C, see published references.

In cold climates, with a SF wetland operating during the winter months, the viscosity effects can be significant. For example, the hydraulic conductivity at a water temperature of 5°C would be 66 percent of that at 20°C. This effect is already taken into account in the previously recommended safety factor (design $k_s \leq 1/3$ of measured "effective" k_s).

Table 5.5.5.2 presents estimates of porosity and hydraulic conductivity for a range of materials with potential for use in SF wetlands. These values should only be used for very preliminary estimates. Prior to the final design of a major project both values should be measured for the actual material intended for use.

Experience has shown it is detrimental for performance of the SF wetland to use very coarse rock with large void spaces. Apparently the plant roots will not grow into these large void spaces and a permanently shallow root zone results. It is preferable to use a smaller size gravel to provide the necessary support for the plant roots; the smaller gravel will have a lower hydraulic conductivity but this can be accommodated by the design. A number of successful systems in the U.S. have been constructed with gravel ranging from 20 to 30 mm in size. It is not necessary to insist on a precise gradation as this will only increase the cost of the project. Any clean, washed gravel smaller than 40 mm is suitable, concrete aggregate is a typical example. Once an available gravel is identified, the porosity and hydraulic conductivity should both be measured prior to final design.

Table 5.5.5.2 Typical Media Characteristics for SF Wetlands

Media Type	Effective D_{10}, mm	Porosity, n %	Hydraulic Conductivity, k_s $m^3/m^2/d$
Coarse sand	2	28 - 32	100 - 1,000
Gravelly sand	8	30 - 35	500 - 5,000
Fine gravel	16	35 - 38	1,000 - 5,000
Medium gravel	32	36 - 40	10,000 - 50,000
Coarse rock	128	38 - 45	50,000 - 250,000

5.5.6 Water Depth

The water depth in FWS wetlands has ranged from 0.15 m to 1.0 m. The shallow depth increases the potential for oxygen transfer and improved ammonia removal but also puts the wetland at risk of freezing during the winter months. The typical vegetation used in FWS wetlands can withstand a depth of 1.0 m for temporary periods but a design for a permanent depth that great is not recommended. The typical average depth used in FWS wetlands is 0.3 to 0.45 m.

It is strongly recommended, as described in Section 5.7.3 that an adjustable outlet structure be used so that the system operator has direct control over the water depth in the wetland at all times. It may be possible to operate the system with a shallow depth of water in the summer months and increase the depth prior to the onset of winter to insure protection against freezing.

The average water depth in most SF system is about 0.6 m on a year round basis. The bed typically contains 0.6 m of medium sized gravel with about 10 to 15 cm of fine pea gravel on top. It is absolutely essential for all SF systems to provide an adjustable outlet, as described in Section 5.7.3 so the system operator can maintain the desired depth and insure subsurface flow conditions. Some surface flow may temporarily occur during extreme storm events but the intended subsurface flow can be maintained at all other times. A few SF systems have been designed with a shallow gravel bed (≈ 0.3 m) and a corresponding water level. All of these have been in warm climates and might be at risk of freezing in most of British Columbia.

5.5.7 Detention Time

Detention time or hydraulic residence time (HRT) is one of the critical variables controlling performance in constructed wetland systems. The HRT in these wetland systems is a function of surface area, water depth and the porosity of the wetland matrix. It is possible to increase the detention time in the system by increasing the depth of water. There will be a time lag involved for the necessary organisms to populate the newly submerged portion of the wetland. In FWS systems the water depth can be increased in the fall of the year, prior to the onset of low temperatures, this will increase the HRT and compensate somewhat for the lower reaction rates during winter operation. The practical upper limit for long term water depth in a FWS wetland is about 0.45 m.

Based on the monthly design temperature conditions and the expected flow and ammonia concentrations the wetland is sized to provide the desired treatment during the coldest month of the year. Similar calculations for the remainder of the year will indicate the HRT required to provide acceptable treatment and the water depth can be adjusted accordingly.

There is less flexibility with SF wetland cells since they are typically operated at maximum depth year-round to insure contact with the root zone of the plants. See Section 5.7.1 for a discussion of multiple cell SF systems as a method to insure operational flexibility and plant root development.

Eq. 5-6 will typically require an HRT of between 3 and 4 days to produce an effluent ammonia nitrogen of 10 mg/L-N under summer conditions in a SF wetland, with a fully developed root zone, and about 13 days with a winter water temperature of 3°C. A 13 day HRT for a SF wetland would probably not be cost effective except for relatively small systems (< 200 m^3/d). An effluent ammonia concentration of less than 5 mg/L-N might require an HRT of 6 to 7 days in the summer, and would in effect double the size of the wetland. At 50 percent root penetration in the SF bed the HRT would have to be about 15 days to produce an effluent with 10 mg/L-N ammonia. This illustrates the importance of insuring full depth root penetration in these SF wetlands.

Eq. 5-6 would typically require an HRT of 5 to 6 days to produce an effluent ammonia nitrogen of 10 mg/L-N under summer conditions in a FWS wetland, and theoretically about 30 days during the winter months with an under ice water temperature of about 3°C. A 30 day HRT in a FWS wetland in a very cold climate might put the system at risk of freezing. This suggests that winter storage and warm weather FWS wetland treatment might be the desirable system configuration in interior British Columbia.

5.6 VEGETATION

The presence of vegetation is absolutely essential for both SF and FWS wetland systems but the type of vegetation is less critical, except for insuring adequate root penetration in the SF case. A wide variety of aquatic plants have been used in wetland systems designed for wastewater treatment. The emergent aquatic macrophytes are the most commonly found species in the marsh type constructed wetlands used for wastewater treatment. The most frequently used are cattails (*Typha spp.*), reeds (*Phragmites communis*), rushes (*Juncus spp.*), bulrushes (*Scirpus spp.*), and sedges (*Carex spp.*). Bulrush and cattails, or a combination of the two, are the dominant species on most of the constructed wetlands in North America. There are a few systems with *Phragmites* in North America, but this species is the dominant type selected for the constructed wetlands in Europe. Systems which are specifically designed for habitat values in addition to treatment usually select a greater variety of plants with an emphasis on food and nesting values for birds and other aquatic life.

5.6.1 Types

Information on some typical plant species common in North America, and a discussion of advantages and disadvantages for use in a constructed wetland is provided in the following subsections.

5.6.1.1 Cattail

Typical varieties: *Typha angustifolia*: narrow leaf cattail, *Typha latifolia*: broad leaf cattail. Distribution: world wide. Optimum pH: 4 to 10. Salinity tolerance: narrow leaf: 15-30 ppt, broad leaf: <1 ppt. Growth: rapid, via rhizomes, spreads laterally, dense cover in <1 yr with plant spacing 0.6 m (2 ft). Relatively shallow root penetration in gravel ≈ 0.3 m (1 ft). Annual yield: ≈ 30 (dw) mt/ha (14 ton/ac). Tissue: (dw basis) ≈ 45% C, 14% N; 2% P; 30% solids. Habitat values: seeds and roots a food source for water birds, muskrat, nutria, and beaver; nesting cover for birds. Hydroperiod: can be permanently inundated > 0.3 m (1 ft), can also tolerate drought. Commonly used on many FWS and SF wetlands in North America. The relatively shallow root penetration not desirable for SF systems without adjustment in design depth of bed.

5.6.1.2 Bulrush

Typical *varieties: Scirpus acutus*: hardstem bulrush, common tule, *Scirpus cypernius*: wool grass, *Scirpus fluviatilis*: river bulrush, *Scirpus robustus*: alkali bulrush, *Scirpus validus*: soft stem bulrush, *scirpus lacustris*: bulrush.Distribution: world wide. Optimum pH: 4 to 9. Salinity tolerance: hardstem,

wool grass, river, soft stem: 0 to 5 ppt, alkali, Olney's, 25 ppt. Growth: Alkali bulrush, wool grass, river bulrush: moderate, dense cover in 1 yr with plant spacing 0.3 m (1 ft), all others moderate to rapid, dense cover in 1 yr with plant spacing 0.3 to 0.6 m (1 to 2 ft). Deep root penetration in gravel ≈ 0.6 m (2 ft). Annual yield: ≈ 20 (dw) mt/ha (18 ton/ac). Tissue: (dw basis) ≈ 18% N, 2% P; 30% solids. Habitat values: seeds and rhizomes a food source for many water birds, muskrat, nutria, and fish; nesting area for fish when inundated. Hydroperiod: can be permanently inundated, hardstem up to 1 m (3 ft), most others 0.15 to 0.3 m (0.5 to 1 ft), some can tolerate drought conditions. Commonly used on many SF constructed wetlands in North America.

5.6.1.3 Reeds

Typical varieties: *Phragmites australis*: common reed, wild reed Distribution: world wide. Optimum pH: 2-8. Salinity tolerance < 45 ppt. Growth: very rapid, via rhizomes, lateral spread ≈ 1 m/yr (3 ft/yr), very dense cover in 1 yr with plants spaced at 0.6 m (2 ft). Deep root penetration in gravel ≈ 0.4 m (1.5 ft). Annual yield: ≈ 40 (dw) mt/ha (36 ton/ac). Tissue: (dw basis) ≈ 45% C, 20% N, 2% P; 40% solids. Habitat values: low food value for most birds and animals, some value as nesting cover for birds and animals. Hydroperiod: can be permanently inundated, up to ≈ 1 m (3 ft), also very drought resistant. Considered by some to be an invasive pest species in natural wetlands in North America. Very successful utilization at constructed wastewater treatment wetlands designed for wastewater treatment. The dominant species used for this purpose in Europe. Because of low food value, this species is not subject to the damage caused by muskrat and nutria which has occurred in constructed wetlands supporting other plant species.

5.6.1.4 Rushes

Typical varieties: *Juncus articulatus:* jointed rush, *Juncus balticus:* Baltic rush, *Juncus effusus:* soft rush. Distribution: world wide. Optimum pH 5-7.5. Salinity tolerance: 0 to <25 ppt depending on type. Growth: very slow, via rhizomes, lateral spread <0.1 m/yr (<0.3 ft/yr), dense cover in 1 year with plants spaced at 0.15 m (0.5 ft). Annual yield: 50 (dw) mt/ha (45 ton/ac). Tissue: (dw basis) ≈ 15% N, 2% P, 50% solids. Habitat values: food for many bird species, roots food for muskrat. Hydroperiod: some types can sustain permanent inundation up to <0.3m (1 ft), prefers dry-down periods. Other plants better suited as the major species for wastewater wetlands, rushes are well suited as a peripheral planting for habitat enhancement in FWS wetlands **sedges** Typical varieties: *Carex aquatilis:* water sedge, *Carex lacustris*: lake sedge, *Carex stricata*: tussock sedge. Species distribution: world wide. Optimum ph 5-7.5. Salinity tolerance: <0.5 ppt. Growth: moderate to slow, via rhizomes, lateral spread <0.15 m/yr (0.5 ft/yr), dense cover in 1 year with plants spaced at 0.15 m

(0.5 ft). Annual yield: <5 (dw) mt/ha (<4ton/ac). Tissue: (dw basis) 1% N, 0.1% P; 50% solids. Habitat values: food source for numerous birds and moose. Hydroperiod: some types can sustain permanent inundation, others need a dry-down period. Other plants are better suited as the major species for wastewater wetlands, sedges are well suited as a peripheral planting for habitat enhancement in FWS wetlands.

5.6.2 Establishment

Establishing vegetation at an appropriate density is a critical requirement for construction of both types of wetland systems. Local plants are already adapted to the regional environment and are preferred, if available. A number of commercial nurseries are also capable of providing the plant stock for large projects. Planting densities are discussed in the previous section; the closer the initial spacing the sooner the system will be at full density. Most of the species will propagate from seed, and aerial seeding might be considered for large-scale projects. Plant development from seed takes a significant time, requires very careful water control, and seed consumption by birds can be a problem. The quickest and most reliable approach is to transplant rhizomes of the vegetation of choice in the prepared treatment bed.

Each rhizome cutting should have at least one bud or preferably a growing shoot, and is planted with one end about 4 cm below the surface of the soil or gravel media and the bud or shoot is exposed to the atmosphere, above the saturated media. Planting of seeds or rhizomes can occur in the spring after the last frost; rhizome material can also be planted in the fall. The bed is flooded and the water level maintained at the soil or media surface for at least six weeks or until significant new growth has developed and emerged. At this stage the wetland can be placed in full operation as long as the water level is not above the tops of the new plant growth. If fresh water is used during the incubation period the use of some supplemental fertilizer is desirable to accelerate plant growth.

Paridon Horticultural (1979) Ltd. of Delta, BC (Tel # 596-3422) is a potential local supplier of wetland plants. Paridon carries stocks some of the cattail species; other species could be obtained on a contract basis. Kester's Wild Game Nurseries of Wisconsin, U.S.A. (Tel # 414-685-2929), supplies a wide range of wetland plants.

5.7 PHYSICAL DESIGN AND CONSTRUCTION CONSIDERATIONS

A major consideration in the physical design of wetlands is the determination of how many individual treatment cells will be provided in the system. Another unique feature is an adjustable outlet for control of the water level. The remaining physical design features, (ie: dikes, liners, etc.) are similar to general practice for lagoons.

5.7.1 System Configuration

The total surface area required for the wetland is determined with the appropriate ammonia removal models presented in an earlier section. The issue of concern is then the division of that total area into an acceptable number of treatment cells. An excessive number of cells will unnecessarily increase project costs because of the dike construction required. On the other hand, a single very large cell makes operation and maintenance difficult and risks short circuiting of flow and resulting poor performance. Except for very small systems, it is recommended that the constructed wetland be divided into at least two parallel cells. In many cases, with SF wetlands, division into three parallel cells is desirable to permit alternating flow and enhanced root propagation as described in Section 5.8.1. The minimum cell length or width is determined by Darcy's law or the Manning equation as described previously in Section 5.5.5. As a "rule-of-thumb" it is desirable for optimum flow control in very large systems to divide each of the parallel flow cells into subcells in series so that any single subcell is not larger than 4 to 6 hectares.

The basic construction requirements for these wetlands include a subsurface flow barrier, selection and placement of the bed media for the SF type, establishment of vegetation and the inlet and outlet works. Pumping stations, disinfection facilities, and transmission piping may also be needed, but these are not unique to wetlands and are covered in other references. Since the dissolved oxygen levels in both SF and FWS wetland effluent tend to be low some form of post aeration may be necessary. Where topography permits, a rip rap covered cascade will serve adequately for this purpose.

Some constructed wetlands incorporate deep water zones for enhanced atmospheric reaeration and/or for enhanced habitat values. These deep water zones should be at least 1.2 m deep to prevent colonization by the emergent plant species growing in the remainder of the wetland. These open water zones must also be large enough so that the normal wind action can move duckweed and other floating plants to the edges. In one case, these zones were constructed as relatively narrow trenches perpendicular to the flow direction. The resulting open water surface was too narrow for the wind to have any effect and a persistent mat of duckweed sealed the surface and prevented any significant

reaeration. If open water zones are used they should be at least 20 percent of the cell area and be wide enough to permit effective wind action. Small patch like open water zones will only increase the risk of flow short circuiting in the system. The placement of nesting islands in these open water zones must also be given careful consideration to avoid flow short circuiting.

Both types of wetlands typically require a compacted bottom or some type of impermeable barrier to insure containment of wastewater and to prevent contamination of groundwater. In some cases this may be provided if clay is naturally present or if in situ soils can be compacted to a nearly impermeable state. Chemical treatments, a bentonite layer, asphalt or membrane liners are also possibilities. In the case of a wetland treating landfill leachate a double liner with leak detection may be required by some regulatory agencies.

The bottom of the wetland must be very carefully graded prior to placement of the liner. Topsoil can be stripped from the site and reserved for use as the planting medium for FWS wetlands or for other uses. The bottom surface must be level from side to side for the entire length of the wetland bed. Both types of wetlands may have a slight uniform slope (in the direction of flow) to ensure drainage, but as described previously the bottom slope should not be designed to provide the necessary hydraulic gradient for flow in the system.

The bottom of the wetland, during the final grading operations should be compacted to a degree similar to that used for highway subgrades. The purpose is to maintain the design surface during subsequent construction activities. Several constructed wetland systems, both SF and FWS types have been found with significant flow short circuiting due to inadequate grade control during system construction. A particular concern for the SF type is access by the trucks delivering the gravel media. The ruts from just a few of these vehicles can induce permanent short circuiting in the completed system. Construction traffic should not be permitted on the cell bottom during wet weather conditions.

The membrane liner, if used, is placed directly on the completed cell bottom. The SF media can be placed directly on heavy duty liner materials. In the case of FWS wetlands a layer of reserved topsoil is placed on top of the liner to serve as the rooting medium for the vegetation.

Selection of SF media type is critical to the successful performance of the system. Unwashed crushed stone has been used on a large number of existing projects. Truck delivery of such material during construction can lead to problems due to segregation of fines in the truck during transit, and then deposition of all of the fine material in a single spot when the load is dumped. This can result in a

number of small blockages in the flow path and internal short circuiting in the system. Washed stone or gravel is preferred. Coarse aggregates for concrete construction are commonly available and would be suitable for construction of SF wetland systems.

The dikes and berms for the wetland cells are constructed in the same manner as those for lagoons and similar water impoundments. The top of the dike, for large scale systems should be wide enough for small trucks and maintenance equipment. Each cell in the system must have a ramp into the cell to permit access for maintenance vehicles.

5.7.2 Distribution System

Both SF and FWS wetlands require uniform flow conditions through each cell to achieve the expected performance. This can be achieved in small to moderate sized systems with perforated manifold pipes extending the full width of the cell, for both inlet and outlets. An above surface inlet manifold provides access for adjustment and control and is preferred for most systems. This manifold typically consists of suitable sized plastic pipe 100 to 200 mm with a "tee" placed in the line every 3 m. These "tees" are attached to the line with "O" rings and are adjustable. The operator can move each "tee" through a vertical arc and thereby visually adjust and equalize the flow from each. These surface discharging manifolds may require thermal protection when used in cold climates. Small on-site SF systems typically have a perforated manifold placed on the bottom of the bed in a coarse rock envelope. The effluent manifold for both SF and FWS systems is typically a perforated pipe laid on the bottom of the bed at the outlet end of the cell. In some cases this outlet manifold is laid in a shallow rock filled trench slightly below the bottom of the wetland cell to allow for complete drainage.

Larger systems typically use concrete inlet and outlet structures containing a weir or similar device; in the case of the outlet structure, the weir, or stop logs, or similar control devices, should be adjustable to permit control of the water level in the cell or in each cell for cells in series. These structures should not be spaced more than 20 m apart, a 10 m spacing will provide excellent distribution and uniform collection of the liquid. The cell bottom should be about 2 m below the "normal" water surface in a zone around each of these outlet structures. This will maintain open water in the immediate vicinity of the weir and minimize clogging. The outlet structure should have a baffle plate upstream of the adjustable weir plate to prevent clogging by floating debris.

5.7.3 Outlet Structure

If weirs are used in the final FWS outlet structure or as the transfer structure between FWS cells in series, the weir plate should be adjustable to permit control of the water level in the cell and to allow drainage of the cell if necessary. Perforated manifold pipes, at the bottom of the gravel bed are typically used as for outlet and inter cell transfer in SF wetlands. These manifolds are then connected to an adjustable outlet in an adjacent structure, and this outlet is used to control the water level in the bed. Figure 5.7.3.1 illustrates this type of outlet.

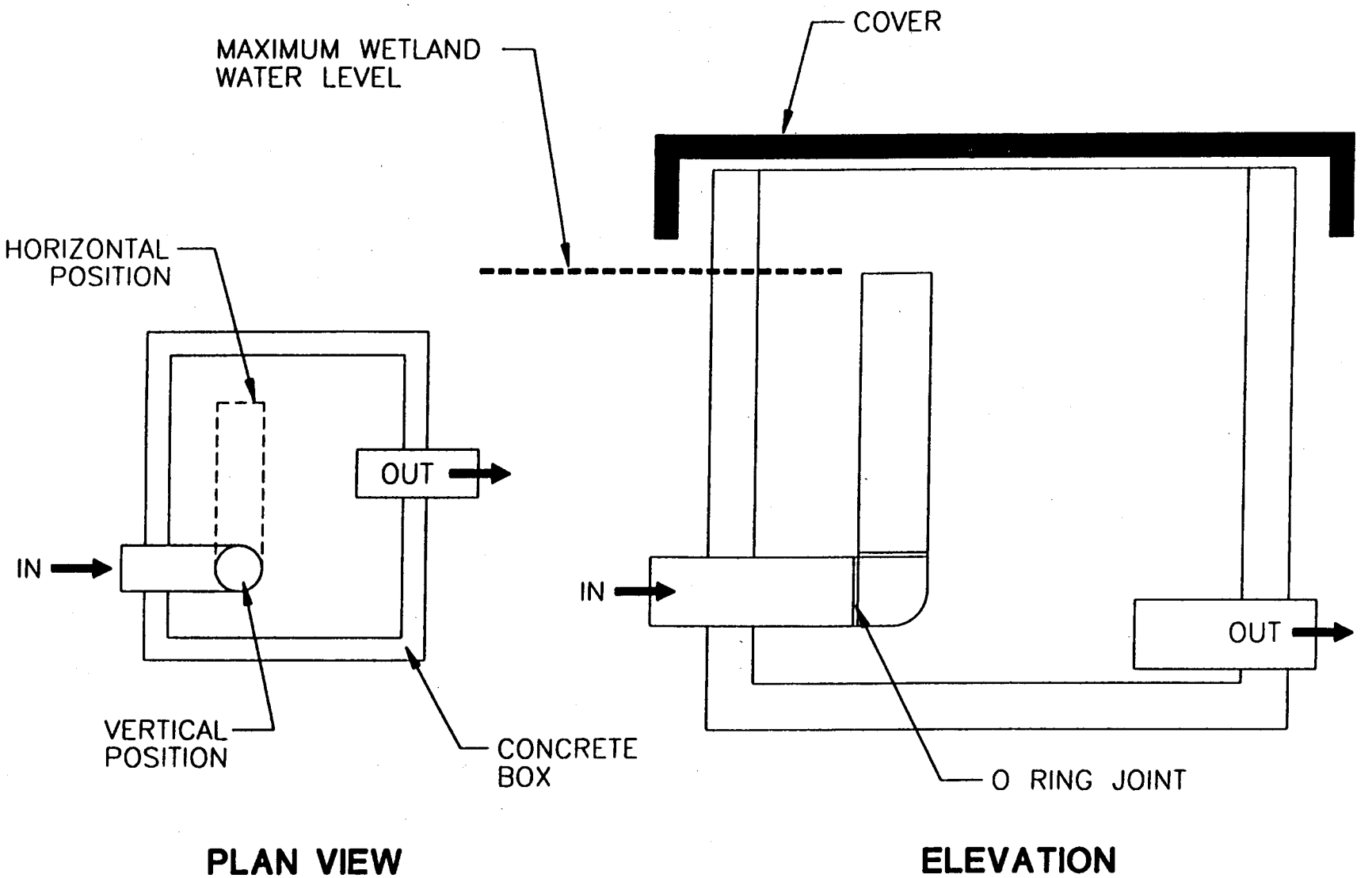
5.7.4 Vector Control

Mosquitoes are common inhabitants of natural wetlands and their presence at approximately the same density in FWS constructed wetlands is to be expected. Mosquitoes should not be a concern for SF wetlands as long as the system is properly operated with the water level maintained below the top of the bed.

Mosquito control is more difficult in polluted waters with a high organic content as might exist near the inlet end of a FWS wetland. Insecticide doses might have to be at least double the normal amount in this portion of the FWS wetland. The same procedures used for general mosquito control are also applicable to FWS wetlands. *Gambusia* fish provide effective control during warm weather conditions. An annual restocking of these fish will be necessary in cold climates with low winter temperatures.

The FWS wetland in Arcata, California, successfully used both *Gambusia* fish and a pupaecide (Altosid) for mosquito control. Bacterial insecticides (*Bacillus thuringiensis israeliensis,* and *Bacillus sphaericus*) have been successfully used on a number of wetland systems. The use of *Bacillus thuringiensis israeliensis* was recommended for use after trials with several insecticides at wetland systems in Kentucky. The side slopes of the containing dikes should be as steep as possible and any vegetation on these surfaces controlled. The presence of duckweed may also contribute to mosquito control by covering the water surface, but this will also interfere with oxygen transfer from the atmosphere.

Figure 5.7.3.1 Adjustable Outlet for Constructed Wetlands



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5.8 OPERATION AND MAINTENANCE

The routine operation and maintenance procedures for these wetlands are similar to those typically required for lagoons. Grass mowing on the dikes and regular inspections for damage from muskrats and other burrowing rodents are the major tasks. Special requirements for the FWS case may include mosquito control. Removal of vegetation residues on a long term basis may also be desirable if interference with the flow in FWS wetlands is observed. An access ramp into each cell should be included in the design and construction of the system.

5.8.1 Root Propagation

Root propagation is especially critical for the SF wetland since effective ammonia removal depends on the oxygen present at the surfaces of the plant roots and rhizomes. It is essential to manage the system to insure development and penetration of the roots through the full depth of the SF bed. This can be accomplished by gradually lowering the water level in the bed in the fall of each year. Another option is to build the system with three parallel cells. Typically all three will be required during colder weather, but during the warm summer months it is likely two would suffice (this can be checked with the ammonia removal design models). In this case two cells are operated in rotation and the third cell is full of effluent but dormant. The nutrients will rapidly be consumed in the dormant cell and this should induce deeper root penetration. Another alternative is to build the SF wetland cell with a depth of only 0.3 m; this should result in full root penetration during the first year. Such a shallow wetland might be at risk of freezing so this concept is limited to seasonal operations.

Special measures for root propagation are not necessary for the FWS wetland type since ammonia removal is not dependent on wastewater contact with the roots. If the vegetation is planted properly and is well established prior to full scale operation there should be no problem with the plant roots in FWS systems. In a few documented cases, large scale spills of sludge at the head end of FWS wetlands overwhelmed the oxygen transfer capability of the plants and there was a die-back. This should not happen when the wetland is used to upgrade lagoons, but if there is a significant die-back anywhere in the wetland revegetation is necessary.

5.8.2 Vegetation Harvesting

The vegetation is a critical component on both SF and FWS wetlands and successful performance depends on a relatively dense stand of healthy plants. Disease and insect infestation can cause problems but this has not been experienced to date at operational constructed wetlands in the U.S.

A major threat to some of the emergent vegetation used in constructed wetlands is damage from muskrat and nutria. These animals use both *Typha* and *Scirpus* as a food source and for nesting material. At one FWS system in western Kentucky an invasion of muskrat during a drought period completely wiped out the vegetation in the wetland cell. This system was revegetated with *Phragmites* which does not serve as a food source for these animals.

Routine harvest of the vegetation is not necessary for either FWS or SF wetland systems. Some of the early SF systems in the southern U.S. used soft tissue flowering plants. These proved highly susceptible to frost, and their rapid decomposition has adverse water quality impacts. In some cases, a fall harvesting program was adopted to control this problem. The problem can be avoided altogether by not using such soft tissue vegetation on constructed wetlands. Removal of accumulated plant detritus may be necessary on an infrequent basis in FWS wetlands to avoid restriction of flow in the system.

5.8.3 Monitoring

Monitoring of the wetland system can be divided into two categories: compliance monitoring to insure the system is meeting discharge standards, and operational monitoring to insure the system is operating in an optimum manner.

Compliance monitoring for wetlands is no different than any other wastewater treatment facility and specific requirements can be found elsewhere. Operational monitoring includes regular inspections by the operator to observe and adjust water levels in the system, to insure maintenance of a dense healthy stand of vegetation, mowing of grass and other vegetation on the dikes, checking for animal burrows in the dikes, and other activities similar to those performed at lagoon systems.

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6 UPGRADING TECHNOLOGIES TO REDUCE LAGOON EFFLUENT TOXICITY

6.1 STORAGE PONDS

6.1.1 Toxicity Reduction Considerations

Sewage lagoons are very common and popular form of sewage treatment as they are capable of providing a high degree of treatment. However, short detention time lagoons are not capable of reducing ammonia to a level that is non-toxic to aquatic organisms. Lagoon effluent polishing and ammonia reduction to a non-toxic concentrations can be achieved using a long detention cell(s) (storage pond(s)).

The performance of lagoon systems with storage ponds has been studied extensively in Alberta. A comprehensive study of the lagoon systems' performance operating in Alberta was conducted by Beier (1987). The results obtained from lagoons having at least 12 months' retention time showed that lagoon effluents are of excellent quality during fall discharge periods, while spring discharge results are not as good; although effluent levels are still comparable to secondary treatment. Much lower ammonia levels were observed in lagoon effluents for fall discharge (approximately 0.3 to 4 mg/L) compared to spring discharge (approximately 6 to 20 mg/L). Similar results were observed for lagoons discharging twice per year (spring and fall discharge) with fall discharges achieving better effluent quality than spring discharges.

The poorer ammonia removal and overall effluent quality noted during spring discharge can be attributed to several factors including: low temperatures; ice and snow covers; and reduced oxygen levels.

The major removal mechanism of ammonia is believed to be air stripping (Pano and Middlebrooks, 1982; Reed, 1985). Sedimentation of organic nitrogen also occurs. Although in the warm, bright summer months some ammonia assimilation in algal biomass occurs, Pano and Middlebrooks stated that only 4% nitrogen removal was achieved by harvesting floating clagophora fracta. The actual amount of ammonia loss to nitrification and denitrification is thought to be limited, especially during summer algae blooms (E.J.Middlebrooks, pers. comm., 1991). Data collected by Beier (1978 and pers. comm.) show that nitrate and nitrite concentrations in storage pond effluents are low for both spring and fall discharges (one to three orders of magnitude lower than ammonia concentrations).

Nitrification probably plays a major role in ammonia reduction in the early summer prior to algal blooms and also in systems that do not experience algal blooms (D.Smith, pers. comm., 1991). It has been stated that algae and nitrification are mutually exclusive since algae and the nitrifiers compete for the same nutrient sources; inorganic carbon and ammonia (Huey, 1982). Thus the major ammonia removal mechanism over an entire summer storage period is probably gaseous stripping (volatilization).

Ammonia volatilization is influenced by temperature and pH, which control the amount of ammonia in solution. Significant ammonia removal occurs at elevated pH levels, as alkaline pH shifts the equilibrium equation towards gaseous ammonia production. As pH is influenced by algal activity in the pond, little or no ammonia volatilization will occur during cold periods, and no ammonia volatilization will take place during the ice cover period. Additional ammonia can be expected during the winter months due to anaerobic decomposition occurring during periods of ice cover (Sherwood, 1991 pers. comm.). During the summer period algal blooms will elevate pH up to 9 and 10 during the day which increases the rate of ammonia stripping.

Temperature and mixing also affect ammonia removal by affecting the mass transfer coefficient and equilibrium constant (NovaTec, 1993).

A study completed by NovaTec (1991) for the City of Whitehorse indicates that lagoon systems with long term storage ponds produce an effluent that is non-toxic. Twelve lagoon systems from Northern Alberta with one year effluent storage and a fall effluent discharge were selected to study the ability of a lagoon system to produce a non-acutely toxic effluent as measured by the LC_{50} fish bioassay. This study showed that ammonia levels are generally reduced to less than 2 mg/L in the storage pond, and 90 % of the samples achieved an LC_{50} greater than 100 %. It was also concluded that 11 out of the 12 lagoon systems were capable of producing an effluent that meets the 100 % LC_{50} requirement, and that the risk factor associated with constructing a lagoon system with effluent storage and annual discharge in the fall is less than 10 %.

Consequently, a lagoon system with a 10 month storage pond prior to discharge was designed for the City of Whitehorse. It is expected that the system will be constructed during 1994. The designed storage duration will allow effluent generated in late fall, winter, and early spring to undergo full summer storage, enabling most of the ammonia contained in the effluent to be volatilized prior to discharge thus achieving non toxic effluent.

6.1.2 Design Considerations

Careful evaluation of present and future community conditions, determination of the site conditions, and environmental protection requirements are necessary prior to design of lagoon and storage pond system.

Storage volumes should be calculated based on the expected maximum annual wastewater flows. Factors such as precipitation, evaporation, and infiltration should also be considered when determining the storage volume.

Important design components for reducing the ammonia toxicity are detention time and the liquid depth. In their lagoon design guidelines Alberta Environment recommends that storage ponds have a maximum depth of 2.5 m and a minimum storage period of one year.

The time of exposure to sunlight during ice free conditions is dictated by detention time and is very important. Smith (1993, pers. comm.) indicates that, generally, after several weeks to two months of ice free conditions good effluent quality is achieved. Beier (1987) reported that April discharges were higher in all parameters studied than May discharges. The optimum effluent quality will be achieved with a fall discharge after five months of ice free conditions. In southern Canada, less than 6 months detention is practical under some conditions, but the trend is towards one year of detention for many locations in western Canada (Heinke et al., 1990). Less than one year detention may be satisfactory, but this should be determined on a site specific basis.

A storage pond can be used as either a means of storage over the winter months (6 months) for continuously discharging lagoons, or as a means of treatment in itself for wastewaters stored over the summer (6 months). Storage pond waters stored over the winter months require further treatment by an upgrading technology before discharge, (i.e. negligible treatment occurs over the winter months). If a storage pond is to be used as both a winter storage facility, and as a summer upgrading technology, the pond must be designed in size to accommodate both these functions.

The data related to depth are not as definitive (Smith, 1993 pers. comm.). More nitrification will occur as the aerobic zone is driven deeper in the storage pond. Although it cannot be quantified at this time, it is expected that the lower portion of the storage lagoon would provide the best total nitrogen reduction if an anoxic zone existed. This is due to the use of nitrate by the facultative microorganisms as an oxygen source causing denitrification. The lower the BOD_5 of the storage pond influent, the closer to the bottom of the storage cell will the denitrification occur. However,

it is not possible to estimate the degree of ammonia removal with different operating depths at the present time (Smith, 1993 pers. comm.).

The designed depth of the storage pond for the City of Whitehorse is 4 m (NovaTec, 1993). Although no data is available for ammonia losses from ponds with depth in this range (the City of Whitehorse will be built in 1994) the depth was dictated by site specific conditions. Ammonia levels and effluent toxicity will be monitored as the level in the storage pond rises and, if needed, action will be taken to assess potential problems. Some of ammonia reduction can also be attributed to nitrification process and sedimentation of organic nitrogen.

A length to width ratio no greater than 2:1, preferably 3:1 is recommended to minimize the effects of short circuiting.

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6.2 NATURAL WETLANDS

Natural wetland systems used to upgrade lagoon effluents are FWS type wetlands. All the reduction factors and considerations described in Chapter 5 for FWS wetlands will generally apply to natural wetland systems. The major difference is that discharge to a natural FWS wetland deals with an existing complex eco-system versus a constructed one. To determine the potential impact of introducing effluent into a natural wetland, an assessment of the wetland should be completed prior to the addition of effluent, which can affect the hydroperiod, organic and nutrient balance in the wetland and may result in a shift in the percentage of vegetation cover and/or a shift in the vegetation type.

6.2.1 Toxicity Reduction Considerations

As stated previously, the most likely toxic constituent in secondary effluent is ammonia. The presence of sulphides in non-aerated lagoons effluents can also cause toxicity. As in the case for constructed wetland, sulphide toxicity would be eliminated during the hydraulic residence time (HRT) needed for ammonia removal.

To determine the potential ammonia removal, the influent total kjeldahl nitrogen (TKN) concentration and minimum wetland operating temperature is needed. The nitrogen removal mechanisms are described in Section 5.4.3.

The limited ammonia reduction that occurs in natural systems during cold temperatures may necessitate storing the lagoon effluent prior to discharge to the wetland.

6.2.2 Wetland Assessment

As for constructed wetlands, site selection is important when selecting a natural wetland system. However, natural wetland systems preclude the feasibility of moving the wetland system to a location that may be more desirable. The variation in topography and geometry that exists in natural wetlands generally results in only a small percentage of the wetland being available for wastewater treatment. In fact, without extensive header systems to distribute the lagoon effluent, and hydraulic control structures to control channelling, the effective treatment area may be as low as 5 - 10% of the total wetland. Therefore, a very large wetland may be needed to provide the necessary treatment area as determined by the design procedures outlined in Chapter 5. To more accurately determine the effective treatment area, a tracer study could be completed. In most cases a conservative estimate

based on topographic information, the proposal effluent distribution system, and the proposed hydraulic control structures would suffice.

The impact of adding lagoon effluent to a natural wetland should be assessed prior to implementing the discharge. The addition of wastewater will increase the water, organic and nutrient loading to the wetland and may alter the natural balance of these parameters in the wetland. The affect and degree of the change will depend on the quantity and quality of the lagoon effluent and on the size and chemical, physical, and biological make up of the wetland.

Additional hydraulic inputs to a natural wetland may change its hydroperiod and/or water depth. The extra water along with the increase in organics and nutrients may cause a vegetation shift to a less desirable dense monoculture. The vegetation shift may also affect the wildlife utilization of the wetland.

In summary the impact of adding wastewater to a natural wetland would need to address the impact on the wetland's;

- hydroperiod,
- water depth,
- organic loading,
- nutrient loading,
- vegetation,
- wildlife habitation.

6.2.3 Design Considerations

Taking into account the reduction in available treatment area for natural FWS wetlands, the design considerations given in Section 5.5 for constructed FWS wetlands will apply. As for constructed FWS wetlands, the design procedure for ammonia-nitrogen removal is an iterative process. It is first necessary to assume a water temperature to calculate the size of the wetland required to achieve the desired ammonia removal during the period of concern. Having determined a tentative size it is then possible using heat transfer models to determine the theoretical water temperature in the wetland. If this theoretical water temperature and the original assumed water temperature do not agree it is necessary to repeat all of the calculations until the two values converge.

Should the required land area be greater than what exists in the natural wetland, than either modifications to the existing wetland must be made to increase the available treatment area or an additional wetland area used. Alternatively the influent ammonia concentration may be lowered by applying another technology prior to the wetland.

The hydraulics of the FWS type are controlled by Manning's equation. Modification to the natural wetland may be considered to insure an appropriate configuration and maintenance of desired flow conditions in the system. A large safety factor is necessary at this stage of design because of the uncertainty involved with any wetland modifications constructed and unknown future operating conditions for the wetland system.

6.2.3.1 Nitrogen Removal Models

A natural wetland system can be considered to be attached growth biological reactors, and their performance can be estimated with first-order plug-flow kinetics. The nitrogen removal and land area requirements can be determined by using Eq. 5-6 to 5-8, as explained in Section 5.5.3. Since nitrogen removal is very temperature dependent, consideration must be given to determine the minimum operating temperature.

It is necessary to assume a wetland water temperature in order to calculate the wetland surface area required to achieve the desired level of ammonia removal. It is then necessary, using appropriate heat transfer models to calculate the theoretical water temperature in the wetland, based on the previously determined surface area. It may be necessary to repeat several iterations of both these calculations until the assumed and theoretical water temperatures converge on the same value.

A conservative assumption for preliminary calculations is to assume that the water temperature in the natural FWS is equal to the average monthly air temperature at the site during ice free periods, but never less than 3°C. In interior British Columbia it is conservative to assume that a FWS wetland will be ice covered most of the winter and the average water temperature would be between 2°C and 3°C. Use of these assumptions for preliminary designs will eliminate the need for detailed thermal analysis. It will still be necessary to calculate the maximum potential depth of ice formation during the winter months to insure that a wetland is physically capable of passing the design volume of water during the winter months.

A more detailed estimate can be made of the minimum water temperature and depth of ice by following the three part procedure outlined in Section 5.5.4.2. If the thermal models for the FWS

wetland predicts a sustained internal water temperature of less than 1°C the wetland may not be physically capable of winter operations at the site under consideration at the design hydraulic residence time (HRT). Ammonia removal will be negligible at those temperatures. Similarly, if Eq. 5-28 predicts a seasonal ice thickness which is greater than about 75 percent of the design depth of a FWS wetland the use of a wetland during the winter months may be questionable. It may be possible to increase the operating depth in these cases as long as the desired treatment results can still be achieved at <3°C and beneath an ice cover which will further impede oxygen transfer for nitrogen removal.

The development of lower water temperature in the wetland may require storage of wastewater over the winter months. In addition to using lagoons to retain wastewater over the winter months, it may be possible to use the wetland as a storage lagoon. This would be feasible as long as the wastewater can be held until the desired treatment level is reached. The impact of this type operation on the natural wetland must be investigated.

6.2.3.2 Hydraulic Design Procedures

As stated earlier, the natural wetland's topography and geometry will impact on the hydraulic design. The hydraulic design of constructed wetland systems is critical to their successful performance. All of the design models in current use assume uniform flow conditions and unrestricted opportunities for contact between the wastewater constituents and the organisms responsible for treatment. These assumptions and goals can only be realized through careful attention to the hydraulic design and to proper construction methods.

The topography and geometry of the natural wetland will determine how much area is actually available for treatment with and without hydraulic modification. The use of distribution headers and berms may allow more of the natural wetland to be applied to treatment. The hydraulic design formula's outlined for constructed FWS wetlands in Section 5.5.4 can be used to determine the hydraulic gradient and flow velocities through the natural wetland.

6.2.3.3 Water Depth

In natural FWS wetlands, the topography and geometry may not make it possible to maintain a desired water level of between 0.3 and 0.45 m. The presence of slopes in the natural wetland may result in shallower water levels. The existence of depressions or the need for berms may result in

deeper water levels. If possible an adjustable outlet structure should be used so that the system operator has direct control over the water depth in the wetland at all times.

6.2.3.4 Detention Time

Detention time or hydraulic residence time (HRT) is one of the critical variables controlling performance in constructed wetland systems. The HRT in these wetland systems is a function of surface area, water depth and the porosity of the wetland matrix. It is possible to increase the detention time in the system by increasing the depth of water. In FWS systems the water depth can be increased in the fall of the year, prior to the onset of low temperatures, this will increase the HRT and compensate somewhat for the lower reaction rates during winter operation. The practical upper limit for long term water depth in a FWS wetland is about 0.45 m.

Based on the monthly design temperature conditions and the expected flow and ammonia concentrations the wetland is sized to provide the desired treatment during the coldest month of the year. Similar calculations for the remainder of the year will indicate the HRT required to provide acceptable treatment and the water depth can be adjusted accordingly.

The use of Eq. 5-6 in Section 5.5.3 would typically require an HRT of 5 to 6 days to produce an effluent ammonia nitrogen of 10 mg/L-N under summer conditions in a FWS wetland, and theoretically about 30 days during the winter months with an under ice water temperature of about 3°C. A 30 day HRT in a FWS wetland in a very cold climate might put the system at risk of freezing. This suggests that winter storage and warm weather FWS wetland treatment might be the desirable system configuration in interior British Columbia.

6.2.4 Vegetation

The vegetation in natural wetland systems is normally left to develop naturally. If plant density needs to be increased in certain area's, Section 5.6 can be referred to. Common plants used in constructed wetlands and establishment procedures are discussed in Section 5.6.

6.3 RAPID INFILTRATION

In rapid infiltration (RI) systems, wastewater is applied to large earthen basins or channels according to a regular schedule of application, percolation into the soil, and adequate drying of the soil matrix. The systems are characterized by a relatively high rate of wastewater application onto the soil followed by rapid percolation, either vertically and/or laterally, away from the application point. In some cases, the applied wastewater is collected and/or recirculated using recovery wells and underdrains beneath the soil zone of treatment. Only the single pass system (no effluent recovery) will be examined in this document. For information on more complicated applications, see references EPA (1981) and EPA (1984).

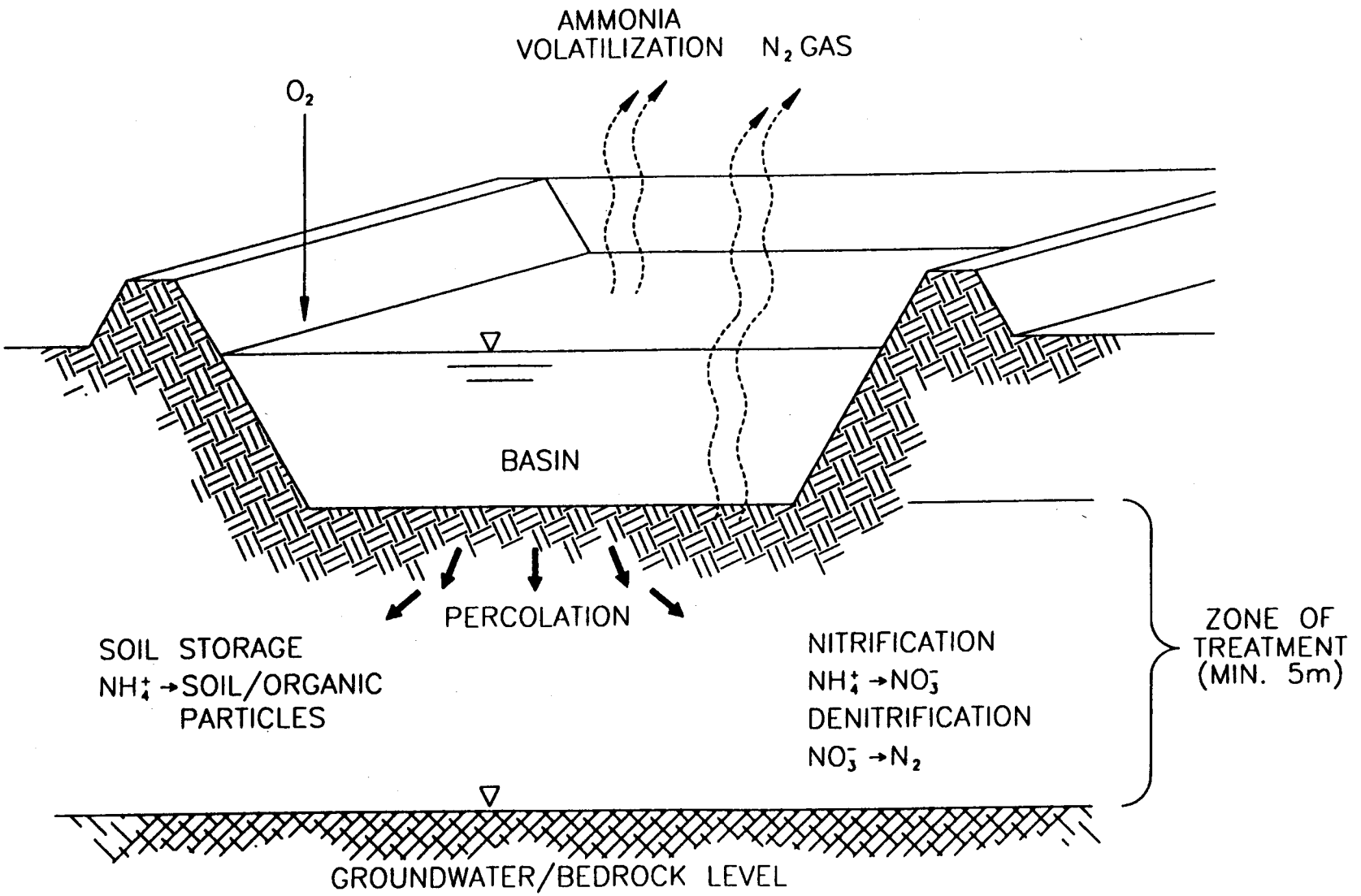
Rapid infiltration systems are used to treat both primary and secondary wastewater, and can effectively reduce effluent toxicity. Gravity percolation of wastewater through a soil matrix provides an environment for the growth of bacteria that biologically detoxify the wastewater. The soil provides a large surface area for bacterial growth: an underground system better insulated against cold ambient air temperatures, and typically large soil depths which provide a large area for treatment. Systems are normally designed to provide acceptable treatment of influent before the percolate reaches groundwater.

Compared to other upgrading technologies, with the possible exception of winter storage, rapid infiltration is one of the techniques most easily adapted to cold weather climates (EPA, 1981). It does not depend on vegetation, is less susceptible to freezing as the majority of the treatment occurs underground, and ice formation on the surface is not a serious hinderance, sometimes even acting as an insulation layer. However, if soil with a low permeability is used in a cold climate, there is a danger of the water freezing in the soil before percolating through. If this is a concern, winter storage should be considered. Cold weather can also adversely affect the biological reactions responsible for toxicity reduction.

6.3.1 Toxicity Reduction Considerations

There are three main mechanisms which potentially play a role in toxicity reduction, specifically related to ammonia removal. These are soil storage, volatilization, and nitrification and/or nitrification/denitrification. Nitrate removal is also achieved as part of the third mechanism. A cross sectional area of an RI system during an application period illustrating the toxicity removal mechanisms associated with ammonia is given in Figure 6.3.1.

Figure 6.3.1 Rapid Infiltration Basin



I:\1231\08\FIG 6.31.DWG

6.3.1.1 Soil Storage

Ammonium ions (NH_4^+) can be removed from wastewater and stored on soil particles through ion exchange. The positively charged ion is bound to the soil particles. Soils with a low cation-exchange capacity (CEC) store little ammonium. For soils in arid climates with less than 2% organic material, soil storage can be a factor for the first three or four years, until an equilibrium is reached in the soil matrix. For soils rich in organics (>5%), nitrogen storage is insignificant (WPCF, 1990).

6.3.1.2 Ammonia Volatilization

Some ammonia can be removed through volatilization. Ideal conditions for this mechanism are soils with low cation-exchange capacity (CEC) that store little ammonium, have a pH greater than 7.8, a spray distribution of wastewater, and ideal climatic conditions (WPCF, 1990), (i.e. dry and windy). It is suggested that this removal mechanism also be considered negligible in gravity feed systems design as it is usually insignificant and so variable with respect to weather conditions and soil types.

6.3.1.3 Nitrification and Nitrification/Denitrification

Nitrification and nitrification/denitrification are the major removal mechanisms for ammonia. Denitrification, although only indirectly related to ammonia removal, is an important part of the nitrogen cycle and so is included here for completeness. The following sections describe these mechanisms in greater detail.

6.3.2 Nitrification

In nitrification, soil storage through ion exchange is an important intermediate step. Applied ammonia (ammonium ion NH_4^+) is first immobilized by ion exchange onto receptor sites on the surface of soil particles. This ammonia is then available to soil nitrifying bacteria for nitrification, provided there is sufficient oxygen in the environment. Upon nitrification, the soil receptor sites are again free to immobilize new ammonia particles. The nitrate resulting from nitrification is either flushed away to the groundwater or denitrified (to be discussed in Section 6.3.3).

Nitrifying bacteria are autotrophs, (i.e. CO_2 is their carbon source), so a low BOD_5 level of incoming lagoon effluent is not a deterrent. Nitrification rates are poor in acidic conditions, with a limiting pH of 4.5. In colder climates, temperature also plays a key role in nitrification rates. Optimum system temperatures are 30-35°C. At 2-5°C nitrification is significantly reduced, effectively ceasing at 0°C

(EPA, 1981). Recent experience has demonstrated that an RI system can operate successfully at 4°C (Reed et al., 1988). However, for some colder climates, winter storage may be necessary for adequate treatment.

Table 6.3.2 presents nitrification efficiencies of several operating RI systems. Application schedules are expressed by a Wet:Dry ratio, i.e. the duration of wastewater application : time between applications.

As is evident from Table 6.3.2.1, there is no one consistent set of design criteria which applies to all locations. System performance is very site specific and design variables must be adjusted to suit the site in question. A number of site specific factors affect nitrification including: influent characteristics; soil type; soil depth; application rate; and climate. In designing a new system, it is advisable to first search for information on the performance of existing similar systems and, taking into consideration the guidelines given here, and design accordingly.

6.3.2.1 Application Schedule

The application schedules presented in Table 6.3.2.1 vary considerably. In Boulder, Colorado, an 88% nitrification rate was attained with a 6 week wet to 1-4 week dry schedule. A similar nitrification rate (86%) was attained with a 9 days wet to 12 days dry schedule in Pheonix, Arizona. Application schedules must be custom designed for each system to optimize nitrification.

Most importantly, RI systems implemented to reduce ammonia must be designed to ensure aerobic conditions throughout the soil matrix. This is primarily realized through careful scheduling of the wetting and drying periods of the soil basin to suit local soil and climate conditions. Ammonia is readily converted to nitrate in the aerobic environment during initial application of wastewater to the soil basins. However, the constant application of wastewater prevents reaeration of the soil. Oxygen then becomes the limiting factor preventing further nitrification. The wastewater application period should be short enough such that the upper soil layers remain predominantly aerobic, and the drying time is long enough to reaerate the lower soil matrix allowing nitrification of immobilized ammonia. The US EPA (1981) recommended loading cycles for nitrification of secondary wastewater are given in Table 6.3.2. In contrast, a second reference (WPCF, 1990) recommends that if the influent of the RI basin has a low SS content and therefore lower chances of clogging the basin, hydraulic loading rates could be maximized using flooding periods of 2 to 3 weeks followed by 10 to 20 days drying to achieve optimum removal. Generally speaking, the application schedule for maximum nitrification is the same as that for maximum hydraulic loading (EPA, 1981). In many cases, drying

Table 6.3.2 Nitrification in RI systems

Location	Influent (mg/L)			Effluent (mg/L)			Percentage of ammonia Removed	Application Schedule Wet:Dry	Hydraulic Loading (m/yr)	Ref.	Ambient Air Temperature
	TN	Total Ammonia-N	NO3-N	TN	Total Ammonia-N	NO3-N					
Boulder Co.		9.2 6.6 6.7			<1.5 <1.5 <2.5	5-25 5-15 1-12	>84 >77 >63	6 wk: 1-4 wk (1 dry, summer) (4 dry, winter)	30.5 12.8 48.8	Carlson et al. 1982	Year round 23.3-0.6 C Mean Annual 11.2 C
Pheonix, Arizona	18	16	trace	5.56	0.09	5.3	99	2 wk:2 wk	90	Bouwer and Rice, 1984	--
Pheonix, Arizona	27.4	20.7	3	9.6	2.8	6.2	86	9:12 d	61	Bouwer et al., 1980	--
Israel	13	5	0.1	7.2	<0.02	6.5	99	1:2 or 3 d	100-150	Idelovitch and Michail, 1984	Summer 20 - 30 C Winter 10 - 20 C

Table 6.3.2.1 Application Scheduling for Nitrification of Secondary Wastewater

	Days of Application	Days of Drying
Summer	1 - 3	4 - 5
Winter	1 - 3	5 - 10

(Reference EPA, 1981)

times of only four to five days have proven to be adequate to allow for re-aeration (EPA, 1984). In colder climates, proportionately longer drying times are recommended to allow for the slower conversion rate. The use of vegetated bottoms or gravelled surfaces may prevent drying of the soil matrix; this should also be taken into consideration when determining drying times.

6.3.2.2 Nitrogen Loading

The hydraulic loading of a system should be adjusted to accommodate the allowable nitrogen loading and the permeability of the soil. The correlation between loading and application rate can be seen in the last three cases of Table 6.3.2; a decrease in hydraulic loading rates (100-150, 90, 61 m/yr) is accompanied by a corresponding increase in influent ammonia concentration (5, 16, 20.7 mg/L-N).

With respect to nitrification, the estimated mass nitrogen loading rates can be determined by (EPA, 1981):

$$L_n = \frac{10 \cdot C_n \cdot L_w}{365} \qquad (6\text{-}1)$$

Where:

L_n = nitrogen loading rate, kg/ha·day

C_n = applied total nitrogen concentration, mg/L

L_w = annual loading rate, m/yr

Under favourable temperature and moisture conditions, assuming nitrification takes place in the top 10 cm of the soil, up to 67 kg/(ha·day) of ammonia nitrogen can be nitrified (EPA, 1981). However, loadings within the range of 3.4 - 41.4 kg/(ha·day) have been consistently successful (WPCF, 1990). For colder climates, loading rates in the lower half of the range (< 20 kg/(ha·day)) are more appropriate (Leach et al, 1990).

After a drying period, the effluent is characterized by an initial flush of nitrates; up to an order of magnitude greater than the equilibrium value. This is the result of nitrification during the drying period. Following an initial flush, equilibrium conditions determine nitrate concentrations (Leach, 1983).

6.3.3 Nitrification/Denitrification

RI systems implemented to reduce ammonia by nitrification/denitrification are designed to accommodate the aerobic environment required for nitrification and the anaerobic environment required for denitrification. Denitrification rates depend on the available carbon to nitrogen ratio, soil temperature, pH and moisture. Unlike nitrifying bacteria, denitrifying bacteria require an organic carbon source. Total Organic Carbon (TOC) to Total Nitrogen (TN) ratios of at least 2 are required (EPA, 1981), or Biochemical Oxygen Demand (BOD_5) to nitrogen ratios of 3 (Reed et al, 1988). If denitrification is a priority and organic carbon loading rates are low, an external source of carbon should be considered. Similar to nitrification, denitrification is also effected by temperature and pH. The optimal temperature range is 30-35°C, with significantly reduced rates at 2-5°C, and effectively ceasing at 0°C. For colder climates, winter storage of wastewater may be necessary to achieve desired effluent characteristics. Denitrification has a limiting pH of 5.5 (EPA, 1981).

Examples of some RI systems achieving nitrogen removal are presented in Table 6.3.2, 6.3.3.

6.3.3.1 <u>BOD_5 Loading</u>

As previously stated, influent carbon to nitrogen ratios should be at least 3 to 1 for successful denitrification (EPA, 1981). It is also recommended that application rates be maintained between 20-125 kg BOD_5/(ha.d) to control infiltration surface clogging (EPA, 1992). More than half of the locations shown in Table 6.3.2, 6.3.3 have an influent BOD_5:N ratio of less than 3:1. These installations generally achieve nitrogen removal rates lower than 50%. For BOD_5:N ratios of 3.6:1 and 5.5:1, the removal efficiencies are 71% and 93% respectively.

Table 6.3.3 Nitrogen Removal in RI Systems

Location	Influent TN (mg/L)	Loading Rate (m/yr)	Loading Rate (cm/d)	BOD5:N Ratio	Application Schedule Wet:Dry	Effluent TN (mg/L)	Effluent NO3- (mg/L-N)	Removal % of total N
Boulder, Colorado	16.5	48.8		2.3:1	1:3	9-16	6-16	10-20
Brookings, South Dakota	10.9	12.2		2:1	1:2	6.2	5.3	43
Calumet, Michigan	24.4	17.1		3.6:1	1:2	7.1	3.4	71
Fort Devens, Massachusetts	50	30.5		2.4:1	2:12	19.6	13.6	61
Hollister, California	40.2	15.2		5.5:1	1:14	2.8	0.9	93
Lake George, New York	11.5 12	58 58		2:1 2:1	1:4 1:4	7.7 7.5	- -	33 38
Phoenix, Arizona	27.4	61		1:1	9:12	9.6	6.2	65
Pontotoc County, Oklahoma (soil a)	17.2 17.4 17		47.5 43 25.7		3:4 d 1:1 d .01:.05 d	11 12.1 14.7	9.9 10.1 13.6	36 30 14
(soil b)	17.2 17.4 17		33.2 15.2 24.2		3:4 d 1:1 d .01:.05 d	7.1 3.6 14.2	6 3.1 13.1	59 79 16

Taken from EPA, 1981; Leach & Enfield, 1983.

6.3.3.2 Application Schedules

The application schedules shown in Table 6.3.2.1, 6.3.3.2 vary significantly. Because of the low BOD_5 loadings of most of the locations discussed, it is not appropriate to compare performances with application schedules.

Denitrifying systems typically have application/drying cycles with longer application periods than systems designed only for nitrification, in order to allow an anaerobic environment to develop; near-saturated conditions in the soil media are desired. This long application period is also important for maximum ammonium adsorption. Drying periods must be long enough to allow almost complete soil re-aeration. Recommended cycles for maximum nitrogen removal in secondary wastewater are presented in Table 6.3.2.1. In colder climates, longer drying times may be required.

Table 6.3.3.2 Application Schedule for Nitrification/Denitrification of Secondary Wastewater

	Days of Application	Days of Drying
summer	7 - 9	10 - 15
winter	9 - 12	12 - 16

(Reference EPA, 1981)

6.3.3.3 Nitrogen Loading

An outline of how to calculate nitrogen loading is presented in Table 6.3.2.3. In soil column studies, a maximum infiltration rate of 15 cm/d for treating secondary wastewater is recommended for 80% nitrogen removal (EPA, 1981). Faster infiltration rates only flush the nitrates through the soil matrix before they can be denitrified.

Table 6.3.3.3 Calculating Nitrogen Loading
(Taken from EPA, 1984 and Reed et al., 1988)

1. Determine the mass of ammonium that can be stored in the soil profile per unit area for the unsaturated depth based on the cation-exchange capacity of the soil.

2. Calculate the length of the loading period required for maximum ammonium adsorption (as determined in step 1), using the daily application rate and the known ammonium concentration of the applied effluent.

3. Check that the ammonia and organic nitrogen combined loading are less than 67 kg/(ha.d) (the maximum value under ideal conditions).

4. Determine the length of the design loading period based on estimated ammonia adsorption and application schedules as shown in Table 6.3.2.1.

5. Compare the mass of nitrate nitrogen produced against influent BOD_5 to ensure an adequate ratio (BOD_5:N >3) for denitrification.

The purpose of discussing denitrification is in recognition that nitrate loading to groundwater is a significant concern in many locations, and should be considered in any RI application.

Denitrification can be optimized by reducing the infiltration rate in the flooded basin. This is done by lowering the depth of flooding, incorporating finer-textured soils into the infiltration surface, or compacting the soil.

Optimal nitrogen removal then dèpends on:

- loading periods for complete nitrification of influent ammonia, and
- infiltration rates to provide desired level of denitrification.

6.3.4 Hydrogeological Considerations and Assessment

6.3.4.1 Soil Types

The best soil types for rapid infiltration are relatively coarse textured soils (sand, sandy loam, loamy sand and gravels). Soils used for infiltration should not have significant amounts of silts, fines, or clay. These particles will rise to the surface of the basins during flooding and cause clogging after drying. This can effect the reaeration ability of the basins. EPA (1981) recommends that less than 10% of the soil being considered for rapid infiltration purposes consist of fine textured soils (silt, clay).

6.3.4.2 Soil Depth

For effective effluent treatment, a minimum soil depth to the bedrock or groundwater is required. The soil depth to the bedrock should be at least 5 metres (Reed et al., 1988). Depths of 5 meters or more are preferred from the base of the infiltration basin to the groundwater (Reed et al., 1988), although depths as shallow as 1.2 metres have been used successfully (EPA, 1984). The distance between the capillary fringe above the groundwater and the bottom of the infiltration basin should always exceed 0.6 m (EPA, 1981).

6.3.4.3 Mounding

Because of the volume of water being discharged, groundwater mounding may occur. This is not a concern if it does not effect the infiltration rates of effluent at the surface or interfere in aerobic restoration during the drying periods. Mounding can be minimized by the use of long thin channels in place of larger basins. Calculations of the expected level of mounding and its possible effects should be part of the design process. Refer to EPA, (1981) for details.

6.3.4.4 Infiltration and Permeability Testing

In RI systems, infiltration rates determine loading rates which, in turn, determine application rates. Soil permeability should preferably be greater than 50 mm/hr (EPA, 1981).

Soil permeability and condition must be assessed at the site being considered for rapid infiltration use, and at the intended design depth. Tests should be conducted during the coolest and calmest (ie. low winds) time of the year. If the soil profile is varied, the permeability of the most restrictive layer

should be used for design. Systems have failed because this layer was considered too thin to be of any relevance (Reed, 1985).

The preferred method of soil permeability measurement is the basin infiltration test using clean water. Test basins with 3 m diameter or greater should be used. Other methods have shown to be extremely inconsistent and inaccurate (EPA, 1981).

6.3.5 Groundwater

Knowledge of the seasonal rise and fall of the groundwater is essential. Systems have failed because spring water table levels are too high to allow percolation at design rates (Reed, 1985). This can significantly effect nitrification/denitrification rates.

In some cases, the percolate from the RI basins will mix with the groundwater shortly after infiltration. Concerns may then arise as to the contamination of the groundwater by nitrate if intended for potable use. Before design, groundwater resource surveys, level measurement and chemical analysis should be done for all RI basins. If nitrate contamination proves to be a concern, calculations should be done to determine the level of nitrate treatment required by the RI system to allow uncontrolled mixing with the groundwater. To estimate the level of mixing, refer to EPA, 1981.

Wells used for groundwater characterization in testing and design should be maintained to record system performance.

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6.4 OVERLAND FLOW

In overland flow (OF), treated effluent is distributed across the upper portions of carefully graded, vegetated slopes, and allowed to flow over the slope surfaces to run-off collection ditches at the bottom of the slopes. Application cycles alternate between wastewater application and slope drying. The systems are characterized by relatively impermeable soils (<5.0 cm/hr), shallow grades (0-12%), and grassy slopes. Crop uptake is a fairly significant pathway for nitrogen removal. In a cold climate, overland flow system operation should only be seasonal. Storage of lagoon effluent over the winter months would be required.

6.4.1 Toxicity Reduction Considerations

Toxicity reduction here specifically refers to ammonia reduction. In OF systems, there are three significant ammonia/nitrogen removal mechanisms:

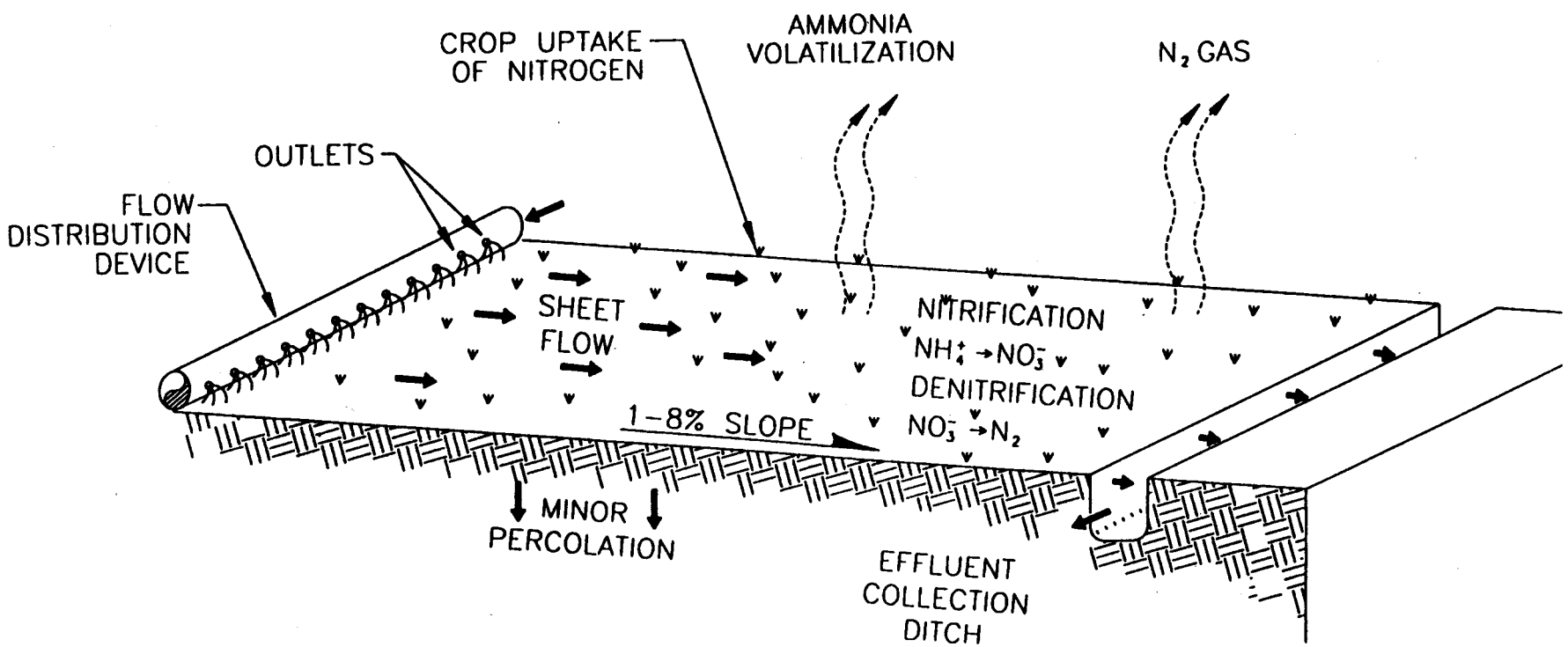
1. crop uptake,
2. ammonia volatilization and
3. nitrification/denitrification.

These three ammonia/nitrogen removal mechanisms are illustrated in Figure 6.4.1, and discussed in greater detail in this section.

6.4.1.1 Crop Uptake

Grasses on the slopes absorb ammonium and nitrate through roots and incorporate the nitrogen into their plant mass. Depending on the type of vegetation and climate, crop uptake can account for up to 20 - 30% of nitrogen removal (WPCF, 1990). With average nitrogen loadings (TKN = 35 mg/L-N) and a full scale overland flow slope (50m), 6 - 25% of the applied nitrogen can be expected to be taken up by the crop (Kruzic, 1990). Mass estimates of crop cover removal are 200-400 kg N/ha, depending on the type of grass used and harvesting practices (Kruzic, 1990). Climatic conditions determining growing seasons and rates are obviously also an important factor in the crop uptake mechanism.

Figure 6.4.1 Overland Flow



6.4.1.2 Ammonia Volatilization

With a gravity flow pipe distribution system, volatilization losses can be expected to be less than 5%. Higher losses (up to 40%) can be expected using spray irrigation (WPCF, 1990).

6.4.1.3 Nitrification and Denitrification

Nitrification and denitrification is the most important mechanism for ammonia reduction and overall nitrogen removal. Denitrification, although not directly related to ammonia removal, is an important part of the nitrogen cycle and so is included here for completeness. The majority of the nitrification/denitrification reactions occur on the soil surface or in the accumulated mulch, not in the water flowing over the slope (Kruzic, 1990). During application, influent ammonium (NH_4^+) is immobilized onto soil and organic particles. Some of this immobilized ammonium is nitrified immediately in an aerobic environment, and possibly denitrified in anaerobic zones. A longer application period produces a more anaerobic environment, which in turn encourages denitrification. While drying, the soil is reaerated and nitrification of the remaining immobilized ammonium occurs, freeing the sites on the soil particle surfaces for new ammonium ions. In the following application period, the nitrate produced during the drying period is flushed out, and new ammonium ions are immobilized, and anaerobic zones are again gradually created as available oxygen is used up. Design guidelines to optimize nitrification and denitrification are considered in this section.

Depending on the soil type, some wastewater percolation through the soil may occur. This should only account for a very small percentage of the applied wastewater if appropriate impermeable soils are used.

6.4.2 Nitrification

Examples of some OF systems' performances with respect to nitrification are presented in Table 6.4.2.

As is evident in Table 6.4.2, there are several variables which effect nitrification rates. These include: the application schedule; hydraulic loading and application rates; temperature and pH; and slope length. Each is discussed in more detail in this section.

Table 6.4.2 Overland Flow Nitrification/Nitrogen Removal

Type of Wastewater	Application Rate (m3/hr.m²)	Hydraulic Loading (cm/d)	Applic. Schedule	Slope Length (m)	TKN (mg/L)		Percentage removed	Total Ammonia-N (mg/L)		Percentage removed	Ref.	Ambient Air Temp.
					Infl.	Effl.		Infl.	Effl.			
Lagoon effluent	0.21	2.4	5-8 hr/d 5 d/wk	45.7	4.88	2.29	53	1.91	0.14	92	Abernathy, 1985	–
	0.29-0.39	5.4	5-8 hr/d 5 d/wk	45.7	13.2	6.68	49	<0.4	<0.16	>60		
Primary Effluent	0.33	12.3	12 hr/d 6 d/wk	32	-	-	-	14.9	6.3	58	Witherow, 1986	35 - 9 C
	0.11	4	12 hr/d 6 d/wk	32	-	-	-	14.9	1.7	89		
Primary Effluent	0.1	1.9	8 hr/d 5 d/wk	-	35	10	71	27	5	81	Smith, 1985	11 - 26 C
	0.37	7.1	8 hr/d 5 d/wk	-	35	20	43	27	17	37		
Secondary Effluent	0.1	2.1	2 wks wet, 3 wks dry 24hr/day	46	-	-	-	-	0.3-3.3	81	Overman, 1986	–

6.4.2.1 Application Schedule

Application schedules determine the duration of wastewater application to OF slopes and the length of drying periods between those applications. The ratio between the wetting and drying periods effects ammonia removal by controlling the oxygen availability (aerobic/anaerobic zones) and the time available for nitrification. From the examples presented in Table 6.4.2, hydraulic loading rates ranging from 5-8 hours per day for 5-6 days a week to 24 hours a day, for two weeks application followed by 3 weeks of drying are used with varying success. The inconsistency of removal efficiencies suggests that other, more dominant factors, control ammonia removals in the cases considered. Generally, the standard application schedule for treating stabilization pond effluents is 8-12 hours per day, operating 5-7 days per week (EPA, 1984).

The ratio between application and drying time on a given day can be expressed as the wet:dry (W:D) ratio. Average ammonia removals at varying W:D ratios is given in Table 6.4.2.1.

Table 6.4.2.1 W:D Ratios and Ammonia Removal for OF

W:D Ratio	Percentage ammonia removal
0.25	85
0.5	70
1.0	50

(Adapted from WPCF, 1990.)

As the W:D ratio increases, the percentage of ammonia removed decreases. Therefore, for optimal nitrification, shorter application periods followed by longer drying times should be used.

6.4.2.2 Hydraulic Loading Rate and Application Rate

In each study presented in Table 6.4.2, an increase in hydraulic loading rate results in a decrease in ammonia removals. Higher loading means shorter hydraulic retention times and a greater volume of wastewater for treatment per square meter of slope.

Suggested hydraulic loading rates for OF systems are given in Table 6.4.2.2.

Table 6.4.2.2 Hydraulic Loading for OF Systems

Preapplication Treatment	Application Rate ($m^3/h.m^*$)	Hydraulic Loading Rate (cm/d)	Reference
Primary aerated pond (1-2 days detention time)	0.2-0.4		WPCF, 1990
Wastewater treatment pond (not including algae removal)	0.09-0.15	2.5-9.0	EPA, 1984
Secondary	0.11-0.17	3.0-10.0	EPA, 1984
Secondary	0.25-0.6		WPCF, 1990

* per meter width of slope.

The values recommended by the EPA are more conservative than those by the WPCF. The lower end of the range of rates suggested by the WPCF should be used in colder climates. In designing an OF system, it is recommended that the application and loading rates of similar systems that operate successfully be used.

6.4.2.3 Temperature and pH

Ammonia removal in OF systems is possible down to 5°C (WPCF, 1990). In colder climates, adequate removal of ammonia and/or nitrates may not be possible. In these cases, winter storage should be considered (EPA, 1981). Winter loadings (at temperatures less than 10-15°C) of successful systems are usually 30-50% lower than summer values (EPA, 1981). At soil temperatures near 0°C, physical operation of systems is possible and BOD_5 removal will continue, but nitrogen removal will be nil. Daytime application at temperatures above zero, and drying when night temperatures fall below zero, is a feasible option (Reed, 1988). pH only becomes a serious concern below 5; this is hardly ever a problem with secondary effluent (EPA, 1981).

6.4.2.4 Slope Length

Longer slope lengths increase the surface area for nitrification, resulting in improved ammonia removal (Abernathy, 1985). Slope lengths of 45-60 m should be used for maximum removal. A minimum of 30 m is recommended (Reed, 1988).

6.4.3 Nitrification/Denitrification

Nitrogen removal above 80% is nearly impossible to attain in OF systems (WPCF, 1990). Because of the short detention time as wastewater flows down the slope, not all of the nitrate will pass through the mulch to the anaerobic zones. Denitrification rates are dependent on the influent organic carbon levels, the application rate, and scheduling. Temperature, pH and slope length requirements are the same as for nitrification.

6.4.3.1 Influent Organic Carbon

Denitrification requires a carbon (as TOC) to nitrogen ratio of at least 3 to 1 (EPA, 1981). In some secondary effluents, the carbon content may be a limiting factor. Exogenous carbon can be added to increase this ratio (WPCF, 1990)

6.4.3.2 Application Rates and Scheduling

Compared to nitrification only systems, nitrogen removal systems have longer application periods at lower rates to promote the development of anaerobic zones and nitrate removal. In several systems using stabilization pond effluent during the warm season, hydraulic loading varied from 1.27 cm/d to 5.08 cm/d, resulting in nitrogen removals of 79% and 51% respectively (EPA, 1981). Again, when designing an OF system, it is recommended that the application rates, loading rates and scheduling of similar systems that operate successfully be used.

6.4.4 Vegetation

The type of vegetation used is important because it effects the surface area available for the biological slime to grow on, the amount of nitrogen taken up, and the amount of maintenance required. A close growing grass crop is essential to success. Perennial grasses are better as they have a higher moisture tolerance. Grasses with longer growing seasons are preferred as they have a higher nutrient uptake. Grass should be suited to climate and soil conditions. Single stalk species are not suitable as they

provide minimum surface area for bacteria and do not slow the flow of the wastewater sufficiently for proper treatment. A mixture of species is best to provide resilience and year-round treatment.

For cool seasons, reed canary grass, fescue and rye grass are recommended. For warm seasons, common and coastal Bermuda, Dallis and Bahia grasses are recommended (EPA, 1992). A local agricultural expert should be consulted before choosing the grass(es) to be used.

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6.5 FIXED FILM SYSTEMS

Fixed film systems, also known as attached growth systems, are primarily composed of a dense growth of microorganisms on an inert medium with possibly aerobic and/or anaerobic zones. Fixed film systems biologically convert organic matter and other constituents to gases, water, other organic and inorganic by-products and cell tissue. With respect to toxicity reduction, specifically ammonia reduction, ammonia volatilization and nitrification/denitrification are the two key mechanisms. The two fixed film systems described here are trickling filters and rotating biological contactors.

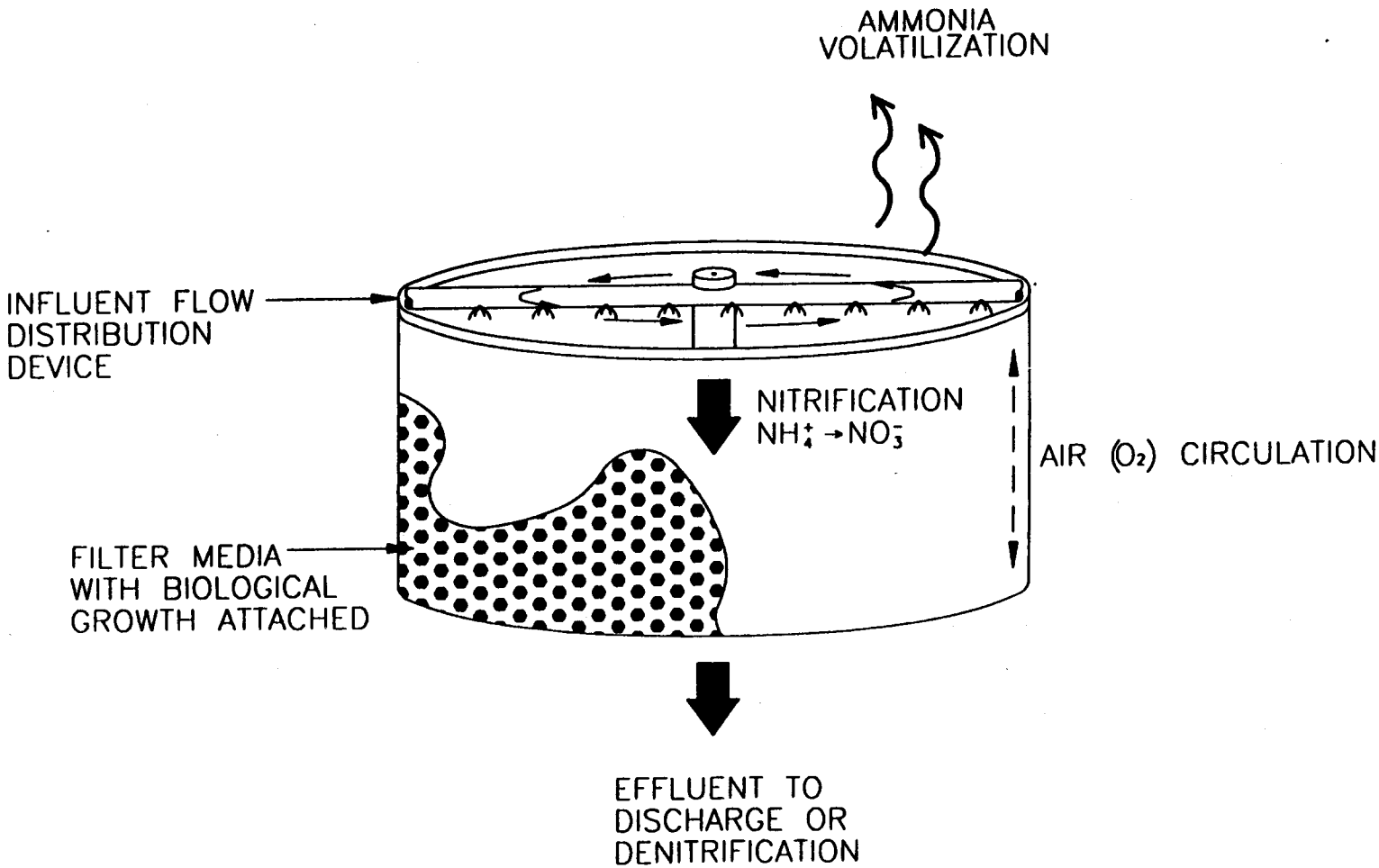
All types of fixed film systems will be severely limited by winter conditions in cold climates. The exposure of a fixed film to the cold ambient air and wastewater effectively halts the growth and treatment capability of the microorganisms. With the exception of coastal temperate climates (i.e. the Lower Mainland), fixed film systems should be designed in conjunction with winter storage of lagoon effluent (6 months) and be used for wastewater treatment only during the summer (6 months).

6.5.1 Trickling Filters

Trickling filters (TF) have traditionally been composed of circular or rectangular beds of rock media stacked at least 2 m deep. Settled wastewater, applied at the top of the filter, flows vertically by gravity feed, and is collected at the bottom for discharge. Modern improvements include the use of plastic media instead of rock, resulting in a higher surface area to volume ratio and an easier environment for bacterial colonization. The application of wastewater has been improved by flow distributors that evenly divide the influent over the top of the tower. In more complex systems, effluent recycling and fan blowers have been used to improve biological treatment and volatilization respectively.

Trickling filters are only examined here in their capacity to remove ammonia from wastewater in order to reduce ammonia toxicity. The two major removal mechanisms operative in TF systems are ammonia volatilization and biological nitrification. For significant denitrification, a separate unit and external carbon source are required. A diagram of a typical trickling filter illustrating ammonia removal mechanisms is shown in Figure 6.5.1.

Figure 6.5.1 Trickling Filter



I:\1231\08\FIG 651.DWG

6.5.1.1 Ammonia Volatilization

The rate of ammonia volatilization increases with higher pH, temperature, and air flow in the TF. Volatilization can be encouraged by mechanically blowing air through the filter, stripping the ammonia. This mechanism, however, is usually minor compared to nitrification.

6.5.1.2 Nitrification

Nitrification is the major ammonia removal mechanism in TF systems. The rate of nitrification primarily depends on the filter depth, the influent characteristics, temperature, BOD_5 loadings, hydraulic loading, and the type of media used. Table 6.5.1.2 presents some TF design value recommendations (Tchobanoglous and Burton, 1991) and examples of several TF systems.

Because of the number of variables, it is not possible to provide any specific standard for a universally successful nitrifying trickling filter. It is possible, however, to offer guidelines for better nitrification.

Hydraulic Loading Rate

As the hydraulic loading rate increases, the nitrification rate decreases. This effect is evident when examining the results reported by Duddles, 1977 in Table 6.5.1.2. With an increase in hydraulic loading from 29.3 to 117.2 $m^3/(m^2.d)$, the ammonia removal efficiencies drop from 80-90 % to 60-70%. The results reported by the EPA also show a drop of 20% in efficiency when hydraulic loading rates are increased from 9.35 to 28.05 $m^3/(m^2.d)$. Preferably, hydraulic loading for a given systems should be adopted from an already existing system with similar temperature and influent characteristics. Expected hydraulic loading rates are within the range given in Table 6.5.1.2.

BOD_5 Loading

For successful nitrification, it is necessary to have BOD_5 loadings low enough to prevent a large population of heterotrophs (organic carbon users) outcompeting the autotrophic nitrifiers for the available media surface area. From Table 6.5.1.2 it is evident that BOD_5 loadings of 80-128 $g/(m^3.d)$ have enabled the nitrifying bacteria to successfully populate the filter media, although higher loadings are recommended by Tchobanoglous and Burton (1991).

Table 6.5.1.2 Trickling Filters Nitrification

Influent (mg/L)		Effluent (mg/L)		NH3 removed %	Temp (C)	Hydraulic Loading (m3/m2.d)	BOD5 Loading (g/m3.d)	Height of filter (m)	Media	Ref.
Total Ammonia -N	NO3-N	Total Ammonia -N	NO3-N							
				75-85 85-95 75-85 85-95			160-96 96-48 288-192 192-96		rock rock plastic plastic	Tchobanoglous and Burton, 1991
				72 52		9.35 28.05		2 2		EPA, 1977
10-20 10-20		1-2 1-2		>90 >90 >90	12-18 18 6	0.3	93		105 m2/m3	Barnes and Bilss, 1983
13.1 17.6 12 15.5 13.3 8.1 7.5 11.8 13.4 10	1.1 0.4 0.8 0.3 1.2 0.5 1.1 0.3 0.4 0.6	4.9 5.9 1.7 6.1 2.5 1.1 1.2 1.5 2 1.9	8.9 9.7 10.3 8.8 10.7 8.5 7.5 8.7 9.5 8.5	63 66 86 61 81 87 84 87 85 81	15.5 14.4 12.7 7.8 18.3 10 7.2 14.4 15.5 10	0 58.6 58.6 58.6 58.6 41.6 41.6 41.6 41.6 29.3	80-128 80-128 80-128 80-128 80-128 80-128 80-128 80-128 80-128 80-128	7.5 7.5 7.5 7.5 7.5 7.5 7.5 7.5 7.5 7.5	Plastic 82 m2/m3 82 m2/m3 82 m2/m3 82 m2/m3 82 m2/m3 82 m2/m3 82 m2/m3 82 m2/m3 82 m2/m3 82 m2/m3	Duddles et al., 1973

H:\1231\08\CHP6\TF WB1

Depth

For a given flow rate, specific surface area, and temperature, nitrification rates will improve as filter medium depth increases (EPA, 1977). As the medium depth increases, a larger surface area for nitrification results, naturally leading to better nitrification of influent. In the study by Duddles et al., 1973, nitrification was shown to occur throughout the entire 7.5 m trickling filter depth. The influent BOD_5 concentration range of 15-30 mg/L made the entire use of the trickling filter for nitrification possible.

Temperature

As temperature decreases, TF nitrification rates decrease. This is evident in the results reported by Duddles et al., 1973, in Table 6.5.1.2. At a hydraulic loading of 58.6 $m^3/(m^2.d)$, a temperature drop from 12.7 °C to 7.8 °C corresponded with a decrease in nitrification from 86% to 61%. At lower loading rates, the difference in nitrification rates between higher and lower temperatures are not as marked. Trickling filters can therefore operate successfully during winter months (down to 2.8°C in this case) if the hydraulic loading rate is adjusted to accommodate the slower removal rates (Duddles, 1973). For temperatures below 15°C, pilot studies should be done to evaluate nitrification rates under specific environmental conditions (EPA, 1993).

Recycle

Some sources report that recycling of the effluent does not improve nitrification. It only stabilizes performance by providing adequate flow for wetting (EPA, 1977; Duddles, 1973). Other sources, however, report that nitrification rates are improved by recycling (Barnes, 1983). Recycling effluent can improve nitrification if the influent BOD_5 concentration is too high to promote autotroph growth. A diluted BOD_5 controls the growth of competing heterotrophs, allowing a large nitrifying population to cover the filter media. If recycling is to be considered, the increased costs for pumping must be weighed against any potential improvement in nitrification.

Solids

Because nitrifying TF systems are subjected to very low BOD_5 loads, solids production is minimal and clarifiers are usually not required for TF effluent. Only if the hydraulic loading rates vary widely in regular operation should a clarifier be necessary (Duddles, 1977).

Media

Plastic media is preferred to rocks or other materials as it has a higher surface area to volume ratio and allows better ventilation. Because the solids production in filters designed for nitrification is so low, media densities of 300 $m^2/m^3.d$ can be used without clogging. In contrast, filters treating effluent with a high BOD_5 use media densities of 100 $m^2/m^3.d$ (Barnes, 1983). Differences in loading rates and nitrification efficiencies for plastic and rock media are given in Table 6.5.1.2 (Tchobanoglous and Burton, 1991). Plastic media with cross flow characteristics are preferred to vertical flow media because of an increased hydraulic retention time and oxygen transfer (EPA, 1993).

Typical ammonia removal rates in TF systems have been reported to be 500-1000 mg $N/m^2.d$ (Barnes, 1983). Trickling filters seem to have a limiting final effluent quality of 1 - 2 mg/L ammonia nitrogen (EPA, 1977).

For a detailed design guide, see the EPA Manual Nitrogen Control (EPA, 1993)

6.5.1.3 Denitrification

Although denitrification is only indirectly related to ammonia removal, it is an important part of the nitrogen cycle and is included here for completeness. Nitrification and denitrification rarely occur in the same trickling filter. The low BOD_5 loadings necessary to promote the growth of nitrifiers simultaneously inhibits the growth of carbon requiring denitrifiers. In addition, nitrifiers require an aerobic environment whereas denitrifiers require an anaerobic environment. Effluent recycling encourages denitrification (Barnes, 1983), possibly by creating anaerobic zones in the trickling filter. However, for high denitrification rates, a separate unit must be used, adding an external carbon source. As an example, methanol was added to TF effluent (3.5 mg methanol/mg nitrate) in a mixed media filter achieving 95% nitrate removal (Duddles, 1977).

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Tchobanoglous G., and F.L.Burton. 1991. "Wastewater Engineering Treatment, Disposal, and Reuse 3rd Edition. Metcalf & Eddy, Inc., McGraw-Hill Publishing Company, New York.

6.5.2 Rotating Biological Contactors

Rotating biological contactors (RBCs) consist of large vertical disks (2-4 m diameter) mounted on a horizontal shaft (4-9 m long), rotated through a trough of settled wastewater, turning the attached media in and out of the trough. The bacteria responsible for treatment grow on the surface area of the disks. The disks are usually half submerged. This allows the bacteria to interact with the wastewater while submerged and to re-aerate in the other half of the cycle. With respect to toxicity reduction, specifically ammonia reduction, nitrification is the primary removal mechanism. A discussion of denitrification is included for completeness. Ammonia volatilization may contribute to overall ammonia reduction depending on pH, temperature and air currents. Volatilization is not, however, a major removal mechanism. A diagram of an RBC is given in Figure 6.5.2.

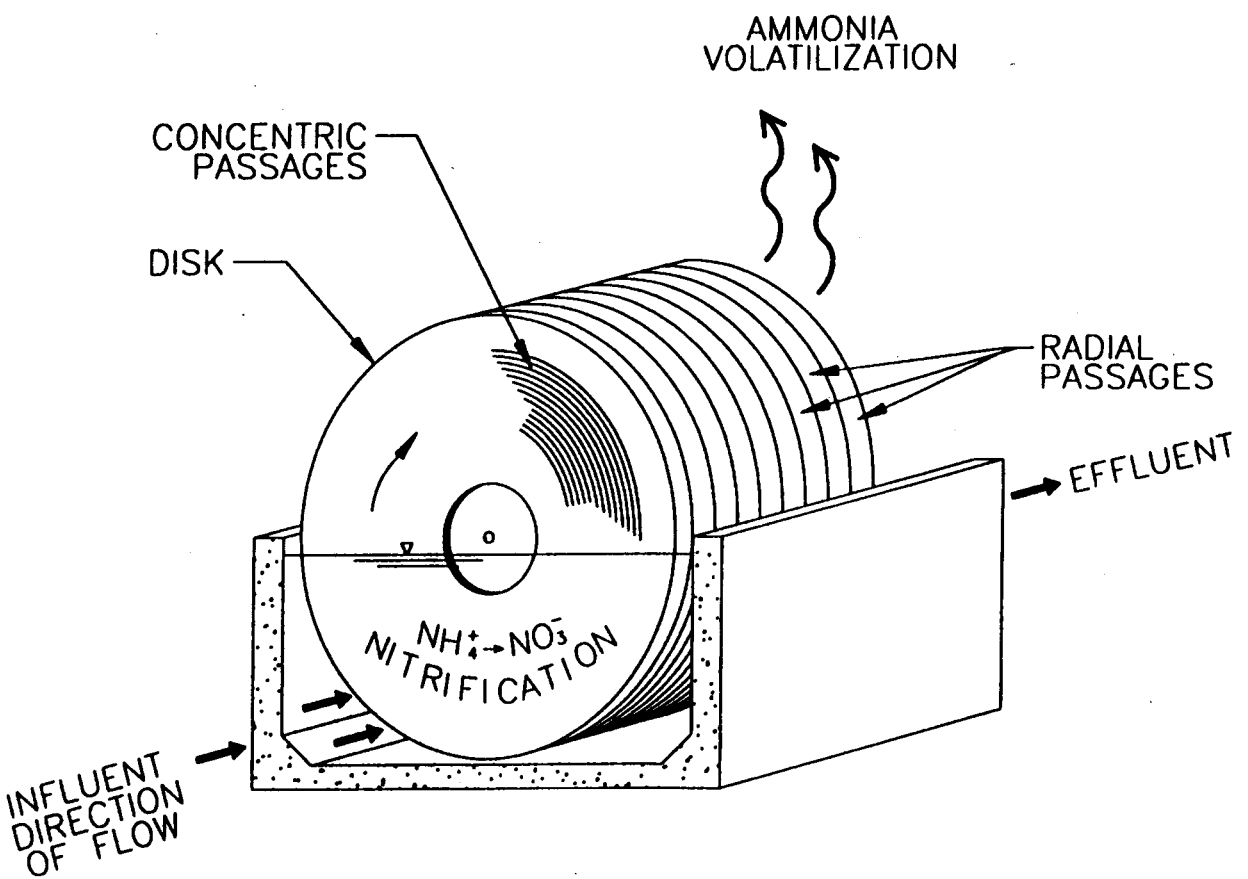
6.5.2.1 Nitrification

Nitrification is the primary mechanism responsible for ammonia removal in RBCs. Ammonia removal is dependent upon influent BOD_5, hydraulic loading rates, temperature, dissolved oxygen levels and pH.

Influent BOD_5

Influent BOD_5 concentration is the most important variable controlling nitrification (Klees and Silverstein, 1992). In order for nitrifying bacteria to significantly populate an RBC, the influent BOD_5 loading must be low. Nitrifying populations will not be established in any significant numbers where influent total biological oxygen demand ($TBOD_5$) is greater than 30 mg/L, or soluble biological oxygen demand ($SBOD_5$) is greater than 15-20 mg/L (Brenner et al, 1984, Klees and Silverstein, 1992). Recommended $TBOD_5$ loadings are 3.2 - 9.6 g/m^3 media per day for temperatures greater than 13°C (Tchobanoglous and Burton, 1991). For lower temperatures, lower BOD_5 loadings should be used. Extremely low $TBOD_5$ (< 10 mg/L) does not improve nitrification rates because the resulting thin biofilm is extremely prone to predation by higher life forms (snails, worms). BOD_5 loadings can be controlled by recirculation of RBC effluent to reduce the influent concentration (Klees and Silverstein, 1992).

Figure 6.5.2 Rotating Biological Contactor (RBC)



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Hydraulic Loading Rates

After BOD_5 concentrations, hydraulic loading rates are the second most important variable (Klees and Silverstein, 1992). Hydraulic loading rates are reported as flow rate per square metre of RBC surface area. Higher flow rates load a larger volume of wastewater on every square meter of surface area, resulting in less effective ammonia removal. Hydraulic loading rates can vary from 0.04 $m^3/m^2.d$ to 0.2 $m^3/m^2.d$. Suggested hydraulic loading rates based on influent and effluent ammonia concentrations are presented in Table 6.5.2.1.1.

Table 6.5.2.1.1 Hydraulic Loading and Ammonia Removal in RBCs at Temperatures Greater than 13°C

Hydraulic Load ($m^3/m^2.d$)	Influent NH_3-N (mg/L)	Effluent NH_3-N (mg/L)
0.05-0.08	30	1-3
0.06-0.12	20	1-3
0.06-0.16	15	1-3
0.1-0.2	10	1-3

(Adapted from Brenner et al., 1984 and Barnes and Bliss, 1983)

Data supporting Table 6.5.2.1.1 can be found in Klees and Silverstein (1992), and EPA (1977). The values in Table 6.5.2.1.1 should be reduced for colder temperatures (<13°C). The RBC surface area is determined from the flow and hydraulic loading; it is therefore also effected by temperature.

At the same hydraulic loading rate, systems built of stages will have greater nitrification (EPA, 1993). See manufacturer's data for the number of stages required for desired effluent characteristics.

Temperature

Temperature is the rate controlling factor at liquid temperatures below 13°C. At temperatures greater than 13°C, dissolved oxygen significantly controls nitrification (Brenner et al, 1984). RBC manufacturers should be contacted for RBC surface area temperature correction factors.

Dissolved Oxygen

A dissolved oxygen concentration of 2.0 - 3.5 mg/L is required for successful nitrification. At higher dissolved oxygen levels (> 3.5 mg/L) and low soluble BOD_5 levels (< 6-8 mg/L) higher life forms may become a serious hinderance to nitrification as they ingest the nitrifying bacteria (Brenner et al, 1984).

pH

The optimal pH for nitrification in RBCs is around 8.5 (Brenner et al., 1984). This higher pH discourages predators and provides sufficient alkalinity for the acid producing nitrification biochemical reaction.

Ammonia mass removal rates are reported in Table 6.5.2.1.2.

From Table 6.5.2.1.2, at 8.3°C ammonia removal decreases with increased influent ammonia concentration. Comparing the removal rates at lower and higher temperatures it is obvious that higher temperatures result in greater removal rates.

For a system operating under ideal conditions, removals of 1.5 to 1.7 g NH_3-N/m^2.d should be expected (Brenner et al, 1984). Other sources report removals of up to 2.5 g NH^3-N/m^2.d (Klees and Silverstein, 1992).

6.5.2.2 Denitrification

As with trickling filters, nitrification and denitrification do not usually occur in the same unit. A separate RBC using added methanol as a carbon source should be used for denitrification (EPA, 1977). 3.0 mg/L of methanol should be used for each mg/L of nitrate nitrogen. An extra aerated RBC stage may be required to oxidize any residual methanol (Brenner et al., 1984).

Table 6.5.2.1.2 Ammonia Removal Rates

Influent NH_3-N (mg/L)	Ammonia removal (g NH_3-N/m^2media.day)	Temperature (°C)	Reference
>5	1.46	12.8	Brenner et al.,1984
5-10	0.98	8.3	Brenner et al., 1984
14	0.49	8.3	Brenner et al., 1984
27.5	1.95	21.4	Klees and Silverstein, 1992

For complete design guidelines for RBC nitrification, see EPA Manual Nitrogen Control (EPA, 1993).

Denitrification RBCs use almost entirely submerged disks to attain the required anaerobic environment (Tchobanoglous and Burton, 1991). RBC units have not been used extensively for denitrification processes. Little information is available as to their performance. The Iron Bridge Plant in Orlando, Florida tried submerged RBC for denitrification and had major problems (W.Reed, pers. comm.)

Process Performance

In one study, removal rates of 10.4-14.7 g/m^3.d were reported. As to whether this process is independent of or dependent on influent nitrate concentrations is unclear (Brenner et al., 1984).

Hydraulic Loading

Hydraulic loading rates vary from 0.16 to 0.65 m^3/m^2.d for influent with 10-30 mg/L NO_3-N have been reported to achieve effluent with 1-10 mg/L NO_3-N at 12.8°C (Brenner et al., 1984).

Temperature

Manufacturers should be contacted for RBC surface area correction factors for low temperatures (i.e. lower than 13°C).

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Tchobanoglous G., and F.I.Burton. 1991. Wastewater Engineering Treatment, Disposal, and Reuse, 3rd Edition. Metcalf & Eddy, Inc., McGraw-Hill Publishing Company, New York.

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7 COMPARATIVE CAPITAL AND OPERATING COST ESTIMATING CONSIDERATIONS

7.1 KEY CAPITAL COST CONSIDERATIONS

The main items that should be considered when generating capital cost estimates are:

1. construction (including materials) works;
2. engineering services;
3. land acquisition;
4. contingencies.

The contingency cost (typically 15% of the construction cost) accounts for situations that develop during construction that were unknown at the time of detail design.

7.2 KEY OPERATING AND MAINTENANCE CONSIDERATIONS

The main items that should be considered when generating operating and maintenance costs are:

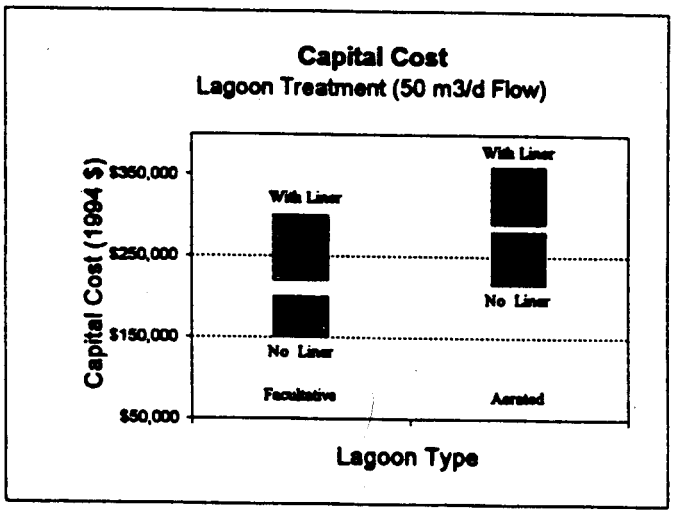
1. labour allowances (including backup coverage);
2. utilities;
3. maintenance supplies.

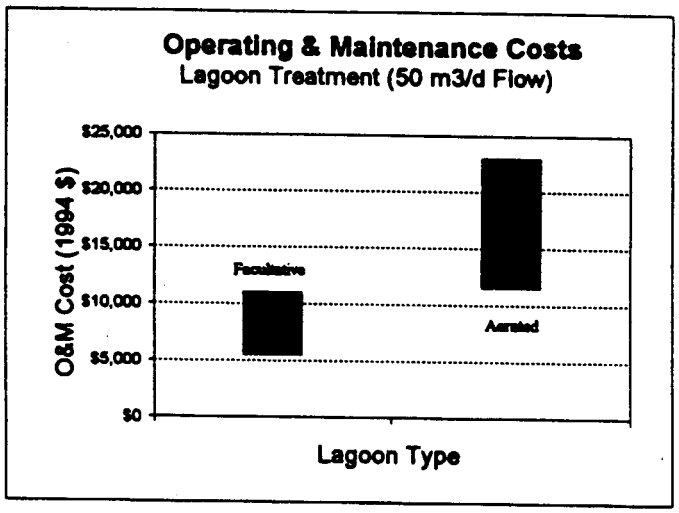
7.3 CAPITAL AND OPERATING & MAINTENANCE ESTIMATES

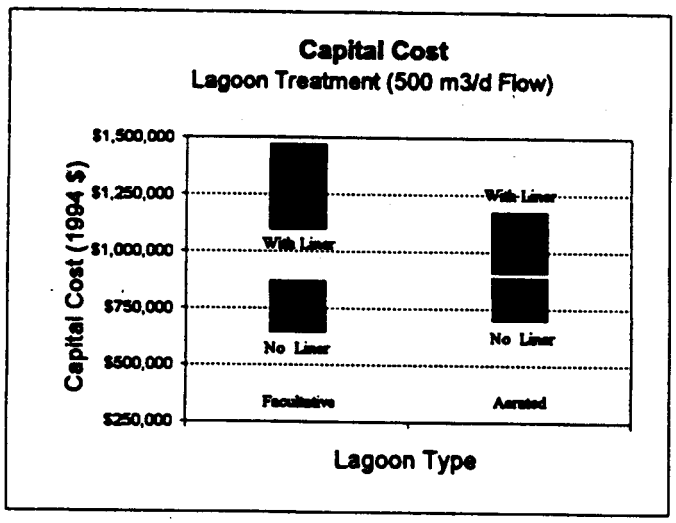
The capital and operating & maintenance cost estimates are presented in Figures 7.1 and 7.2. Cost data was generated from actual construction costs based on the authors' experience using unit costs, and on data provided in the following references:

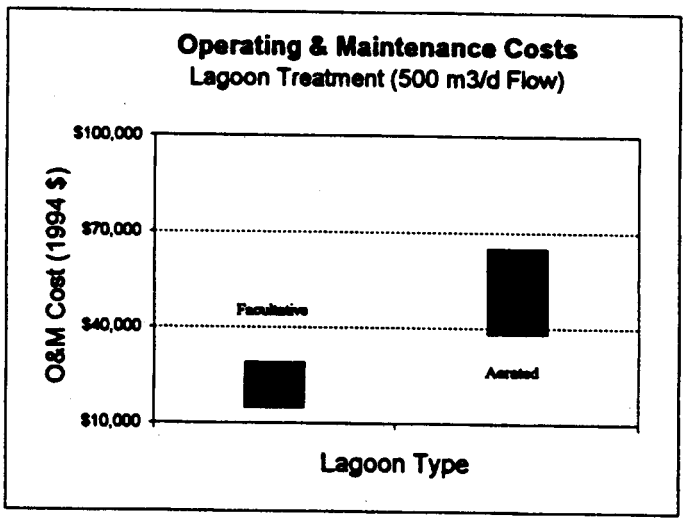
- EPA's 1992 Summary Report entitled "Small Community Water and Wastewater Treatment" (EPA, 1992),

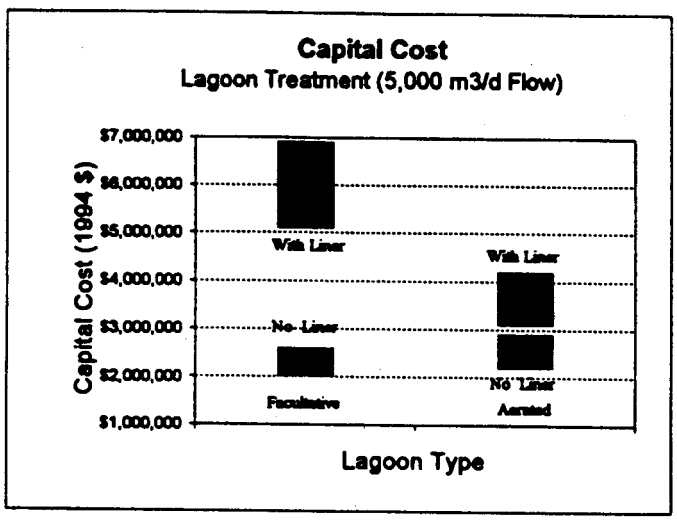
Figure 7.1 Comparative Costs Capital and Operating & Maintenance Lagoon Treatment











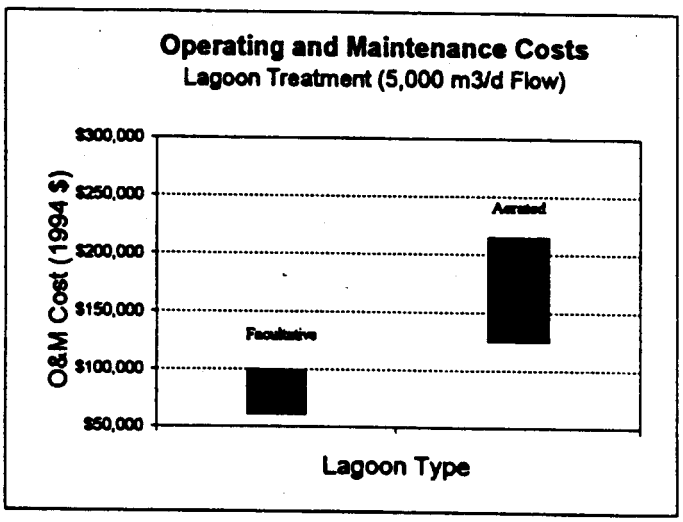
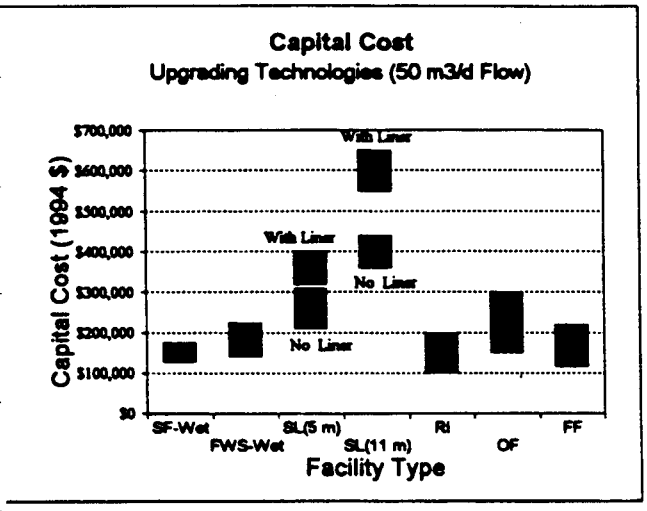
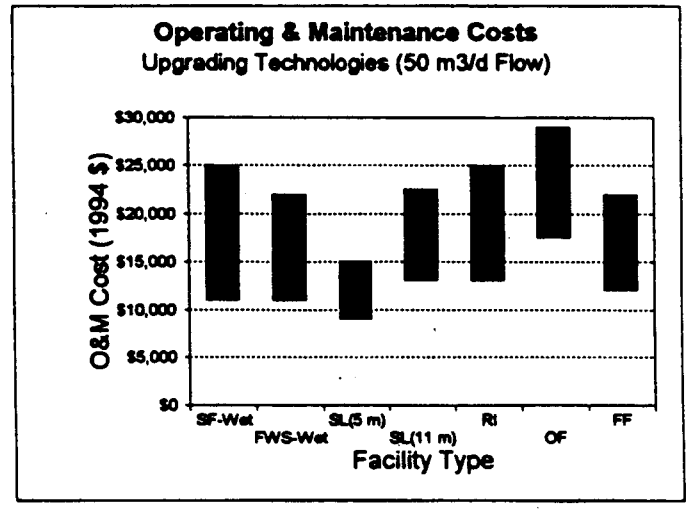
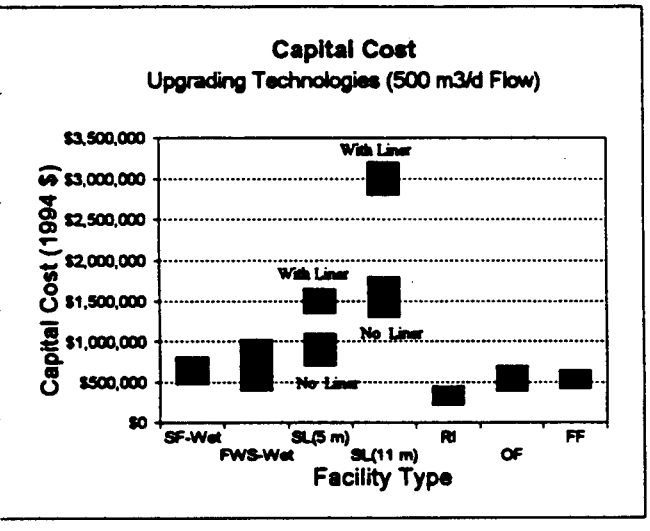
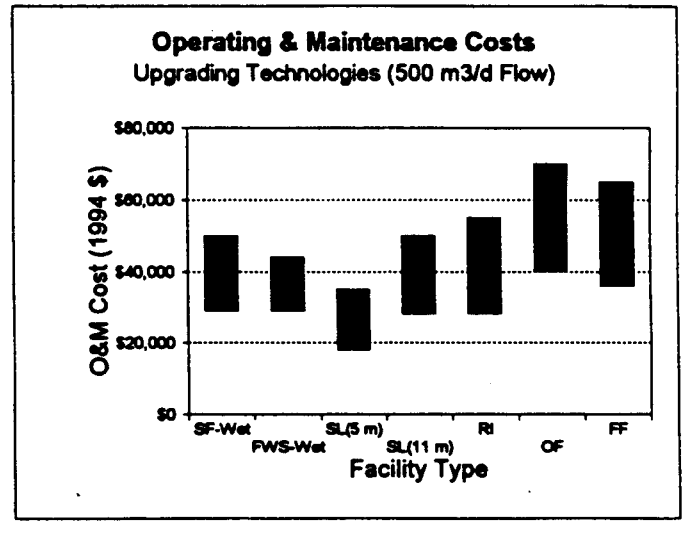


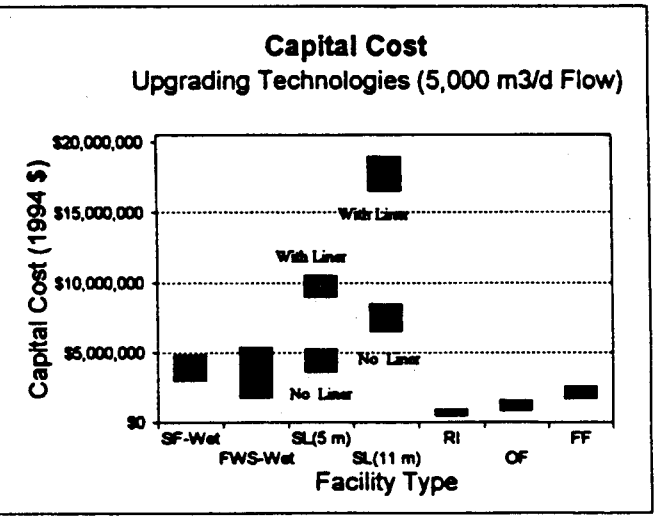
Figure 7.2 Comparative Costs Capital and Operating & Maintenance Upgrading Technologies

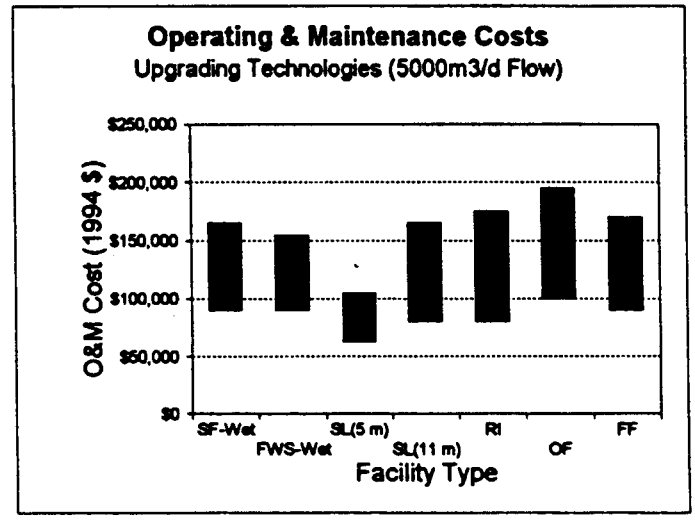












- EPA's 1984 publication entitled "The Cost Digest: Cost Summaries of Selected Environmental Control Technologies" (EPA, 1984), and
- Reed et al.'s paper entitled "Constructed Wetland Design, The Second Generation" (Reed et al., 1991),
- Stanley Associates's 1981 report entitled "Cost Estimating Guide for estimating cost of municipal sewage disposal and solid waste disposal" (Stanley, 1981).

Cost data from the U.S. was multiplied by 1.45 to allow for higher Canadian costs for materials and labour and by 1.40 to convert U.S. dollars to Canadian. Costs were adjusted to 1994 dollars using ENR's (Engineering News Record) construction cost index.
Costs for facultative and aerated lagoons are for conventional facilities treating raw wastewater. Costs for the upgrading technologies assumed treatment of secondary lagoon effluent, specifically for ammonia reduction. Given the variability in the geometry and size of natural wetlands, cost estimates were not made for this upgrading option. None of the capital costs include allowances for land purchases, additional conveyance of effluent, contingencies, engineering or legal services.

Given that EPA's cost data was derived for the U.S., and the authors' cost data was determined for either a site specific location or for a generic situation, the cost data should be used cautiously and only for comparison purposes rather than for estimating actual construction costs. Site specific factors (site topography, soil characteristics, water table location, availability of fill, proximity to construction and engineering services, etc.) will have a great impact on local costs. For example, a water balance is needed for storage lagoons to account for situations where precipitation or evaporation is excessive. This would have the effect of making storage lagoons financially impractical for coastal regions. In the case of constructed FWS wetlands, the presence of ideal topography and soil characteristic may preclude the need for a 1 m berm and synthetic liner, which account for a very significant portion of construction costs. Therefore, actual construction cost estimates for the various treatment technologies should be carried out for each individual site.

In reviewing the costs present in Figures 7.1 and 7.2, it must be remembered that of the upgrading technologies considered, all are applicable for coastal and south western B.C. In the rest of the Province, either storage (up to 11 months) or a combination of shorter term winter storage (about 5 months) with another upgrading technology, such as rapid infiltration, overland flow, fixed film systems (see Section 6) is feasible.

8 PROCESS DESIGN EXAMPLES

Three locations, representing different climatic zones in BC, are used here to illustrate the sizing of facultative and aerated lagoons. These locations are Prince George, Kamloops, and Abbotsford. The use of wetlands and storage cells as upgrading technologies for producing a non-toxic effluent is also examined. These examples are strictly for illustrative purposes; variable values given here should not be used in any site specific design. A flow chart summary of overall design procedure is given in Figure 8.1.

Assumed wastewater characteristics are:

BOD_5	=	150 mg/L
TN	=	30 mg/L
Alkalinity	=	70 mg/L
Flow (Q)	=	820 m^3/d

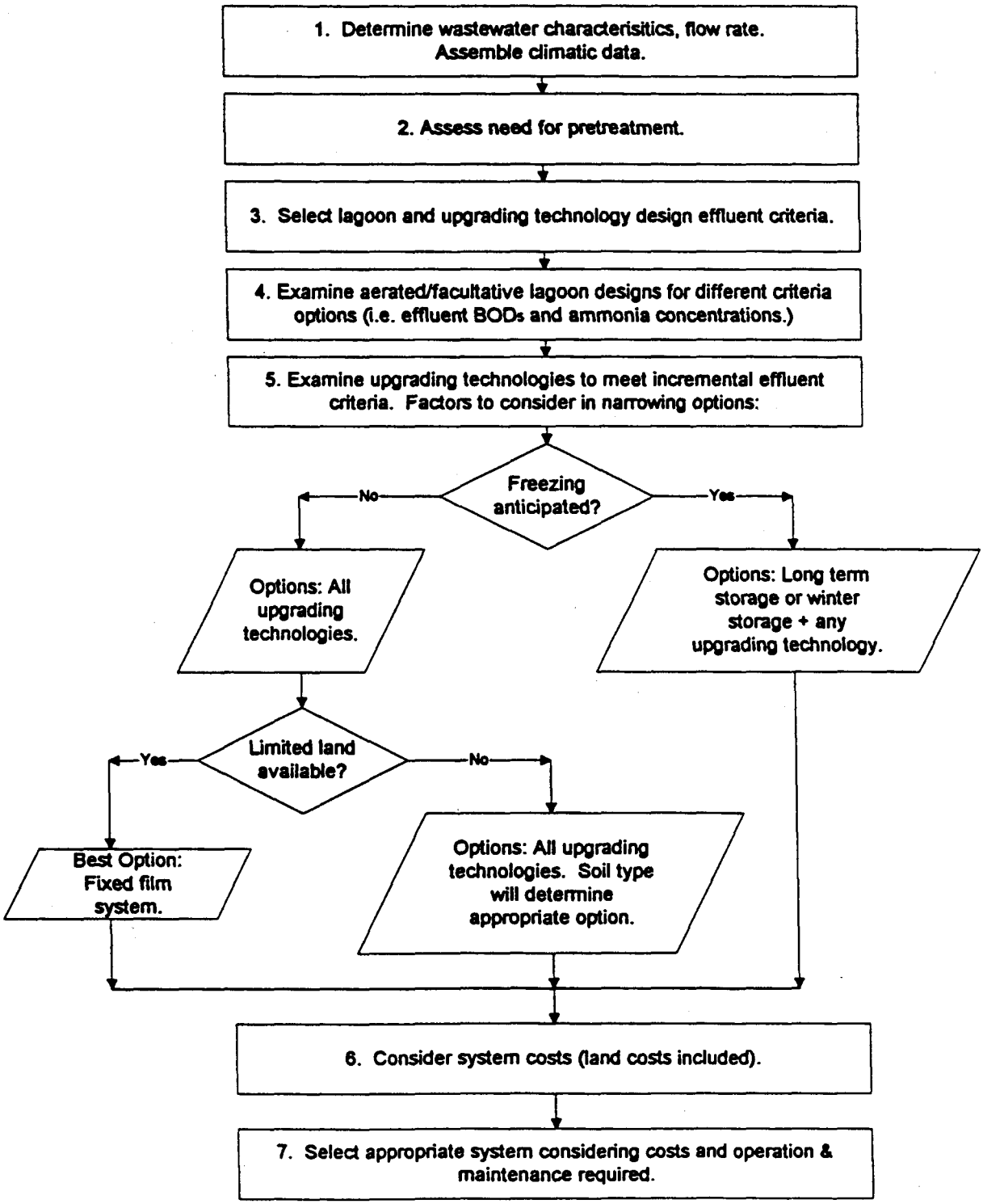
8.1 FACULTATIVE LAGOON AND STORAGE POND/WETLAND DESIGN EXAMPLES

8.1.1 Prince George

Prince George is located in a region where seasonal ice coverage is expected.

Because the flow is greater than 20 m^3/d, there is a minimum requirement of two cells in series for the facultative lagoon system (Section 4.1.1). The design number of cells is accordingly set to two.

FIGURE 8.1
Design Procedure For Lagoons and Upgrading Technologies



8.1.1.1 Anaerobic Primary Treatment Cells

As recommended, two anaerobic primary treatment cells in series are used (Section 4.1.2). With a water depth of 4 m each and a four day retention time, the area of each anaerobic lagoon can be approximated using the following equation (Smith, 1986):

$$V = d \cdot [(a + Sd) \cdot (b + Sd) + 0.0472 \cdot S^2 \cdot d^2]$$

Where:

- V = liquid volume of each lagoon cell (m^3)
- S = slope of berms (ratio of horizontal to vertical dimensions)
- d = liquid depth (m)
- a = length at base (m)
- b = width at base (m)

The total volume to be contained with four days retention time and a flow rate of 820 m^3/day is 3,280 m^3 which, divided between the two cells is 1,640 m^3 each. Setting the berm slope to 3:1 and the length:width ratio to 1:1, the base of each anaerobic cell is calculated as 8 m x 8 m. This is greater than the recommended minimum of 3 m x 3 m. The water surface dimensions of each cell would be 32 m x 32 m.

A sludge layer will develop faster in an anaerobic lagoon than in a facultative lagoon. As a result, the anaerobic lagoon must be desludged regularly.

A BOD_5 reduction of about 10% can be expected in primary treatment cells. The facultative lagoon influent BOD_5 is then estimated to be 135 mg/L.

8.1.1.2 Sizing

The facultative lagoon system is sized using Eq. 4-1.

$$\frac{C_e}{C_o} = e^{-k_p t}$$

A lagoon water temperature of 3 °C under the ice cover during the winter months is initially assumed. The value for k_p is then estimated using Eq.4-3 to be:

$$k_p = 0.045 \ (1.09)^{3-20} = 0.0104$$

Eq. 4-1 becomes

$$\frac{45}{135} = e^{-0.0104 \cdot t}$$

An effluent BOD_5 value of 45 mg/L is chosen here to represent secondary treatment. For lagoon effluent which feeds into a nitrification system, the BOD_5 value must be further lowered to 15 mg/L so that nitrification can proceed. This adjustment can be made in the sizing of either the lagoon or the upgrading technology system.

Solving for t, the facultative lagoon system retention time is found equal to 106 days. This is greater than the recommended minimum of 60 for facultative lagoons. The volume required is then

$$V \ (m^3) = 820 \ \frac{m^3}{d} \cdot 106 \ d = 86{,}920 \ m^3$$

Selecting a liquid depth of 1.2 m (Section 4.1.3), the length and width of the base of each lagoon cell can be determined using the equation presented in Section 8.1.1.1. Substituting in the following values:

V = 86,920 / 2 cells = 43,460 m^3/cell
S = 3
d = 1.2 m
a = 3 * b (i.e. length is 3 times width)

and solving for a and b, the base dimensions of each lagoon cell can be determined as:

a (length) = 322 m
b (width) = 108 m

The cell water surface area is then calculated for use in determining whether the water temperature assumption of 3°C was appropriate:

a' (water depth length)	= a + (2 * S * d)	= 329 m
b' (water depth width)	= b + (2 * S * d)	= 115 m
Water surface area	= 37,835 m^2	

The overall average freezing air temperature for Prince George is determined by referring to the Canadian Climate Normals for the months of December through March (-7.9°C, -12.1°C, -6.1°C and -1.8°C), the average freezing temperature is taken as -7°C (T_a). Using the average winter freezing temperature, assuming an influent temperature of 10°C (T_i), and setting f=0.5 as recommended, Eq. 4-4 calculates the water temperature as:

$$T_w = \frac{AfT_a + QT_i}{Af + Q}$$

$$T_w = \frac{(37,835)(0.5)(-7) + (820)(10)}{(37,835)(0.5) + 820} = -6.3°C$$

Because this value is less than zero, it is less than the minimum value of 2°C. The difference between the previously assumed value of 3°C and the minimum of 2°C is only 1°C, which is an acceptable difference requiring no further iterations. A water temperature value of 3°C will be used for further design.

The total depth of the lagoon involves several terms:

d = liquid depth
+ sludge depth
+ ice cover allowance
+ freeboard allowance

Sludge

The sludge depth is first calculated by estimating the sludge volume using Eq.4-5.

$$Sludge\ \ volume\ \ m^3 = \frac{1.5 m^3}{1{,}000\ m^3 of\ sewage}(Q)(P)$$

Assuming a design period of 20 years, and solving,

$$Sludge\ \ volume\ \ m^3 = \frac{1.5}{1{,}000}\left(820\ \frac{m^3}{d}\right)(20\ yr)\left(365\ \frac{d}{yr}\right) = 8{,}979\ m^3$$

This estimated volume could be reduced in making an allowance for the sludge removed in the primary treatment anaerobic lagoons.

In order to determine the depth, the sludge volume is divided by the area at the bottom of the lagoon system.

$$Sludge\ \ Depth = \frac{8{,}979}{(108)(322)(2\ cells)} = 0.26\ m$$

Ice Cover

An allowance for ice cover is determined using the Stefan equation, Eq.4-6.

$$h = \epsilon\ (SFI)^{0.5}$$

Referring to the Canadian Climate Normals, the degree-days below zero value for Prince George is 1057.9 °C d. An ϵ value of $8.5*10^{-5}$ m/°C$^{1/2}$/s$^{1/2}$, appropriate for medium sized lakes with moderate snow cover, is chosen. Applying these values,

$$h = 8.5*10^{-5}\ [(1{,}057.9)(24)(3{,}600)]^{0.5} = 0.81\ m$$

Freeboard

The freeboard allowance in this case for lagoons with seasonal ice cover is 0.3 m (Section 4.4.13). The designing engineer may opt to increase this to 1.0 m.

Therefore the total depth is:

total depth = 1.2 + 0.26 + 0.81 + 0.3 = 2.57 m

8.1.1.3 BOD$_5$ Loading

Using these values, then check that the BOD$_5$ loadings are less than 22 kg/ha/d as required. The daily BOD$_5$ loading is:

$$820\,\frac{m^3}{d} \cdot 135\,\frac{mg}{L} \cdot \frac{1000\,L}{1m^3} \cdot \frac{1kg}{1{,}000{,}000\,mg} = 110.7\,\frac{kg}{d}\,BOD_5$$

$$\frac{110.7\,\frac{kg}{d}}{(115\ m)(329\ m)(2\ cells) \cdot 10^{-4}} = 14.6\,\frac{kg}{ha \cdot d}\,BOD_5$$

This value is less than the recommended maximum system loading of 22 kg/ha/d. Because there are two cells, the BOD$_5$ loading on the first cell would be doubled to almost 30 kg/ha/d. This is within the recommended loading for the first cell of 40 kg/ha/d.

8.1.1.4 Nitrogen Removal

Estimates of the anticipated winter nitrogen removal can be made using the equations presented in Table 3.2.4. Choosing Model 1, first calculate the pH using the alkalinity of 70 mg/L as defined in the wastewater characteristics.

$$pH = 7.3\ e^{0.0005\ (70mg/L)} = 7.56$$

Assume a water temperature of 3 °C as previously established.

$$k_T = 0.0064\ \ (1.039)^{3-20} = 0.0033$$

$$= 30 \cdot e^{-0.0033[106 \cdot 60.6\ (7.56-6.6)]} = 17.5\,\frac{mg}{L}$$

$$N_e = N_o \, e^{-k_T \, [t \cdot 60.6 \, (pH-6.6)]}$$

For comparison, Model 2 is applied.

$$N_e = N_o \, [1 + (106)(0.000576 \, (3) - 0.00028) \, e^{(1.08-0.042(3))(7.56-6.6)}]^{-1} = 21.7 \; mg/L$$

A design value for nitrogen removal is made by averaging the results of the two models, resulting in 19.5 mg/L. These models can also be used to estimate summer nitrogen removals, using a higher pH value to account for the pH increasing presence of algae. For example, these models predict summer effluent total nitrogen concentrations of 10.3 and 15.0 mg/L using a pH of 8.3 and a wastewater temperature of 14°C.

During winter months, it is conservative to assume that all the effluent nitrogen is in the form of ammonia-nitrogen. During the summer, however, more than half of the effluent nitrogen may be in the form of organic-nitrogen due to high algal populations.

8.1.1.5 Upgrading Technologies

Options for upgrading technologies examined are storage ponds and wetlands.

Storage Ponds

Storage ponds as a means of further effluent treatment require a minimum of 5 months of ice free storage, preferably being discharged in the fall. In the case of Prince George, these five months would be April through August, with a September discharge.

Depending on predicted effluent ammonia nitrogen concentrations, it may be possible in some cases to allow a direct discharge from the facultative lagoon to receiving waters in the summer. However, the estimated summer effluent nitrogen concentrations calculated in Section 8.1.1.4 would suggest that lagoon summer effluent also requires treatment. Without a detailed analysis, for this example it is assumed that effluent from the facultative lagoon will required storage for twelve months of the year. The storage pond would function primarily as a holding pond for lagoon effluent from October through April, and as an upgrading treatment technology May through September.

The pond would be designed to accommodate eleven to twelve months of lagoon effluent, depending of the storage pond discharge schedule.

Wetlands

Because the flow rate (820 m^3/d) is greater than 400 m^3/d, a FWS wetland is likely to be more economic (Section 5.2).

When considering a FWS wetland in a colder climate you must first ensure that a wetland is feasible in the given climatic conditions. The main hazard is that the wetland might freeze to such a depth that it would no longer be able to effectively treat the wastewater. A different variation of the Stefan equation is used to determine the depth of ice that will form in a wetland over the winter months, Eq.5-28.

$$h = (m)[(T_M - T_{AIR})(t)]^{1/2}$$

As previously discussed, the average freezing temperature is estimated as -7°C. Assuming an (m) value of 0.018 $m/°C^{1/2}/d^{1/2}$ for open water zones with snow, the estimated depth of ice is:

$$h = 0.018 \cdot [(0-(-7))(4 \cdot 30)]^{1/2} = 0.5 \ m$$

Because the recommended maximum wetland depth is 0.45 m, year round use of a FWS wetland is not feasible in this climate. Therefore, some winter storage would be required followed by seasonal wetland operation.

8.1.2 Kamloops

Kamloops is also expected to have seasonal ice coverage. The same methodology is then followed as previously described for Prince George with some temperature related changes. The same influent characteristics are assumed.

The lagoon systems is designed for two cells in series.

8.1.2.1 Anaerobic Cells

The anaerobic cell sizing is the same as for Prince George in Section 8.1.1.1. The same BOD_5 reduction is assumed.

8.1.2.2 Sizing

Because the winter lagoon water is also under ice cover, the same water temperature estimation of 3°C is made. This assumption implies the facultative lagoon design process is almost the same for both regions. The base and water surface dimensions of the lagoon are as given in Section 8.1.1.2. A thermodynamic check of the water temperature (Eq. 4-4) using Kamloops' average winter air temperature still results in a negative water temperature. Therefore an assumed lagoon water temperature of 2°C applies; the initial assumption of 3°C is close enough to this value not to require any further iterations.

Winter air temperature differences do effect the lagoon total depth. The ice cover depth is reduced. The components of total depth are:

Sludge Depth

The sludge depth is again calculated as 0.26 m. This allowance could be reduced because anaerobic cells precede the lagoon.

Ice Cover

The ice cover is calculated in a similar manner using a different SFI value.

$$h = 8.5 \cdot 10^{-5} \, [(418.4)(24)(3{,}600)]^{0.5} = 0.51 \ m$$

Freeboard

A freeboard of 0.3 m is again recommended for climates with seasonal ice coverage. The designing engineer may choose to increase this allowance to 1.0 m.

The total depth is then

total depth = 1.2 + 0.26 + 0.51 + 0.3 = 2.27 m

8.1.2.3 BOD_5 Loadings

Because the two lagoon water designs depths are identical, the BOD_5 loadings are identical. See Section 8.1.1.3.

8.1.2.4 Nitrogen Removal

The nitrogen removal estimates are the same as those given for Prince George in Section 8.1.1.4.

8.1.2.5 Upgrading Technologies

Storage Lagoon

Storage lagoons require 5 months of ice free storage, preferably being discharged in the fall. In the case of Kamloops, these five storage months would be April through August, with a September discharge. Depending on predicted effluent ammonia nitrogen concentrations, it may be possible to allow a direct discharge from the facultative lagoon to receiving waters in the summer.

A similar design to that used for Prince George should be used.

Wetlands

Because the flow of 820 m^3/d is greater than 400 m^3/d, a FWS wetland is likely to be more economic (Section 5.2).

Again, the main concern when considering wetlands for cold/temperate climate regions is to first determine the depth to which ice will form and compare this to the maximum allowable wetland design depth. The different variation of the Stefan equation is again used to determine the depth of ice that will form in a wetland over the winter months, Eq.5-28.

$$h = (m)[(T_M - T_{AIR})(t)]^{1/2}$$

For Kamloops, the overall average freezing air temperature was determined. For the months of December through February (-2.8°C, -6.1°C, and -1.3°C), the average temperature is taken as -3.4°C. Assuming an (m) value of 0.018 $m/°C^{1/2}/d^{1/2}$ for open water zones with snow, the estimated depth of ice is:

$$h = 0.018 * [(0-(-3.4))(3*30)]^{1/2} = 0.3\ m$$

It is recommended that for regions where ice penetration is greater than 75% of design wetland depth, wetlands are not used during the winter months without careful consideration. Using a maximum

wetlands depth of 0.45 m, 0.3 m ice formation is less than this maximum and so year-round wetland use may be feasible. A more detailed analysis of this possibility is done.

First the area required to reduce the BOD_5 from 45 to 15 mg/L is determined. Assume a wastewater temperature Eq. 5-2 gives a k_T of

$$k_T = 0.678 \ (1.06)^{2-20} = 0.238$$

The total area required for BOD_5 removal is then calculated using a flow depth(y) of 0.15 m (0.45 m less 0.3 m) and a porosity(n) of 0.7,.

$$A_s = \frac{(820)\ \ln(\frac{45}{15})}{(0.238)(0.15)(0.7)} = 36{,}049 \ m^2$$

Area required for nitrogen reduction is calculated in a similar manner. K_T is estimated as (Section 5.5.3):

$$K_T = 0.2187 \ (1.048)^{(T-20°)} = 0.2187 \ (1.048)^{(2°-20°)} = 0.094$$

Then using Eq.5-5

$$A_s = \frac{(820)\ \ln(\frac{19}{5})}{(0.094)(0.15)(0.7)} = 110{,}912 \ m^2$$

The wetland surface area required is then 110,912 m^2. Effluent BOD_5 for the given area is calculated to be less than 10 mg/L.

The individual wetland cells should be no greater than 4 to 6 ha in size (Section 5.7.1). The total area is therefore divided over two individual cells (recommended minimum of two cells , see Section 5.7.1.), each with an area of approximately 55,460 m^2.

Eq. 5-32 can then be used to calculate the maximum length of each wetland cell:

$$L = \frac{A_s\, y^{2/3}\, m^{1/2}\, 86{,}400}{a\, Q}$$

Assuming a head differential of 0.2 (20%) over the length of the lagoon and an a value of 1.6 s*m$^{1/6}$ for moderately dense vegetation,

$$L = \left[\frac{(55{,}560)(0.15)^{2/3}(0.2)^{1/2}(86{,}400)}{(1.6)(\frac{820}{2})}\right]^{2/3} = 755 \ m$$

Given the surface area previously calculated, this would result in a width of 73.5 m, and a corresponding L:W ratio of almost 10:1. The recommended L:W ratio is 3:1. With such a ratio, the length and width are

Length	=	411 m
Width	=	135 m

This length is within the limits of a maximum length.

Having assumed a wetland water temperature and determined the surface area required to achieve the desired ammonia removal, the next step is to use heat transfer models to calculate the theoretical water temperature in the wetland based on these dimensions. The wetland water temperature (T_W) under an ice cover can be estimated with Eq.5-25

$$T_W = T_M + (T_O - T_M)\, e^{-\left[\frac{U_i (x - x_o)}{(\delta)(y)(v)(C_p)}\right]}$$

Eq. 5-26 defines U_i as

$$U_i = (\Phi)\, \frac{(v)^{0.8}}{(y)^{0.2}}$$

The velocity (v) term can be determined from :

$$v = \frac{Q}{Wy} = \frac{\frac{820}{2}}{(135)(0.15)} = 20.2\ \frac{m}{d} = 0.234 \cdot 10^{-3}\ \frac{m}{s}$$

Using a Φ value of 1622 $J/m^{2.6}/s^{0.2}/°C$, and a depth of 0.15 m, the U_i value is 2.95 $W/m^2/°C$. Eq. 5- 25 becomes:

$$T_W = 0 + (3 - 0)\ e^{-\left[\frac{(2.95)(411)}{(1000)(0.15)(0.234 \cdot 10^{-3})(4215)}\right]} = 0.8 \cdot 10^{-3}\ °C$$

Because this thermal model predicts a temperature less than 1°C, a wetland may not be physically capable of winter operations. Instead, a combination of winter storage and seasonal wetland treatment should be considered if wetlands are to be used in such a climate. Economically, it may also not be possible to build such a large lagoon for year round use. Seasonal use would appear to be the only economically viable option.

8.1.3 Abbotsford

Abbotsford is not expected to have seasonal ice coverage. The method used previously for both Prince George and Kamloops is altered accordingly.

The lagoon systems contains two cells in series as recommended.

8.1.3.1 Anaerobic Cells

The anaerobic cell sizing is the same as for Prince George in Section 8.1.1.1. The same BOD_5 reduction is assumed.

8.1.3.2 Detention Time and Volume

For sizing the lagoons, a lagoon water temperature of 5°C is estimated. Because there is no ice cover, a different equation to estimate k_p is used (Section 4.1.3).

$$k_p = 0.071\ (1.09)^{5-20} = 0.0195$$

$$\frac{45}{135} = e^{-0.0195 \cdot t}$$

Solving, t = 56 days. This is less than the recommended minimum of 60 days for facultative lagoons. The retention time is therefore set at 60 days.

8.1.3.3 Sizing

Using a flow rate of 820 m^3/d and a detention time of 60 days, the total volume is then 49,200 m^3. Because there is no ice cover, the liquid depth is set at 1.5 m (Section 4.1.3). Using the a berm slope of 3:1 and a L:W ratio of 3:1, the individual cell base dimensions are:

a (length)	=	210 m
b (width)	=	70 m

The dimensions at the liquid depth are calculated to be:

a' (water depth length)	=	219 m
b' (water depth width)	=	79 m
Total water depth surface area	=	17,300 m^2

Then check to see if the original water temperature assumption of 5°C was reasonable using Eq. 4-4.

$$T_w = \frac{AfT_a + QT_i}{Af + Q}$$

The average winter air temperature in Abbotsford is taken as 3.4°C. Eq. 4-4 then becomes:

$$T_w = \frac{(17{,}300)(0.5)(3.4) + (820)(10)}{(17{,}300)(0.5) + 820} = 4.0°C$$

The assumed temperature (5°C) and the calculated temperature (4.0°C) are within the 1°C value required; no further iterations are required.

The total depth of the lagoon must then be calculated.

Sludge Depth

The sludge volume is the same as before. The sludge depth can then be calculated using the base width and length.

$$Sludge\ \ Depth = \frac{8{,}979}{(205)(68)(2\ \ cells)} = 0.32\ \ m$$

This may be reduced as the lagoon is preceeded by anaerobic lagoons.

Freeboard

A minimum freeboard of 0.6 m is required in areas with no ice allowance. The design engineer may choose a freeboard of 1.0 m.

The total depth is then

total depth = 1.5 + 0.32 + 0.6 = 2.42 m

8.1.3.4 <u>BOD_5 Loadings</u>

The daily BOD_5 load of 110.7 kg/d BOD_5 was calculated in Section 8.1.1.3. The system BOD_5 loading is then:

$$\frac{110.7\ \frac{kg}{d}}{(219)(79)(2\ \ cells) \cdot 10^{-4}} = 32\ \frac{kg}{ha \cdot d}$$

This value is less than the recommended 45 kg/ha/d.

8.1.3.5 <u>Nitrogen Removal</u>

The winter nitrogen removal is estimated using both models, as illustrated in the Prince George example. Using temperatures and retention times from Abbotsford calculations, the estimated winter effluent total nitrogen concentrations are 22.5 and 19.9 mg/L, averaging at 21.2 mg/L.

8.1.3.6 Upgrading Technologies

Storage Lagoon

It is recommended that storage lagoons provide 5 months of ice free storage, preferably being discharged in the fall. In the case of Abbotsford, these five months could theoretically be anytime of the year as there is no ice coverage period; the most effective treatment would occur over the summer months. The five months whose discharge is of most concern are January through May. All effluent from the lagoon from January through May would be directed to the storage lagoon, stored over the summer and discharged in September. Effluent for June through December could be directly discharged provided the estimated effluent ammonia concentrations met the design requirements. The total volume of wastewater that would require storage over five months at 820 m^3/d would be 123,000 m^3. Using a L:W ratio of 3:1 as recommended (Section 6.1.2), a berm slope of 3:1, a depth of 2.5 m, and the equation presented in Section 8.1.1.1, the storage pond base dimensions are:

a (length)	=	369 m
b (width)	=	123 m

Wetlands

Ice cover is not a concern for the Abbotsford area, so wetland systems are potentially operational year round. Because the flow rate of 820 m^3/d is greater than 400 m^3/d, a FWS system is likely to be more economic (Section 5.2). Accordingly, a FWS wetland is chosen.

The area required for BOD_5 reduction from 45 to 15 mg/L is calculated assuming a wastewater temperature of 5°C, a flow depth of 0.3 m and a porosity (n) of 0.7.

$$k_t = 0.678 \ (1.06)^{5-20} = 0.283$$

$$A_s = \frac{(820)\ \ln(\frac{45}{15})}{(0.283)(0.3)(0.7)} = 15{,}158 \ m^2$$

K_T for nitrogen reduction is estimated as (Section 5.5.3):

$$K_T = 0.2187 \quad (1.048)^{(T - 20°)} = 0.2187 \quad (1.048)^{(5°-20°)} = 0.1083$$

Then using Eq.5-5

$$A_s = \frac{(820)\ \ln(\frac{21.2}{5})}{(0.1083)(0.3)(0.7)} = 52{,}084 \ m^2$$

The wetland area required is then 52,084 m^2. This area could be contained in the minimum of two wetland cells, each 26,050 m^2.

Eq. 5-32 can then be used to calculate the maximum length of the wetland:

$$L = \frac{A_s\ y^{2/3}\ m^{1/2}\ 86{,}400}{a\ Q}$$

Assuming a head differential(m) of 0.2 (20%) over the length of the wetland, an (a) value of 1.6 $s*m^{1/6}$ for moderately dense vegetation, and a flow to each cell of 410 m^3/d,

$$L = [\frac{(26{,}050)(0.3)^{2/3}(0.2)^{1/2}(86{,}400)}{(1.6)(410)}]^{2/3} = 1{,}565 \ m$$

Given the surface area calculated above, this would result in a width of 16.6 m, and a corresponding L:W ratio of almost 94:1. The recommended L:W ratio is 3:1. With such a ratio, the length and width are:

Length	=	280 m
Width	=	93 m.

Obviously this length is within the maximum length limit. Having assumed a wetland water temperature and the surface area required to achieve the desired ammonia removal, the next step is to use heat transfer models to calculate the theoretical water temperature in the wetland based on this surface area. The wetland water temperature can be estimated with Eq. 5-23

$$T_W = T_{AIR} + (T_O - T_{AIR})\ e^{-[\frac{U_s(x - x_o)}{(\delta)(y)(v)(C_p)}]}$$

The velocity (v) term can be determined from

$$v = \frac{Q}{Wy} = \frac{410}{(93)(0.3)} = 14.7\ \frac{m}{d} = 0.1701 \cdot 10^{-3}\ \frac{m}{s}$$

Assuming a U_s value of 5, and taking the average winter air temperature to be 3.7°C over the months November through February, Eq. 5-23 is then

$$T_W = 3.7 + (10 - 3.7)\ e^{-[\frac{(5)(280)}{(1000)(0.3)(0.00017)(4215)}]} \approx 3.7°C$$

The estimated value of 5°C and the calculated value of 3.7°C differ by more than 1°C, so further iterations are necessary.

Instead of immediately changing the initial temperature, a deeper wetland design is evaluated for illustrative purposes. Assuming a flow depth of 0.45 m and a porosity(n) of 0.7,

$$A_s = \frac{(820)\ \ln(\frac{21.2}{5})}{(0.1083)(0.45)(0.7)} = 34{,}723\ m^2$$

The area required for BOD_5 removal is similarly calculated to be 10,106 m^2. The area required is then 34,723 m^2.

Eq.5-32 can then be used to calculate the maximum length of the wetland:

$$L = \frac{A_s\ y^{2/3}\ m^{1/2}\ 86{,}400}{a\ Q}$$

Assuming a head differential(m) of 0.2 (20%) over the length of the wetland and an (a) value of 1.6 $s*m^{1/6}$ for moderately dense vegetation,

$$L = [\frac{(17{,}360)(0.45)^{2/3}(0.2)^{1/2}(86{,}400)}{(1.6)(410)}]^{2/3} = 2{,}455\ m$$

Given the surface area calculated above, this would result in a width of 7 m, and a corresponding L:W ratio of almost 347:1. The recommended L:W ratio is 3:1. With such a ratio, the length and width are:

Length	=	229 m
Width	=	76 m.

This is obviously safely within the maximum length limits.

Now heat transfer models are used again to calculate the theoretical water temperature in the wetland based on this surface area. The wetland water temperature can be estimated with Eq. 5-23

$$T_W = T_{AIR} + (T_O - T_{AIR})\, e^{-\left[\frac{U_s (x - x_o)}{(\delta)(y)(v)(C_p)}\right]}$$

The velocity (v) term can be determined from

$$v = \frac{Q}{Wy} = \frac{410}{(76)(0.45)} = 11.9\ \frac{m}{d} = 0.1388 \cdot 10^{-3}\ \frac{m}{s}$$

Assuming a U_s of 5 and an influent water temperature (T_O) of 10°C, Eq. 5-23 is then

$$T_W = 3.7 + (10 - 3.7)\, e^{-\left[\frac{(5)(229)}{(1000)(0.45)(0.00014)(4215)}\right]} \approx 3.7°C$$

Changing the wetland depth did not have a significant effect on the wetland water temperature. Iterations must be repeated using a different temperature.

Assuming a water temperature of 4°C and following the procedure just outlined, a wetland with the following characteristics is calculated:

Length	=	234 m
Width	=	78 m
Depth	=	0.45 m
One Cell Surface Area	=	18,203 m^2

Using the thermal equations to check the wetland water temperature, a value of 3.8°C is calculated using variables associated with 4°C. This design meets the iteration requirements.
Because of land/economic limitations, it may be more practical to store lagoon effluent over the winter and treat the summer and stored winter effluent in a wetland over the summer months.

8.2 AERATED LAGOON AND STORAGE POND/WETLAND DESIGN EXAMPLES

8.2.1 Prince George

8.2.1.1 Anaerobic Primary Treatment Cells

The same design procedure as that outlined in Section 8.1.1.1 applies here for anaerobic primary treatment cells.

8.2.1.2 Sizing

A minimum of two cells in an aerated lagoon system are recommended, three to four being preferred (Section 4.2.2). In this example, three cells will be used. Eq. 4-8 is used to determine a k_{pm} value. A wastewater temperature of 3°C is assumed.

$$\frac{k_{pm}}{k_{pm20}} = (\Theta_w)^{T_w - 20}$$

$$k_{pm} = 0.276 \cdot (1.036)^{3-20} = 0.15$$

The detention time can then be calculated using Eq. 4-7.

$$t = \frac{n}{k_{pm}} \left[\left(\frac{C_o}{C_n}\right)^{\frac{1}{n}} - 1 \right]$$

$$t = \frac{3}{0.15} \left[\left(\frac{135}{15}\right)^{\frac{1}{3}} - 1 \right] = 21.6 \ d$$

However, the minimum detention time for cold climates is 30 days. This is then the determining factor in sizing the aerated lagoon.

The total volume is

$$820 \ \frac{m^3}{d} \cdot 30 \ d = 24{,}600 \ m^3$$

Using a liquid depth of 3.7 m (Section 4.2.4), a length to width ratio of 1:1 (Section 4.6.7), and a berm slope of 3:1 (Section 4.6.14), the lagoon cells are sized, using the equation presented in Section 8.1.1.1, as:

Width and length at base	=	36 m
Base area	=	1,296 m^2
Width and length at water surface	=	58 m
Water surface area	=	3,364 m^2

Using Eq.4-9 to check the thermodynamic balance, a negative wastewater temperature value results. In this case, a default lagoon water temperature of 2°C should be used. The assumed value of 3°C is within reasonable range of this value. No further iterations are required.

The full depth of each aerated lagoon must also take into consideration the sludge depth, ice cover and free board allowances. The volume of sludge generated is the same as for facultative lagoons, ie. 8,979 m^3 over a 20 year period. This estimate value could be reduced as primary anaerobic class have been incorporated into the design. Because of the much smaller dimensions of aerated lagoons, assume here that the sludge is removed every 5 years. In this case the maximum sludge depth in each cell is:

$$\frac{8{,}979 \ m^3}{1{,}296 \ m^2 \cdot 3 \ cells \cdot 4 \ removals \ over \ 20 \ years} = 0.58 \ m$$

The estimation for ice cover in Section 8.1.1.2 of 0.8 m also applies to aerated lagoons. For an area with ice cover, a 0.3 m freeboard is recommended. This could be increased to 1.0 m by the design engineer.

The total depth is then:

total depth = 3.7 + 0.58 + 0.81 + 0.3 = 5.39 m

The surface dimensions can then be calculated using the same L:W ratio (1:1), berm slope (3:1), and the total depth:

Width and Length at surface	=	68 m
Surface Area	=	4,624 m^2

8.2.1.3 Nitrogen Removal

Estimates for the winter nitrogen removal can be made using the equations presented in Section 8.1.1.4. Estimates for the winter effluent nitrogen concentration (assumed to be mostly ammonia) are 22.4 and 27 mg/L, averaging at 24.7 mg/L.

8.2.1.4 Upgrading Technologies

Storage Ponds

Given that the same climatic conditions and flows apply to the aerated lagoon as the facultative lagoon, the same storage pond configuration can be used. See Section 8.1.1.5.

Wetlands

As was shown in section 8.1.1.5, wetlands are not a viable alternative for year-round use in this climate. Winter storage followed by seasonal wetland use is the only option for wetlands use.

8.2.2 Kamloops

8.2.2.1 Anaerobic Primary Treatment Cells

The same design procedure as that outlined in Section 8.1.1.1 applies for anaerobic primary treatment cells.

8.2.2.2 Sizing

A minimum of two cells in an aerated lagoon system is recommended, three to four being preferred. In this example, three cells will be used. Eq. 4-8 is used to determine a k_{pm} value.

$$k_{pm} = 0.276 * (1.036)^{4-20} = 0.15$$

The detention time can then be calculated using Eq. 4-7

$$t = \frac{3}{0.15} [(\frac{135}{15})^{1/3} - 1] = 21.6 \ d$$

However, the minimum detention time for cold climates is 30 days. This is then the determining factor in sizing the aerated lagoon. .The procedure for sizing the aerated lagoon is then essentially the same as that used in the Prince George example.

width and length at base	=	36 m
base area	=	1,296 m^2

The sludge depth is again 0.58 m. The estimation for ice cover in Section 8.1.2.2 of 0.51 m is the same for aerated lagoons. For an area with ice cover, a 0.3 m freeboard is recommended.

The total depth is then:

total depth = 3.7 + 0.58 + 0.51 + 0.3 = 5.09 m

The surface dimensions can then be calculated using the same L:W ratio (1:1), berm slope (3:1), and the total depth:

Width and Length at surface	=	67 m
Surface Area	=	4,490 m^2

8.2.2.3 Nitrogen Removal

The winter effluent nitrogen concentration estimated using techniques previously described are 22.0 and 26.2 mg/L respectively, with an average of 24.1 mg/L.

8.2.2.4 Upgrading Technologies

Storage Ponds

The same storage pond configuration discussed is Section 8.1.2.5 would be applied.

Wetlands

As shown previously (Section 8.1.2.5), wetlands are not a year-round workable option for this climatic region. A combination of storage and seasonal wetland use would be possible.

8.2.3 Abbotsford

8.2.3.1 Anaerobic Primary Treatment Cells

The same design procedure as that outlined in Section 8.1.1.1 applies here for anaerobic primary treatment cells.

8.2.3.2 Sizing

A minimum of two cells in an aerated lagoon system are recommended, three to four being preferred. In this example, three cells will be used. Eq. 4-8 is used to determine a k_{pm} value.

$$k_{pm} = 0.276 \cdot (1.036)^{5-20} = 0.16$$

The detention time can then be calculated using Eq. 4-7.

$$t = \frac{3}{0.16}\left[\left(\frac{135}{15}\right)^{1/3} - 1\right] = 20.3\ d$$

However, the minimum detention time for temperate climates is 25 days. This is then the determining factor in sizing the aerated lagoon.

Calculating the total volume to be

$$820\ \frac{m^3}{d} \cdot 25\ d = 20{,}500\ m^3$$

Using a liquid depth of 3.7 m, a length to width ratio of 1:1, and a berm slope of 3:1, the lagoon cells are sized, using the equation presented in Section 8.1.1.1 as:

Width and length at base	=	32 m
Base area	=	1,024 m^2
Width and length at water surface	=	54 m
Water surface area	=	2,916 m^2

Using Eq.4-9 to check the thermodynamic balance, a wastewater temperature of 6.0°C results. This temperature is close enough to the estimated value of 5.0°C not to require any further iterations.

The full depth of each aerated lagoon must also take into consideration the sludge depth, ice cover and free board allowances. The volume of sludge generated is the same as for facultative lagoons, ie. 8,979 m^3 over a 20 year period. Because of the much smaller dimensions of aerated lagoons, assume again that the sludge is removed every 5 years. In this case the sludge depth in each cell is:

$$\frac{8979\ m^3}{1{,}024\ m^2 \cdot 3\ cells \cdot 4\ removals\ over\ 20\ years} = 0.7\ m$$

Due to economic constraints, it may be unrealistic to accommodate this depth of sludge. More frequent removals may be considered.

For an area without ice cover, a 0.6 m freeboard is recommended.

Assuming sludge removal every five years, the total depth is then:

total depth = 3.7 + 0.7 + 0.6 = 5.0 m

The surface dimensions can then be calculated using the same L:W ratio (1:1), berm slope (3:1), and the total depth:

Width and Length at surface	=	66 m
Surface Area	=	4,356 m^2

8.2.3.3 Nitrogen Removal

The winter effluent nitrogen concentration estimated using techniques previously described are 22.2 and 27.5 mg/L respectively, with an average of 24.9 mg/L.

8.2.3.4 Upgrading Technologies

Storage Ponds

The same storage pond configuration discussed is Section 8.1.3.6 would be applied.

Wetlands

A similar design to that applied for wetlands of facultative lagoon effluent would apply. A wetland water temperature of 4°C is assumed. The same area for BOD_5 removal would be required. Because of smaller nitrogen removal estimates, a larger area would be allowed for aerated lagoon effluent nitrification. Following calculations for a two celled operating wetland system and flow depth of 0.3 m, as previously described (Section 8.1.3.6), the new surface area per cell required would be

Total surface area	=	30,343 m^2
Length	=	304 m
Width	=	100 m

Because this results in a calculated wastewater temperature of 3.7°C, no further iterations are required.

8.3 SF WETLAND DESIGN EXAMPLE

Assume the following influent characteristics:

BOD_5	=	100 mg/L
TN	=	20 mg/L
Q	=	300 m^3/d
Temperature of influent	=	5°C

Climate: temperate (eg. Abbotsford). Given the climate in Abbotsford, ice formation in the wetland is not a concern.

First the area required to reduce the BOD_5 from 100 to 15 mg/L is calculated. Assuming a wetland water temperature of 4°C, the K_T value is calculated as 0.435 using Eq.5-2. The wetland area required is calculated using Eq.5-5 with porosity (n) = 0.4 and water depth (y) = 0.6.

$$A_s = \frac{(300)\ \ln(\frac{100}{15})}{(0.435)(0.6)(0.4)} = 5{,}452\ \ m^2$$

The area required for nitrogen removal is calculated next. Here, a wetland to reduce nitrogen concentrations to 8 mg/L is designed. Solving Eq. 5-10, using rz=0.5 (i.e. 50% of the SF bed depth occupied by the root zone), K_{NH} is found equal to 0.083 at 20°C. Using this value and Eq. 5-12, K_4=0.0392.

Knowing the reaction constant, the area required for nitrogen removal is calculated using Eq. 5-5.

$$A_s = \frac{(300)\ \ln(\frac{20}{8})}{(0.0392)(0.6)(0.4)} = 29{,}218\ \ m^2$$

The wetland area required is then 29,218 m^2. Following the recommendations of Section 5.7.1, a wetland system with two parallel cells is selected. Each cell size is then 14,610 m^2.

A thermodynamic model is then used to check the water temperature assumption which led to this sizing. The available energy in the water flowing through the SF wetland is defined in Eq. 5-14.

Assigning the following values to variables:

c_p = 4215 J/kg/°C

δ = 1000 kg/m^3

A_s = 14,610 m^2

y = 0.6 m

n = 0.4

q_G = 25.66 * 10^{10}.

Eq. 5-15 defines the heat losses from the entire SF wetland. One of the variables used in this equation is the heat transfer coefficient. This is determined by the layers of materials in the wetland. Assuming the wetland consists of 0.6 m medium gravel and water (40% water, 60% gravel), covered with by 0.15 m fine pea gravel, topped by 0.05 m leaf litter, and using the thermal conductivity values given in Table 5.5.4.1:

$$U = \frac{1}{\frac{0.24}{0.58} + \frac{0.36}{2.0} + \frac{0.15}{1.5} + \frac{0.05}{0.05}} = 0.590 \ \frac{W}{m^2 \ °C}$$

Using the hydraulic detention time definition in Eq.5-3, a value of 23.5 days is calculated. An influent temperature of 5°C is estimated and the average winter air temperature of 3.7°C derived from the Canadian Climate Normals.

Eq. 5-15 then defines the energy lost via conduction to the atmosphere as:

$$q_L = (5 - 3.7 \ °C) \ (0.590 \ \frac{W}{m^2 \ °C}) \ (86{,}400 \ \frac{s}{d}) \ (14{,}610 \ m^2) \ (23.5 \ d) = 22.75 \cdot 10^{10} \ J$$

Eq.5-18 then defines the temperature change in the wetland as:

$$T_c = \frac{q_L}{q_G} = \frac{22.75 \cdot 10^{10}}{25.66 \cdot 10^{10}} = 0.89 \ °C$$

The temperature of the effluent according to Eq.5-19 is then:

$$T_e = 5 - 0.89 \ °C = 4.11 \ °C$$

The average water temperature in the wetland according to Eq. 5-21 is then:

$$T_w = \frac{5° + 4.11\,°C}{2} = 4.5\,°C$$

This value agrees with the assumed value of 4°C to within 1°C, so no further iterations are required.

The dimensions of the wetland are then determined using Eq.5-34 which defines the minimum width. First a k_{sT} value is determined using Eq. 5-36.

$$\frac{k_{sT}}{k_{s20}} = \frac{\mu_{20}}{\mu_T} = \frac{1.002}{1.518} = \frac{k_{sT}}{30{,}000}$$

Solving, K_{s4} = 19,800. As recommended, a value less than a third of this is used. For design purposes, K_{s4} = 6,000. Setting m = 0.1 (increment of depth serving as head differential), Eq. 5-35 defines minimum width as:

$$W = \frac{1}{0.6}\left[\frac{(150)(14{,}610)}{(0.1)(6000)}\right]^{1/2} = 100\ m$$

The length is determined from the minimum width and area to be 146 m.

As a check, calculate the Reynold's number to ensure that the flow is laminar. Setting the diameter of the void spaces equal to the media size (0.032 m), and setting v=Q/(Wy), Eq.5-35 calculates the Reynold's number to be:

$$N_R = \frac{(150\ \frac{m^3}{d})(0.032\ m)}{(100\ m)(0.6\ m)(1.519 \cdot 10^{-6}\ \frac{m^2}{s})} \cdot \frac{1}{(3600)(24)\ \frac{s}{d}} = 0.36$$

This number is less than 10 (maximum), so the assumption that the flow is laminar is valid.

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