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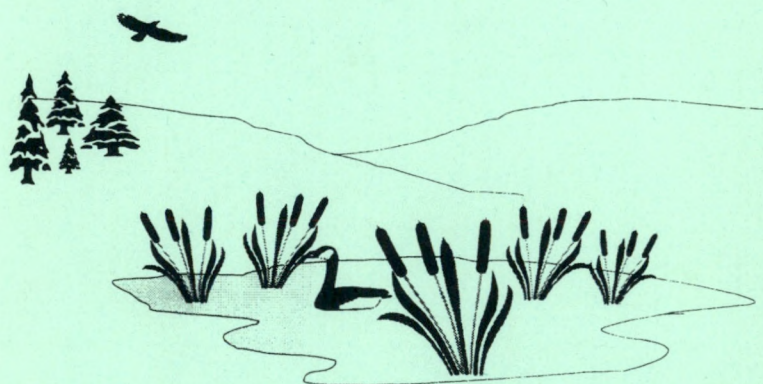
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WETLANDS WASTEWATER TREATMENT FOR THE CANADIAN PRAIRIES

**A Technical Manual on Natural and Wetlands Ecotechnologies
for Improving Quality of Runoff, Municipal Sewage and
Industrial Wastewater in Western Canada**



**A Masters Degree Project Report Prepared by:
Kim Eastlick, P. Eng.**

**Faculty of Environmental Design
The University of Calgary
Calgary, Alberta**

August, 1994

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THE UNIVERSITY OF CALGARY

**Wetlands Wastewater Treatment for the
Canadian Prairies**

by

BRIAN KIM EASTLICK, P.Eng.

A Masters Degree Project
submitted to the Faculty of Environmental Design
in partial fulfillment of the requirements for the degree of
Master of Environmental Design
(Environmental Science)

CALGARY, ALBERTA

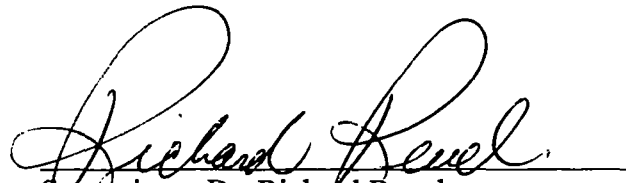
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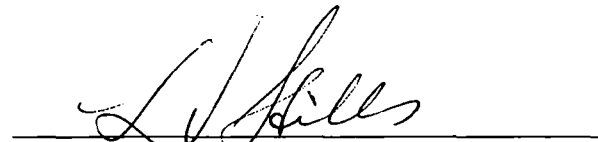
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THE UNIVERSITY OF CALGARY
FACULTY OF ENVIRONMENTAL DESIGN

The undersigned certify that they have read and recommended to the Faculty of Environmental Design for acceptance, a Master's Degree Project entitled "Wetlands Wastewater Treatment for the Canadian Prairies" submitted by Brian Kim Eastlick in partial fulfillment of the requirements for the degree of Masters of Environmental Design.


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ABSTRACT

Wetlands Wastewater Treatment for the Canadian Prairies

By: Brian Kim Eastlick

August 1994

Prepared in partial fulfillment of the requirements of the M.E.Des. degree in
the Faculty of Environmental Design, The University of Calgary.

Supervisor: Dr. R.D. Revel

Wetlands and other natural wastewater treatment processes present a low cost opportunity to improve the quality of contaminated water. A large and diverse body of literature must be assessed, however, before designers and decision makers can consider the technology for solving specific problems. This study consolidates information about the technology and provides a compilation of natural and wetlands system design procedures.

Design factors and pollutant removal correlations were summarized in a way that permits comparison and integration of design approaches for a range of applications including sewage treatment, acid mine drainage mitigation, agricultural wastewater treatment and runoff management. Information on issues and problems was summarized and indicates that wetlands systems should perform effectively in the Canadian prairies. Proper design and biological controls can mitigate mosquito and other nuisance concerns.

It is recommended a program of wetlands technology development be initiated in Alberta with a focus on urban and agricultural runoff.

KEYWORDS: acid mine drainage, agricultural wastewater, domestic wastewater, natural water treatment, non-point source pollution, passive water treatment, sewage, wastewater treatment, wetlands, water quality.

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The literature review that formed the basis of this document was supported by a grant from the Canada Trust Friends of the Environment Foundation. The grant project involved the assessment of wetlands water treatment technology and development of a research and development prospectus to further the use of the ecotechnology in Western Canada. This assistance enabled acquisition of the large body of literature necessary for the design procedures synthesis presented in my MDP.

I thank the Canada Mortgage and Housing Corporation for the scholarship support during the 1993 - 1994 academic year.

I thank my father, Kenneth R. Eastlick, for a lifetime of education and for his example, help, understanding and wisdom as I set out on this new mid-life path.

Lastly and most importantly, I acknowledge the encouragement, support and love of my wife Deborah and son, Quinn. With their help I was able to let go of an established career, refresh my knowledge, gain new understanding and create a spectrum of worthy opportunities for the future.

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LIST OF ABBREVIATIONS

ALD	Anoxic limestone drain. A method of increasing pH of acid water.
AMD	Acid mine drainage
AWRC	Alberta Water Resources Commission (Edmonton, Alberta)
BMP, BMPs	Best management practice(s). The term is used in the text to describe practices, processes and facilities for runoff water quality enhancement.
BOD	Biochemical oxygen demand. A measurement of the oxygen used by microorganisms to biochemically oxidize water contaminants.
BOD ₅	5-day biochemical oxygen demand. A type of BOD evaluation based on sample incubation for five days at 20°C.
BRWQTF	Bow River Water Quality Task Force (Calgary, Alberta).
BTI	<i>Bacillus thuringiensis var. israeliensis</i> . Bacteria used for biological control of mosquitoes.
CCME	Canadian Council of Ministers of the Environment.
CFU	Colony forming units. A measure of the number of organisms such as fecal coliforms in a water sample.
COD	Chemical oxygen demand. A measure of the oxygen required to oxidize water contaminants determined from sample consumption of a strong oxidizing chemical (e.g., potassium dichromate in an acidic medium). COD is typically higher than BOD as more compounds are susceptible to chemical oxidation than to biological processes.
DO	Dissolved oxygen.

ED	Extended detention. ED refers to temporary retention facilities where runoff inflows are collected and released at controlled rates to downstream systems.
ERWQFT	Elbow River Water Quality Task Force (Calgary, Alberta).
FWS	Free water surface. FWS wetlands have standing or flowing water above the soil surface.
hi marsh	Term used to describe shallow marsh (0 - 15 cm deep) sections of stormwater wetlands.
HLR	Hydraulic loading rate. Defined as the inflow rate divided by the area of the system, typically expressed as cm/day or m/day.
HRT	Hydraulic residence time. Defined as the volume of the system divided by the average or influent flow rate, typically expressed in days.
L:W	Length to width or aspect ratio.
LDP	Limiting design parameter. In systems where several factors must be accounted for, the LDP is that factor that defines minimum design criteria (e.g., where separate correlations for BOD and nutrient removal are used to determine system size, the LDP is the relationship that specifies the largest area).
lo marsh	Term used to describe deeper marsh (14 - 45 cm deep) sections of stormwater wetlands.
NPS	Non-point source. NPS water pollution results from stormwater flushing of contaminants from the landscape. Point sources, by comparison are identifiable as single outlets from specific facilities.
OF	Overland flow. Wastewater treatment process where water is purified by shallow flow on vegetated slopes.

OMAF	Ontario Ministry of Agriculture and Food.
OMOE	Ontario Ministry of the Environment
PTS	Persistent toxic substances. Toxic materials that decompose very slowly or persist under normal conditions in the environment.
RI	Rapid infiltration. Wastewater treatment process where water is purified by infiltration into pervious basins.
RS-P	Reductant soluble phosphorus. Phosphorus compounds typically formed with iron under oxidizing (aerobic) conditions that can be reduced and become soluble under anoxic conditions.
RZM	Root zone method. Constructed wetlands wastewater treatment process where water is treated by subsurface flow through plant rooting media.
SFS	Subsurface flow system. Another common term for describing constructed wetlands wastewater treatment processes where water is treated by subsurface flow through plant rooting media.
SR	Slow rate. Wastewater treatment process involving infiltration into vegetation covered soils. Also referred to as wastewater irrigation.
SS	Suspended solids. Suspended particulate matter in water that does not settle but can be filtered out.
SWIS	Subsurface wastewater infiltration system. Wastewater treatment process where water is distributed in soil covered trenches (e.g., typical septic drain field).
$t_{1/2}$	Half-life. Time required for 50% of a substance or population (e.g., coliforms) to be removed or decompose.
TDS	Total dissolved solids. Dissolved constituents of water that remain upon evaporation.

TEH	Total extractable hydrocarbons. A measure of hydrocarbon contamination of water.
TOC	Total organic carbon. A measure of organic material obtained by oxidizing a sample and measuring resulting carbon dioxide.
TSS	Total suspended solids. Sum of volatile (will decompose if heated) and non-volatile suspended particulate matter that cannot be removed by primary settling.
US EPA	United States Environmental Protection Agency.
US SCS	United States Soil Conservation Service (Department of Agriculture).
VS _B	Vegetated submerged bed. The term is used in the text to describe constructed wetlands wastewater treatment processes where water is treated by subsurface flow through plant rooting media.
V _t	Treatment volume. Used to describe the permanent waterbody size of stormwater treatment facilities.
Washington DOE	State of Washington Department of Ecology.
WID	Western Irrigation District (Calgary, Alberta).
WPCF	Water Pollution Control Federation (Alexandria, Virginia).
WWT	Wetlands wastewater treatment. Systems that use natural process occurring in wetlands to treat contaminated water.

1. INTRODUCTION

1.1 Problem Statement

The desire to reduce the impact of human activity on the environment is often restrained by lack of economical means of intervention. Domestic sewage, agricultural wastewater and contaminated runoff contribute to the degradation of surface water and groundwater resources, however, the elimination of these water pollutants from rivers and lakes if it results in the closing of businesses or the saddling of municipalities with excessive infrastructure cost is not acceptable. Conventional wastewater treatment technology, geared to economies of scale, can provide appropriate solutions to point sources of water pollution from major cities and large industrial complexes. These same technologies can be completely inappropriate for non-point source water pollution and for wastewater from small entities.

Appropriate treatment technology for stormwater runoff, individual residence domestic sewage and wastewater from small rural businesses, institutions and communities must meet several challenges. Firstly, the capital cost must not be a barrier to use of the technology. Secondly, the consumption of energy and chemicals should be minimal. Thirdly, the system should be constructable and maintainable with simple equipment and readily accessible materials. Forthly, the processes and equipment should be safe and pose minimal risks to users. Finally, the system should be passive and require minimal operator intervention. While all five constraints are important, it is the latter that is believed to be critical to small wastewater and non-point source pollution control systems. Processes that require specialized skills and significant person-power commitments for operation and maintenance will be a costly burden to small users and performance can be expected to be impaired due to inadequate attention.

Natural wastewater treatment systems harness the physical, chemical and biological processes found in soils, ponds and marshes to purify polluted water. Such systems can be designed to be passive, relying on gravity flow, solar energy, simple

physical processes and the action of resident biota to remove water pollutants. Equipment consists of ponds, shallow basins and rudimentary level control devices. Building materials can be as simple as earth for berms and wood for outlet structures (e.g., stop-log weir). Constructed wetlands in particular, offer the opportunity for low cost, environment enhancing treatment of contaminated water.

It is the author's opinion that the lack of a single reference explaining the basic science behind wetlands and other natural treatment systems is a barrier to wider use of the technologies. Many journal articles, conference proceedings, books and government publications are available on the topic. Gaining an understanding of the basic contaminant removal and transformation processes in natural water treatment systems and obtaining information on design procedures for specific applications requires review of considerable literature. Much of the information, is organized by application with little cross-referencing and, as a result, a great deal of reading is required to gain a broad comprehension of natural wastewater treatment technology. The professional time required to become knowledgeable on the topic can be onerous for organizations that could use the technology and for the professional consultants that advise them.

1.2 Study Objectives

This Masters Degree Project (MDP) study firstly is intended to provide a description of natural processes for the removal and transformation of water pollutants with a focus on constructed wetlands. Secondly, it is intended to provide basic design correlations and procedures for the evaluation of natural water treatment technologies. The study is thirdly intended to enable development of integrated systems of natural technologies by placing design information on a variety of processes and related applications in one reference. Finally, the MDP is intended to serve as an initial assessment of the applicability of the technology for the western Canadian prairies.

It is the opinion of the author that wetlands wastewater treatment (WWT) and related natural processes represent low cost "appropriate" technologies for management of sewage, agricultural, industrial and runoff wastewater quality in northern prairie climates. It is postulated that sufficient research and development has been completed to permit the design and installation of pilot full scale constructed wetlands systems in the Canadian prairies.

The objective of this report is to provide a concise reference from which WWT designs and pilots for the northern prairies can be developed. The report will:

- discuss the basic processes by which wetlands remove contaminants from wastewaters
- summarize the types of natural wastewater treatment technologies
- explain design procedures
- examine potential constructed wetlands applications
- illustrate procedures through a conceptual design example of an integrated domestic waste and storm water systems
- recommend research and development needed to gain WWT experience in, and to adapt urban stormwater quality improvement design procedures to, Western Canada

Although a range of applications are discussed, the report will emphasize the application of WWT for management of non-point source pollution from urban runoff. It is the author's opinion that effective action to reduce nonpoint source pollution is hampered by the lack of proven and cost effective treatment. Natural processes embodied in wetlands may provide the appropriate technology that will enable the urban runoff water quality issue to be addressed.

1.3 Methodology

This report has resulted from a review of the literature on natural wastewater treatment processes with a focus on constructed wetlands. Literature used in the

development of the report included journal articles, conference proceedings, technical books and government publications.

Journal articles and conference proceedings were used as a source of information on specific projects and as a resource to identify publications providing widely accepted design procedures. Computer searches were used to identify journal articles and relevant conference proceedings. Contacts within agencies such as the Tennessee Valley Authority, the US EPA Small Flows Clearinghouse, Washington State Department of Ecology, British Columbia Ministry of the Environment and the Ontario Ministries of Environment and Energy and Agriculture and Food provided guidance to key journal literature. It was found that journal articles and conference proceedings provided examples and information on experimental and field results. The articles, however, supplied limited design procedure information.

Engineering manuals, regulatory guidelines and government publications were used as the principle source for design procedures discussed in the report. The information in these sources resulted from synthesis of technical and experimental information by experts and consequently were judged as the most reliable source of design guidelines. Several documents reviewed were developed as regulatory guidelines for various North American jurisdictions, thus the design procedures contained in them are expected to be reasonably sound.

The report is organized to provide an orderly progression through the technology for the reader. Section 2 introduces the physical, chemical and biological functions of wetlands that explain pollutant removal. The basics of wetlands hydrology are discussed in Section 3. Design procedures and correlations for a variety of interrelated natural water treatment processes are provided in Section 4. Section 5 provides descriptions of how the processes described in Section 4 can be applied to sewage, agricultural wastes, industrial effluents and stormwater treatment problems. While vegetation selection and propagation are situation specific, Section 6 provides descriptions of plants that are expected to be suitable for Western Canada and outlines general approaches to wetland creation. A conceptual design example combining both stormwater management and

domestic sewage treatment is reviewed in Section 7. Wetlands treatment requires additional research and development to resolve several issues, particularly those associated with cold climate operation and effectiveness. Section 8 reviews these issues and discusses research needs.

1.4 Wetlands Wastewater Treatment Background

1.4.1 Overview

Wetland ecosystems are natural nutrient recyclers and water purifiers and can be adapted or constructed to treat water contaminated by human activities (Reed, Middlebrooks and Crites, 1988, Reed, 1991). Such ecosystem "processes" have the advantage of being less expensive to construct and operate than conventional waste process systems. Natural wastewater systems have significant advantage for smaller volumes where the unit costs of conventional processes are prohibitive.

Natural systems can serve multiple purposes, even becoming an amenity asset while providing needed water quality upgrading. Wetland process systems can provide critical wildlife habitat and can be integrated into urban parkland (Palmer and Hunt, 1989) providing ecological, recreational and aesthetic amenity (Ellis, 1988). Many such water treatment systems combine the intrinsic value of wildlife habitat with important economic value to society, justifying the existence of such sites to the naturalist and economist alike.

Wetlands wastewater treatment is a form of ecological engineering which seeks, among other things, to apply the self-designing feature of ecosystems to lower energy costs of waste control, to apply the recycling paths of ecosystems to reduce pollution effects and to take advantage of chemical and biological diversity of ecosystems to "buffer" changes in processing conditions or components (Mitsch and Jorgensen, 1989). WWT technologies have been applied to municipal sewage treatment (Crites, Gunther, Kruzic, Pelz and Tchobanoglous, 1988), acid mine drainage (Hedin and Nairn, 1992), treatment of agricultural wastewaters (Hammer, Pullin, McCaskey, Eason and Payne,

1991), upgrading of petroleum industry effluents (Bishay, Gulley, Hamilton and Nix, 1993, Litchfield and Schatz, 1989), landfill leachate (Trautmann, Martin, Porter and Hawk, 1989) and stormwater management (Livingston, 1988, Olson, 1991). WWT may provide solutions to many western Canadian sewage treatment and stormwater or runoff quality problems where more conventional approaches either are prohibitively expensive or simply export problems downstream.

WWT systems are not yet widely used in Western Canada, despite research on treatment of municipal sewage in Listowel, Ontario (Herskowitz, 1986) and Humbolt, Saskatchewan (Lakshman, 1983), mining wastes in Ontario (Kalin, 1989) and oil sands wastes in Alberta (Bishay et al., 1993). Lack of familiarity with the technology and limited experience with design, construction and operation of the systems relegates WWT to the status of a risky, unproved process for water quality management, particularly for urban stormwater (MacKenzie, 1992). It is believed that development of an inclusive review of natural wastewater treatment and consolidation of design procedures is a first step in enabling greater use of the ecotechnologies.

Of the numerous applications for wetlands wastewater treatment it is believed three uses in particular will be important in the Prairies. These key applications include urban stormwater quality improvement, small sewage treatment systems and agricultural runoff and wastewater treatment. This observation stems from conclusions of recent studies of water quality in the Bow and Elbow Rivers and from inquiries the author has received from individuals, public agencies and private developers. The following three sections summarize needs for affordable water treatment technology to reduce pollution from urban stormwater, small sewage systems and agriculture operations.

1.4.2 Urban Stormwater

Passive natural systems offer the opportunity to address water quality concerns associated with urban runoff. Calgary is the major contributor of urban runoff to the Bow River thus a discussion of the City's stormwater issues can illustrate problems in general

for Western Canada. Estimates of runoff contaminate loads on the Bow River are compared with sewage effluent data in Table 1.1 (BRWQTF, 1991, Appendix VI).

Table 1.1: Comparison of Bow River Contaminant Sources

		Calgary Sewage	Calgary Area
		<u>Effluent</u>	<u>Urban Runoff</u>
Suspended Solids	(tonnes/yr)	1,50	86,000
Total Phosphorus	(tonnes/yr)	130	33
Total Nitrogen	(tonnes/yr)	2,300	850
Lead	(tonnes/yr)	0.6	0.7
Fecal Coliforms	(CFU/100 ml/yr)	2.10E+12	4.10E+07

Urban runoff is a major source of sediment in the Bow. Sediments and nutrients from runoff are reported as causing problems in the Western Irrigation District (WID) system (BRWQTF, 1991). Due to limited mixing in the Bow River, outflow from Nose Creek can bypass into the diversion for the WID system (Schoenekess, 1981, Stanley Associates, 1986). Plant growth, stimulated by the nutrients, affects the quality and aesthetics of Lake Chestermere and impairs flow as well as water quality in the WID system (BRWQTF, 1991). Aspects of Nose Creek water quality are compared with water quality guidelines and the Bow River upstream of Calgary at Canmore in Table 1.2.

Heavy metals, bacteria, biochemical oxygen demand (BOD) and hydrocarbons (particularly dissolved species) also cause concern (Stanley Associates, 1986, MacKenzie, 1992). Bacteria levels in Nose Creek during storm flushes can exceed guidelines for even agricultural use by a factor of 30 (Stanley Associates, 1986). Lead levels rise to undesirable levels during storm pulses although this problem is expected to decline now that tetraethyl lead has been removed from gasoline. Organic compounds, hydrocarbons and bacteria increase chlorine demand of potable water treatment and create conditions for formation of chlorinated hydrocarbons (potentially carcinogenic) in the treated drinking water (Lowey, 1993, Pierce, 1978).

Table 1.2: Runoff Water Quality Comparison

<u>NOSE CREEK vs BOW RIVER vs WATER QUALITY GUIDELINES</u>								
Contaminant	Nose Creek		Bow River		Guideline	Guideline	Guideline	Guideline
	Mouth @ Station 5 (1)		Upstream of Canmore (2)		Raw Water	Recreational	Agricultural	Freshwater
	Average	Storm	Maximums	Minimums**	for	Water (3)	Use (3)	Aquatic
					Drinking (3)			Life (3)
BOD	4.6	9.4						***
TOC	13.8	16.1	2.84	1.2				
Total P	0.16	0.56	0.017	0.007	<0.01 (4)		0.025 (4)	
Dissolved P	0.09	0.16	0.064	0.002				
Total N	2.3	5.2						
NH3-N	0.19	0.55	0.015	0.005	<10.0 (4)			0.06 - 2.2
NO3-NO2-N	0.98	1.47	0.079	0.034	10 & 1.0		100 (4)	0.06
TSS	54.3	417						<80
TDS	515	536	201	114	500		700 (4)	
Zn	0.02	0.06	0.0069	0.0003	5		1.0 - 5.0	0.03
Pb	0.04	0.16	0.0015	0.0007	0.05		0.2	0.001 - 0.007
Cu	0.006	0.01	0.0008	0.0005	1		0.2 - 1.0	0.002 - 0.004
Cr	0.004	0.004			0.05		0.1	0.002
pH	7.9	7.5	8.7	8.2	6.5 - 8.5	5.0 - 9.0		6.5 - 9.0
2-4D	0.41				0.1		0.1 (4)	0.004
DO	7.7		12.5	9.3				6.0 - 8.0
Total Coliform*	2327	39800	16	2	10	200	1000	
Fecal Coliform*	420	2150	6	2	0	35	100	
Fecal Strep.*	312	4475	16	2				
Notes: Units in mg/l unless other wise noted Mean values used for comparison * count per 100 ml **minimums for P, NH3, Zn, Pb, Cu, Total/Fecal Coliforms & Fecal Streptococci represent minimum detectable levels ***must maintain sufficient dissolved oxygen to support aquatic life (game fish)								
Sources: (1) Nose Creek Nose Creek Basin Storm Drainage Study Phase 3 Report, Stanley Associates, Dec. 1986 (2) Bow River Environment Canada, Water Resources Branch, Water Survey of Canada Minima and maxima of 1991 monthly samples taken 4.5 km. upstream of Canmore (3) Quality Guidelines Report of the Task Force on Water Quality Guidelines of the Canadian Council of Ministers of the Environment (CCME) (4) Additional Guidelines Report of the Bow River Water Quality Task Force, Calgary, 1991. (Report criteria based on Alberta Surface Water Quality Guidelines and CCME)								

The Bow River Water Quality Task Force (1991) set out a vision that the Bow River will be a reliable drinking water source and safe for contact recreation. It is

concluded these objectives should also be expected of the Elbow River, Glenmore Reservoir, Lake Chestermere, WID system, Fish Creek, and at least sections of Nose Creek. The latter three are major urban runoff collection systems and, based on information on Nose Creek, suffer degraded water quality. In their current state they represent lost recreation and aesthetic opportunities (especially Nose Creek).

Sediment, nutrient and bacterial loads need to be reduced to meet contact recreation objectives noted in Table 1.2. While source controls may assist in reducing the impacts of urban runoff, effective and affordable stormwater treatment technology is needed to achieve desired surface water quality objectives. Constructed wetlands represent such an appropriate technology. However, before the ecotechnology can be applied, Western Canadian experience must be gained and design procedures adapted to northern prairie climates.

1.4.3 On-Site and Small Community Sewage Treatment

Domestic sewage treatment for individual residences, rural institutions and small communities may consist of septic fields, controlled release lagoons or direct release into rivers. While the former two processes can provide significant treatment under ideal conditions, the technologies may not provide adequate protection for surface and ground water resources for all sites, especially as population density increases. Such are the concerns raised about development west of Calgary by both the Bow River Water Quality Task Force (BRWQTF, 1991) and the Elbow River Water Quality Task Force (ERWQTF, 1993). These reports respectively recommend regional treatment systems and communal sewage treatment for multi-lot country subdivisions.

Provincial requirements for small municipal wastewater systems are based on lagoon treatment with one year storage of effluent (Alberta Environment, 1988 and 1991). These systems are expected to produce effluents containing less than 25 gm/l BOD₅ and suspended solids but the guidelines do not specify nutrient and other contaminant loadings. Anecdotal evidence suggests that when the lagoons are drained

into small streams water quality and nuisance (odors) problems are incurred by those downstream.

As discussed in Section 4, septic drain fields are an effective means of treating domestic wastewater provided soil conditions are adequate and the water table is not too near the surface. Notably, the area that must be reserved for drainage fields becomes large with increasing residents served and with decreasing soil permeability. Such systems, if poorly designed can contaminate surface and groundwater resources and are a concern in the Bow River Watershed (BRWQTF, 1991).

Conventional processes can be capital intensive, consumptive of energy and chemicals and require significant operator control making them undesirable alternatives for on-site or small municipal systems. Based on the author's experience with small rural residential developments, lagoons introduce unacceptable nuisance concerns and restrictions on land use. Development of passive technologies to provide alternatives for inadequate treatment or costly processes is clearly needed. Constructed wetlands can provide affordable secondary and tertiary wastewater treatment and relatively standard design procedures have been published by both the US EPA (Reed, 1993) and the Tennessee Valley Authority (Steiner, Watson and Choate, 1993). Notwithstanding use in other areas, constructed wetland sewage systems are not common in Alberta but pose the opportunity to improve the level of treatment for small communities and rural developments.

1.4.4 Water Quality Treatment and Agriculture

Concerns have been raised regarding the impact of wastewaters and runoff from agricultural operations on the water quality of the Bow River (BRWQTF, 1991). Intensive livestock operations can be source of concentrated effluents from barns and contaminated runoff from outdoor pens. Free range livestock not only may contribute wastes directly to surface waters but also can be responsible for degradation of riparian systems that serve to filter runoff. The use of pesticides and the land spreading of animal wastes can pollute runoff and contaminate both surface and ground water systems. Data

on the Bow River system suggests problems are not yet as serious as in other areas of Canada or the US. Additional monitoring, however, of the impact of agricultural operations on the Bow River and, if issues emerge, the creation of management practices to correct problems has been recommended (BRWQTF, 1991).

Clearly, water quality management technologies for individual farms must be low cost with minimal operating and maintenance inputs. Common practices of storage and land application of wastes can be impractical (Toombs, 1994). Problems can result from sufficient land spreading area not within the control of individual operators and high costs (time and equipment) which discourage proper land application.

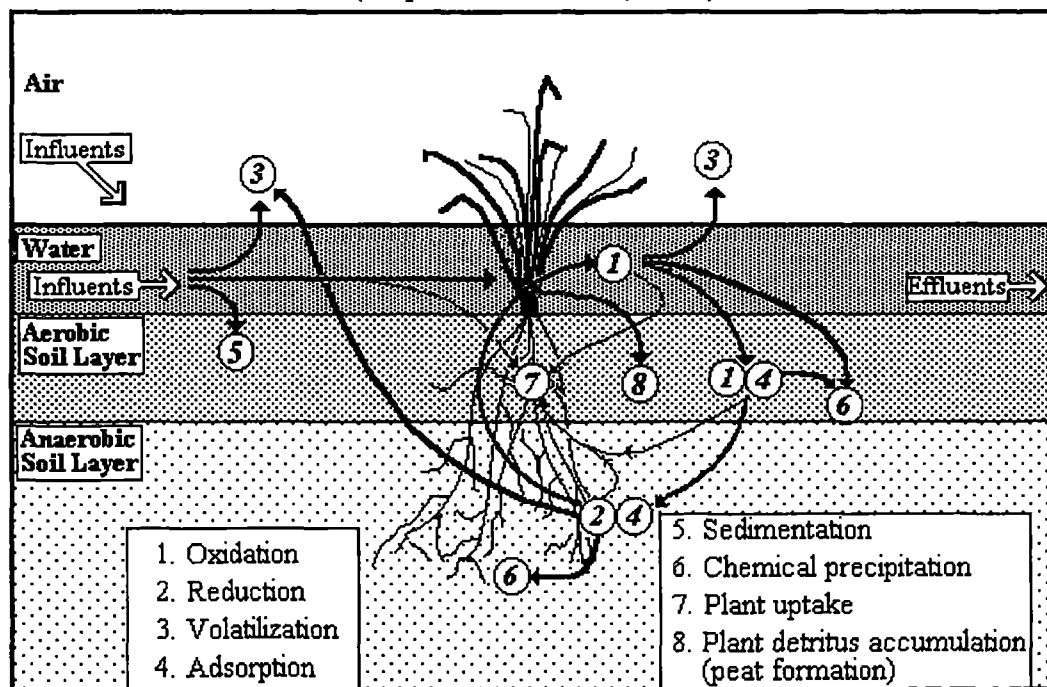
Several natural treatment technologies offer solutions to reduce the impact of agriculture on water quality. Lagoons, commonly used for storage, do provide significant treatment (Hammer, 1991) and restoration of riparian buffer strips, as well as the creation of vegetated buffer strips or wet meadows can further treat runoff from agricultural lands (Toombs, 1994). Constructed wetlands can be integrated into farm water management systems to provide higher levels of treatment and have been successful in treating difficult effluents such as those from swine operations (Hammer, 1991). Several vegetated buffer strip and constructed wetland projects are now in progress in Ontario (Toombs, 1994), however, there appears to be limited experience with the technology in the prairies. It is anticipated that constructed wetlands and related ecotechnologies will become a major contributor to improved agricultural wastewater management practices in western Canada.

2. CONTAMINANT REMOVAL PROCESSES

2.1 Overview

Physical, chemical and biological processes combine in wetlands to remove contaminants from wastewaters and are similar to those found in lagoons and complex sewage treatment facilities. This section will qualitatively discuss the processes by which pollutants can be separated from water and transformed within wetlands. The discussion illustrates the sound scientific basis for the water quality enhancement that occurs in wetlands and provides background for design procedures that will be discussed in Sections 4 and 5. Design correlations for sedimentation are discussed in this section in more detail as they are not dealt with elsewhere in this report.

Figure 2.1: Wetland Processes Schematic
(adapted from Amell, 1993)



Constructed wetland designs usually focus on BOD removal (Reed et al., 1988, Crites et al., 1988), however, an understanding of the many processes that occur in wetlands provides insight on how specific contaminant removal can be optimized and

creates awareness of important design considerations. The processes are schematically represented in Figure 2.1.

Table 2.1: Wetlands Contaminant Removal Processes
(after Watson, Reed, Kadlec, Knight and Whitehouse, 1989)

Process	Contaminants Affected	Explanation
Physical Processes		
Sedimentation	P-settleable solids S-colloidal solids I-BOD, Nitrogen phosphorus, heavy metals, refractory organics, bacteria and viruses	Gravity settling of solids and associated contaminants in pond or marsh settings.
Filtration	S-settleable solids, colloidal solids	Particulates filtered as water passes through soils or substrate, root or vegetation masses or through fish.
Adsorption	S-colloidal solids	Interparticle van der Waals attractive forces.
Volatilization ¹	P-volatile organics, aromatics, lighter halogenated organics	Organic compounds with high vapor pressure volatilize into atmosphere where many are eventually oxidized.
Chemical		
Precipitation	P-phosphorus, heavy metals	Formation of, or coprecipitation with, insoluble compounds.
Adsorption	P-phosphorus, heavy metals S-refractory organics	Adsorption on substrate and organic surfaces.
Decomposition	P-refractory organics	Decomposition of less stable compounds by UV irradiation, oxidation and reduction.
Biological		
Microbial Metabolism	P-colloidal solids, BOD, nitrogen, refractory organics, heavy metals	Removal of colloidal solids and soluble organics by suspended, benthic and plant supported bacteria. Bacteria nitrification/denitrification. Microbially mediated metals oxidation.
Plant Metabolism	S-refractory organics, bacteria and virus	Uptake and metabolism of organics by plants. Root excretions potentially toxic to enteric organisms.
Plant Absorption	S-nitrogen, phosphorus, heavy metals, refractory organics	Significant quantities taken up by plants under proper conditions.
Natural Die-off	P-bacteria and virus	Natural decay of organisms in an unfavorable environment.
P = primary effect, S = secondary effect, I = incidental effect (occurring incidental to removal of other compounds)		
1. Per Reed et al., 1988, Bouwer, 1989 and CCME		

Processes responsible for contaminant removal and their relative importance are summarized in Table 2.1. The key processes are briefly discussed in subsequent parts of this section with the intent of providing a basic understanding of wetland with specific reference to removal mechanisms for suspended solids, BOD, nitrogen, phosphorus, metals and organics.

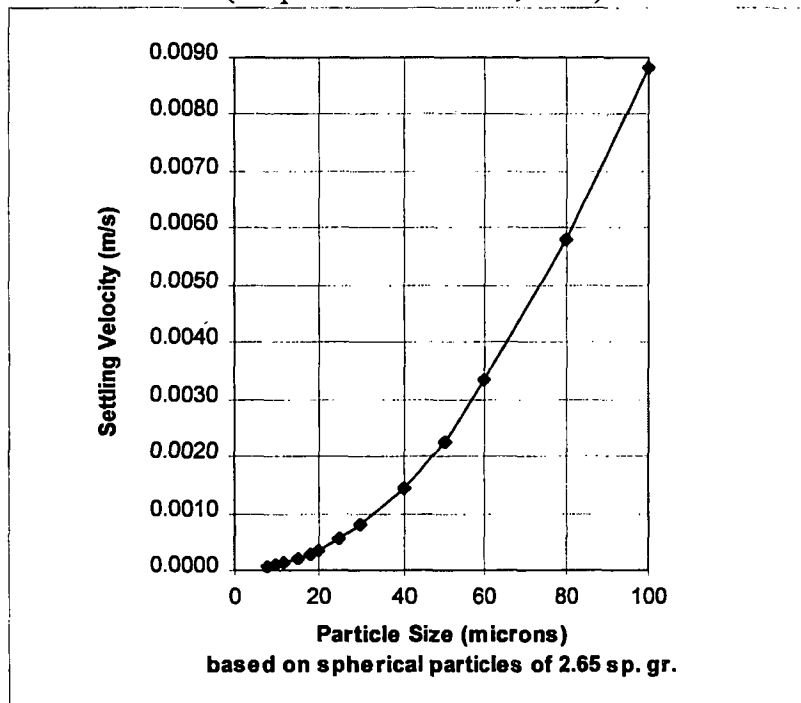
2.2 Sedimentation

The removal of sediments along with sorbed contaminants is one of the most important components of wastewater treatment. This section describes sediment removal processes and provides basic correlations for settling and scour (resuspension of settled solids) evaluations.

Hydraulic properties of wetlands generally result in the removal of settleable solids within the first few metres of the system (Reed et al., 1988). Low velocity coupled with shallow depth and emergent plant protection of the water surface from wind turbulence aid removal of influent suspended solids. Literature on WWT systems does not typically deal with design procedures for sedimentation, however, conventional wastewater treatment engineering manuals do provide insight. Improper consideration of sedimentation and scour conditions probably contributes to inconsistent performance of WWT systems (Strecker, Kersnar, Driscoll and Horner, 1992).

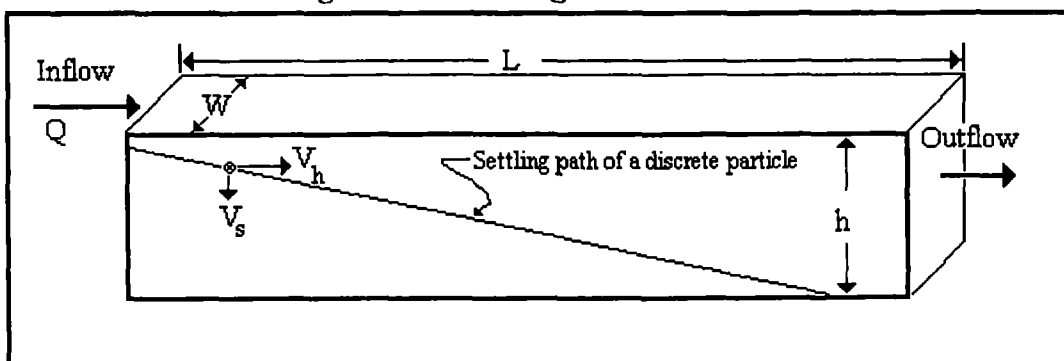
Suspended particles with a density greater than water will attain a downward velocity (settling velocity or V_s) related to particle shape, size and density. Figure 2.2 presents calculated settling velocities for sand (50-100 microns, μm or 10^{-6} m) and silt (3-50 μm).

Figure 2.2: Settling Velocities in Water at 20°C
(adapted from Walesh, 1989)



Design of settling basins must then provide hydraulic residence time (HRT) sufficient for the smallest particles to be removed to travel at V_s from the surface to the bottom of the pool (the depth or distance h). A review prepared for the US EPA (Strecker et al., 1992) found that suspended solids in natural waters ranged between 0.0005 to 100 μm in diameter and that all sediments greater than 60 μm were removed by settling. Runoff sediments from streets were found to include 6% less than 43 μm , 37% between 43 and 246 μm and 57% greater than 246 μm according to studies cited in Strecker et al. (1992).

Figure 2.3: Settling Basin Schematic



From Figure 2.3 various relationships can be drawn:

$$\text{HRT} = (Ah) / Q = h / V_s$$

$$V_h = Q / (A_x)$$

where HRT = hydraulic residence time

L = system length

W = system width

h = system depth

Q = flow rate

V_s = particle settling velocity

V_h = horizontal fluid velocity

A = surface area (A = LW)

A_x = cross-section area ($A_x = hW$)

Settling basins are frequently designed for overflow discharge, thus the required surface area based on the smallest particles to be removed can be determined by (Novotny, Imhoff, Olthof and Krenkel, 1989):

$$A = Q / V_s$$

Horizontal velocities must not create sufficient turbulence to resuspend the settling particles and should be less than 72 m/hr (Novotny et al., 1989). Scour velocity is a term used to define the fluid horizontal velocity sufficient to resuspend or scour particles from the basin bottom (Metcalf and Eddy, 1991). The critical horizontal velocity, above which scour will occur, is defined by the following equation and resulting horizontal velocity limitations are presented in Figure 2.4.

$$V_{\text{hscour}} = [8kgd(s-1) / f]^{1/2}$$

where V_{hscour} = critical horizontal velocity

k = 0.04 for unigranular sand

≥ 0.06 for sticky, interlocking matter

d = particle diameter

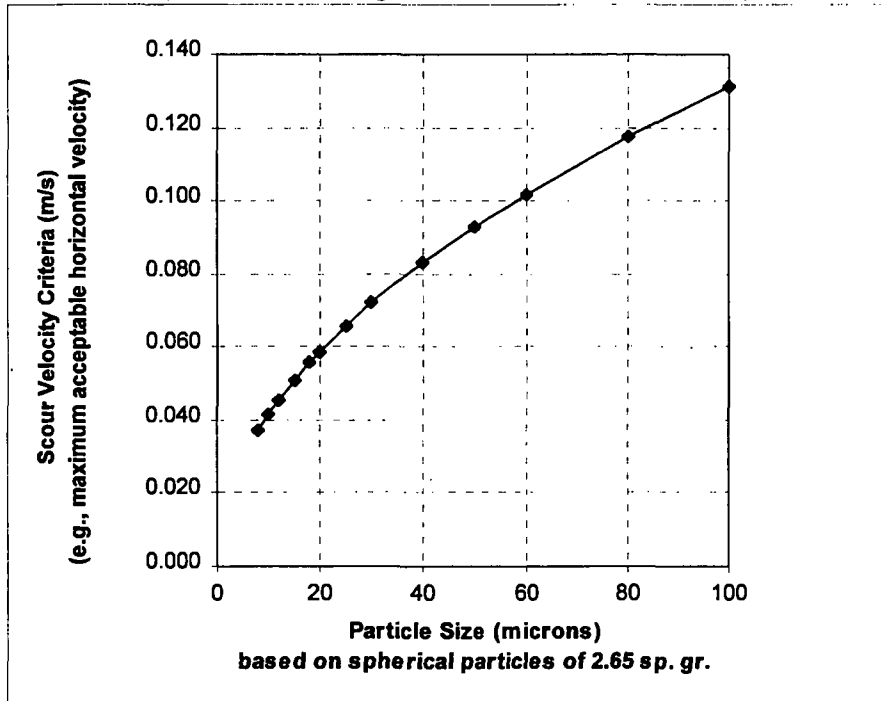
s = specific gravity of particles

f = Darcy-Weisbach friction factor

= 0.02 - 0.03

g = acceleration of gravity (9.81 m/s²)

Figure 2.4: Critical Scour Velocities
(based on settling velocities in Walesh, 1989)



In the case of wetland systems, fluid velocity relationships must account for the space occupied by plants. A porosity factor must be applied to the above relationships and can be defined as (Crites et al., 1988):

$$n = V_v / V = (V - V_p) / v$$

where n = system porosity
 V_v = volume of voids
 V_p = volume of plant material
 V = total system volume ($V = LWh$)

Marsh porosity must account for standing plants and litter, thus, is complex and not easily evaluated. Evaluations in the Listowel pilot (Herskowitz, 1986) allowed 5% (or $n = 0.95$) for the volume on plant material in sewage treatment marshes. Tennessee Valley Authority (TVA) data suggest $n = 0.9$ for *Typha*, $n = 0.86$ for *Scirpus*, $n = 0.98$ for *Phragmites* and $n = 0.95$ for *Juncus* (Steiner and Freeman, 1989). However, literature on free surface flow wetlands suggests $n = 0.75$ should be used in correlations (Reed et al., 1988, Crites et al., 1988). Notably both the latter references base their conclusions in part

on dye tracer tests reported for Listowel. The HRT and horizontal velocity relationships modified to account for system porosity are as follows:

$$\text{HRT} = (nAh) / Q = h / V_s$$

$$V_h = Q / (nA_x)$$

Organic particles are more prone to flocculation than granular materials (Novotny et al., 1989). Such particles tend to adhere to one another when they collide in the water column. As the flocs settle they continue to coagulate into larger masses and the settling velocity increases with system depth (Metcalf and Eddy, 1991). Flocculent settling depends on sufficient system energy to promote particle contact but not enough to destroy flocs or prevent their settling. Wetlands systems tend to provide such enhanced environments due to the combination of complex flow paths through vegetation that enhance particle contact with low velocities that permit settling.

Settling velocities of organic sewage solids ranges from 0.3 - 3.0 m/hr and 1.5 hr detention is often used to size for flocculent settling in conventional treatment processes. Alternatively surface loading rates of 1 m³/hr influent per m² surface area are used (Novotny et al., 1989). Again these design parameters fall well within typical WWT designs.

Literature on WWT does not generally discuss sedimentation design correlations because areas are large and velocities are low relative to conventional settling process vessel conditions. However, the foregoing have value in confirming sizing of sedimentation forebays and assessing the impact of channelized wetland flows.

2.3 Filtration:

Filtration occurs when water flows through dense stands of wetland plants and rooting media. The physical process is important for removing both solids and dissolved pollutants from water. A number of processes separate contaminants from water as it is filtered:

- *Sedimentation*: heavy particles settle within pore matrix of media.

- *Flocculation*: turbulence and impingement on medium causes very small particles to collide and combine into larger bodies that will settle out.
- *Straining*: particles that are too large to pass through the media become trapped in the matrix.
- *Adhesion*: particles collide with and adhere to filter media.
- *Sorption*: particles and dissolved materials attach to surfaces through absorption into biofilms and adsorption onto media surfaces.

Certain types of wetland systems direct all flow subsurface, thus, filtration by the substrate is a major factor. However, dense stands of vegetation provide conditions for most of the filtering processes to work. Even straining occurs on a very macroscopic scale as emergent plants will trap and at least shelter from view influent trash (Schueler, 1992).

Deliberately designed sand or sand-gravel filters can be used to improve suspended solids (SS) removal especially where turbidity from algae growth is a problem in WWT or lagoon effluents. Suggested hydraulic loading rates are 0.37 - 0.56 m³/d per m² of filter cross section area for SS up to 50 mg/l and 0.19 - 0.37 m³/d per m² for higher SS loadings (Reed et al., 1988). Engineering texts such as Ramahlo (1983) provide detailed multimedia filter design procedures. Sand filters can also be used as a secondary treatment system. The US EPA (1980) provides design procedures based on hydraulic loading for buried and open single pass systems, as well as recirculating filters. These systems combine the physical processes of filtration with the action of bacteria and other micro and macro flora and fauna (discussed in the following sections) to remove solids and dissolved organic matter.

2.4 Adsorption and Ion Exchange

Adsorption of contaminants is an important removal process in wetlands systems particularly for phosphorus compounds, refractory organics and metals. Adsorption occurs on wetland soils, plant surfaces and detritus. Ideally adsorption is followed by

further chemical transformation that more permanently sequesters the pollutants and regenerates the adsorption sites.

Adsorption of contaminant molecules or atoms (adsorbate) occurs when they migrate from the bulk solution and become bonded to a surface (adsorbent). Adsorption occurs by electrostatic attractions, physical forces (Van der Waals forces and hydrogen bonding) and chemical reactions (Strecker et al., 1992). Chemical adsorption (chemisorption) is usually not reversible but physical adsorption can be reversed by changing adsorbate concentrations (often with increased temperature). The process is used in conventional wastewater treatment for the removal of metals and organic materials, usually by adsorption on activated carbon (Metcalf and Eddy, 1991). Adsorption is enhanced by residence time, shallow water depths and influent distribution that enhances contact with wetland soils and organic surfaces (Strecker et al., 1992).

Ion exchange occurs when contaminant ions are substituted for others on surface sites, particularly in wetlands soils. The familiar example of ion exchange is the domestic water softener where calcium and magnesium ions (responsible for "hardness") replace sodium ions on a zeolite media. The ions removed by ion exchange may become available for further chemical or biological action on the adsorbent surface.

A given quantity of substrate will possess a finite number of adsorption and ion exchange sites. If chemical or biological modification of the sorbed species does not regenerate the site, the media will become saturated over time and contaminant removal will decrease (Strecker et al., 1992). A peatland WWT system in Houghton Lake, Michigan has been shown to be very effective in treating secondary sewage with high removal levels for metals, nitrogen and phosphorus (Kadlec and Alvord, 1989). The system, however, is expected to have a finite phosphorus removal life span (20 - 30 years) due to saturation of soil adsorption sites (Girts and Knight, 1989).

2.5 Redox Processes

Wetlands support environments for both oxidation (loss of electrons, typically through addition of oxygen or loss of hydrogen) and reduction (gain of electrons, typically through addition of hydrogen or loss of oxygen). The redox reactions, which can be biologically mediated or occur through physical chemistry, provide the pathways for transformation and cycling of materials in wetland systems. Both macro and micro flora and fauna are involved in these biologically mediated transformation processes.

Soil properties and the availability of oxygen in the water column and soils determines the type and direction of the redox reactions. Properties of organic and mineral wetland soils are compared in Table 2.2.

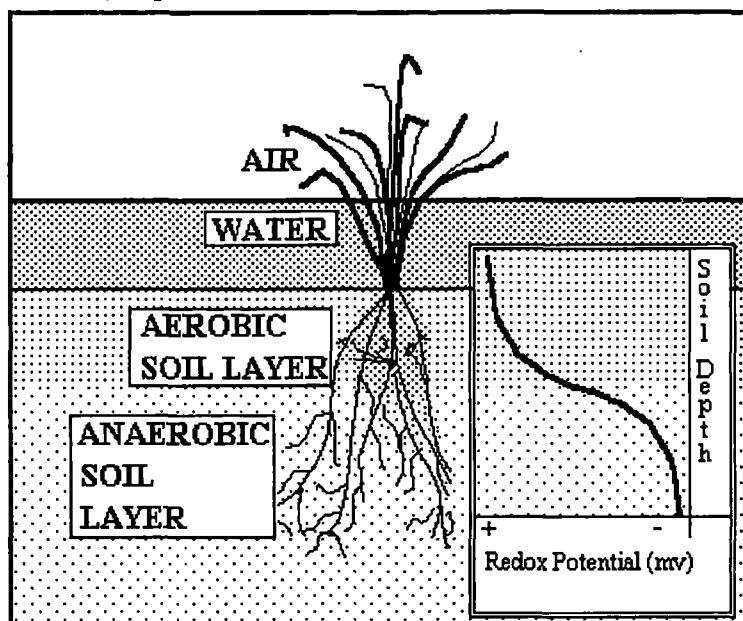
Table 2.2: Comparison of Mineral and Organic Wetland Soils
(adapted from Faulkner and Richardson, 1989 and Mitsch and Gosselink, 1993)

Factor	Mineral Soil	Organic Soil
Organic Content (%)	less than 20 - 35	greater than 20 - 35
Organic Carbon (%)	less than 12 - 20	greater than 12 - 20
pH	usually circumneutral (6.0 - 7.0)	acid (<6.0)
Bulk Density (gm/cu. cm)	1.0 (clay) to 2.1 (gravel)	<0.09 (fibric) to >0.2 (sapric)
Porosity (%)	20 (gravel) to 60 (clay)	<84 (sapric) to >90 (fibric)
Hydraulic Conductivity (m/d)	<0.01 (clay) to 1000 (gravel)	<0.01 (sapric) to >1.3 (fibric)
Water Holding Capacity	low	high (80%)
Nutrient Availability	generally high	often low (nutrients in unavailable forms)
Cation Exchange Capacity	low (dominated by Ca, Mg, K and Na)	high (dominated by H ion)
Typical Wetland	riparian forest	peatland

For the most part shallow WWT systems have vertical transition from aerobic (oxidizing) to anaerobic (reducing) conditions. Such systems are typically layered as shown in Figure 2.5. Diffusion of oxygen across the air-water interface is aided by wind turbulence and water convection currents. The aerobic state of the water column is sustained by photosynthetic oxygen production of algae and submerged plants (provided respiration by plants and animals and chemical oxidation do not exceed diffusion and

photosynthesis). The foregoing usually results in a thin oxidized layer of soil overlying deeper anaerobic sediments (Mitsch and Gosselink, 1993). The vertical variation of redox potential provides the conditions for linked oxidation and reduction pathways that remove and transform wastewater contaminants.

Figure 2.5: Wetland Redox Illustration
(adapted from Mitsch and Gosselink, 1993)



A similar aerobic - anaerobic system can develop around roots of wetland plants. Plants such as cattail have the ability to actively transport oxygen to roots. Radial oxygen loss from root membranes creates an aerobic rhizosphere around the roots in which oxidizing reactions such as nitrification (conversion of ammonia to nitrate) occur. Oxidized compounds can diffuse into surrounding anaerobic soil where reduction processes such as denitrification (conversion of nitrate to elemental nitrogen) proceed (Faulkner and Richardson, 1989). The rhizosphere system is especially important for vegetated submerged bed WWT systems (explained in Section 4).

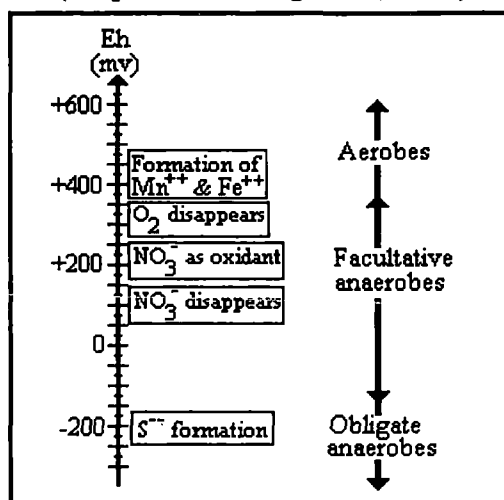
Redox potential (a measure of electron availability) varies between +400 and +700 mv in the presence of free oxygen in solution but drops between +400 and -400 mv in the absence of oxygen in wetland soils (Mitsch and Gosselink, 1993). The implication of redox potential for key elements is shown in Table 2.3.

Table 2.3: Oxidized and Reduced Elements
(adapted from Mitsch and Gosselink, 1993)

Element	Oxidized Form	Reduced Form	Transformation Redox Potential (mv)
Nitrogen	NO_3^- (Nitrate)	N_2O , N_2 , NH_4^+	250
Manganese	Mn^{4+} (Manganic)	Mn^{++} (Manganous)	225
Iron	Fe^{3+} (Ferric)	Fe^{++} (Ferrous)	120
Sulfur	SO_4^{--} (Sulfate)	S^{--} (Sulfide)	-75 to -150
Carbon	CO_2	CH_4	-250 to -350

As soils become reduced, oxygen is consumed followed by the reduction of nitrate and then reduction of manganic ions in an overlapping sequence. Reduction of ferric to ferrous iron, sulfate to sulfide and carbon dioxide to methane sequentially occur only after preceding components are fully reduced (Faulkner and Richardson, 1989). Changing redox conditions are also related to the biological community as illustrated in Figure 2.6.

Figure 2.6: Redox and the Biological Community
(adapted from Jorgensen, 1989)



The redox potential (Eh) is related to soil acidity conditions (pH). Given a general reaction of the form:

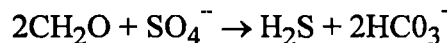


Eh is related to pH by the formula (Faulkner and Richardson, 1989):

$$E_h = E^\circ - 0.059(m/n)\text{pH} + (0.059 / n)\log([\text{Ox}] / [\text{Red}])$$

where n = number of electrons involved in the reaction
 m = number of protons involved in the reaction
 $[\text{Ox}]$ = concentration (activity) of oxidized species
 $[\text{Red}]$ = concentration (activity) of reduced species
 E° = standard potential of redox couple to exchange electrons

Wetlands soils tend to neutral pH (pH 6.7 - 7.2) with flooding of alkaline soils decreasing pH due to carbon dioxide - carbonic acid build up and flooding of acid soils increasing pH due to reduction of ferric iron (Mitsch and Gosselink, 1989). Where acidity is generated by sulfate, sulfate reducing bacteria in anaerobic wetland soils oxidize organic matter as follows:



The hydrogen sulfide further reacts to precipitate metal sulfides and the bicarbonate raises the pH of the water. This feature makes WWT attractive for treatment of acid mine drainage (McIntire, Edenborn and Hammack, 1990)

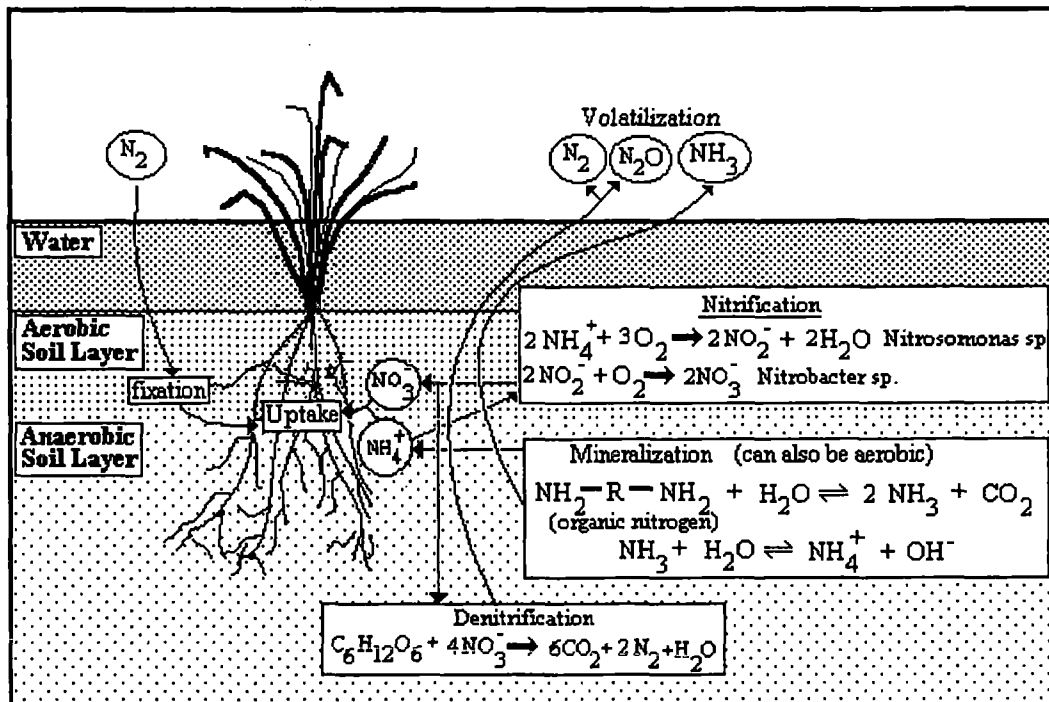
2.6 Nitrogen Cycle

Nutrient forms of nitrogen (nitrate, ammonia and soluble organic nitrogen) are essential to plant growth in aquatic systems but if scarce can limit or control plant productivity (Mitsch and Gosselink, 1993). In systems limited by nutrient nitrogen availability, contributions from runoff, sewage and other wastewaters can lead to explosive plant growth. Such eutrophic conditions are characterized by excessive plant growth and impairment of dissolved oxygen level from diurnal shift of plants from daylight photosynthesis to night respiration or from oxidation of plant detritus

(Vallentyne, 1974). A key objective for WWT design is reduction of nutrient nitrogen in effluents.

The aerobic layer overlying anaerobic wetland soils facilitates several pathways for nitrogen cycling. Pathways exist within wetlands for the fixation of nitrogen to organic forms but of interest in WWT are the pathways for conversion of organic nitrogen, soluble organic nitrogen, ammonia and nitrates to gaseous nitrogen and nitrous oxides which remove the nutrients from the water system. A schematic of the wetlands nitrogen cycle is shown in Figure 2.7.

Figure 2.7: The Wetland Nitrogen Cycle
(adapted from Mitsch and Gosselink, 1993)



Aerobic and anaerobic processes of mineralization convert organic nitrogen to ammonium nitrogen. Ammonia can be absorbed by plants, adsorbed onto sediments, volatilized into the atmosphere or oxidized (nitrification step) by bacteria (*Nitrosomonas* sp.) as it diffuses through the thin oxidized soil layer. The resulting nitrates are more mobile and may be assimilated by plants, reduced to ammonia or denitrified to gaseous nitrogen and nitrous oxide by anaerobes (Mitsch and Gosselink, 1993). Wetland flora can also form organic nitrogen through fixation of atmospheric N_2 and consequently

wetland plants can be sources of ammonia and nitrates when dead vegetation decomposes. Plants, symbiotic bacteria (e.g., *Rhizobium* on the roots of certain plants) and non-symbiotic bacteria (e.g., blue-green algae) can fix N_2 present in surface water, aerobic soils, anaerobic soils, plant rhizosphere and atmosphere (Mitsch and Gosselink, 1993). The processes are cyclical, with wetlands capable of sustaining, reducing or exporting nutrient nitrogen. WWT design seeks to enhance removal pathways for uptake and transformation of nutrient nitrogen into unavailable forms (e.g., N_2).

The process has implications on how constructed systems are designed. Nitrogen entering constructed wetland systems can be in the form of organic compounds and ammonia. Much of the organic nitrogen is converted to ammonia through decomposition and mineralization. Ammonia is toxic to many types of aquatic fauna and removal can be required in certain applications (Reed, 1993). Sufficient oxygen must be supplied to oxidize dissolved organic materials and then to nitrify the ammonia. This is accomplished in wetlands through very shallow marsh or meadow sections. Denitrification, as shown in Figure 2.7, occurs anaerobically with the nitrate acting as the electron receptor for oxidation of organic matter. Wetland processes have been designed with deeper pools to aid denitrification (Hammer, 1991). Adequate carbon sources must be available for the reactions (e.g., dissolved organic matter in wastewater or plant detritus).

2.7 Phosphorus Cycle

In the same sense as discussed for nutrient nitrogen, phosphorus is a major limiting nutrient in freshwater systems. Anthropogenic inputs of phosphorus can rapidly lead to waterbody eutrophication (Vallentyne, 1974). Unlike nitrogen, the oxidation state of phosphorus is not usually altered in wetlands and the phosphorus cycle is predominantly a sedimentary process rather than the gaseous cycle of nitrogen. The majority of the phosphorus species are eventually tied up in sediment - litter (Faulkner and Richardson, 1989). Table 2.4 summarizes principle forms of phosphorus in wetland systems.

Biologically available phosphates (or soluble reactive phosphorus) are related to the level of orthophosphates (e.g., PO_4^{-3} , PO_4^{-2} , H_2PO_4^-). Other forms of phosphorus are usually considered not to be biologically available (Mitsch and Gosselink, 1993) with the availability of phosphorus to plants reduced by:

- aerobic precipitation of phosphates with ferric iron, calcium and aluminum
- phosphate adsorption on clay, peat and ferric and aluminum oxides and hydroxides
- incorporation into biomass, primarily through soil uptake routes.

Table 2.4: Insoluble and Dissolved Types of Phosphorus
(adapted from Mitsch and Gosselink, 1993)

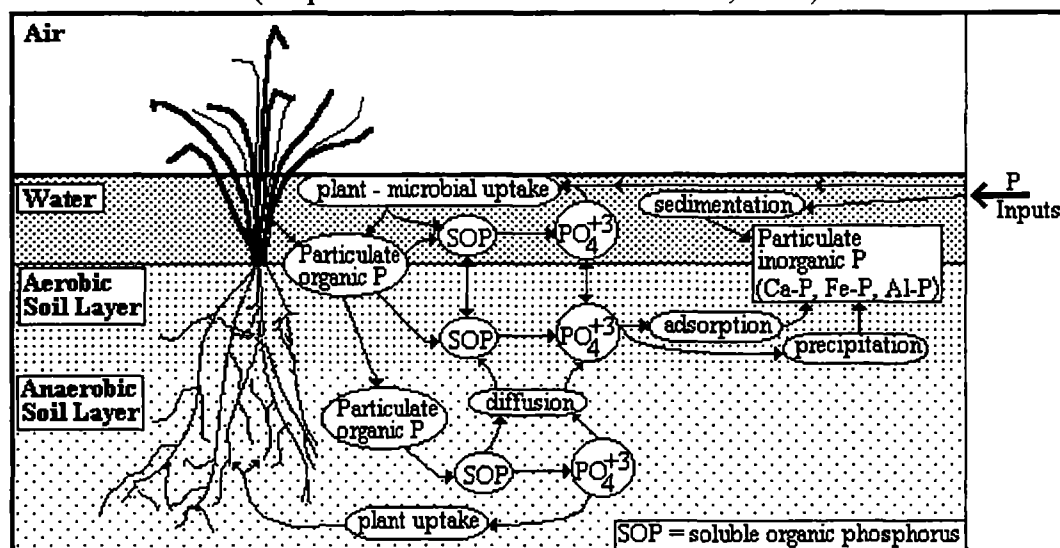
<i>Phosphorus</i>	<i>Soluble Forms</i>	<i>Insoluble Forms</i>
Inorganic	Orthophosphates(PO_4^{-3} , PO_4^{-2} , H_2PO_4^-)	clay-phosphorus complexes
	Ferric phosphate (FeHPO_4^+)	metal hydroxide-phosphates {e.g., vivianite $\text{Fe}_3(\text{PO}_4)_2$, variscite $\text{Al}(\text{OH})_2(\text{PO}_4)_2$ }
	Calcium phosphate ($\text{CaH}_2\text{PO}_4^-$)	minerals {e.g., apatite - $\text{Ca}_{10}(\text{OH})_2(\text{PO}_4)_6$ }
Organic	Dissolved organics (e.g., sugar phosphates, inositol phosphates, phospholipids, phosphoproteins)	insoluble organic phosphorus bound in organic matter

Wetland processes remove phosphorus from water and retain the nutrient within the system, thus are important for treating contaminated water which contains elevated nutrient phosphorus concentrations. As illustrated in Figure 2.8 phosphorus enters wetland systems as a constituent of runoff and wastewater, as well as from the decomposition of wetland vegetation. Phosphates from the decomposition of dissolved organic phosphorus are removed by adsorption on to settling clay particles, through precipitation of metal phosphates and through plant and microbial uptake. Particulate inorganic phosphates adsorbed or complexed with ferric or aluminum hydroxides under

aerobic conditions may be released as sediments become anaerobic (from deeper flooding or from burial in accreting sediments). The available phosphate then may be captured by plant uptake, diffuse to aerobic zones to repeat the adsorption - sedimentation process or be released into the water column and exported

The calcium-phosphate precipitation dominates at pH greater than 7.0 (Faulkner and Richardson, 1989). The adsorption of phosphorus on ferric and aluminum oxides and hydroxides occurs in acid soils. It is suggested that the key prediction of phosphorus sorption capacity is the quantity of oxalate-extractable iron and aluminum in the soil (Mitsch and Gosselink, 1993). In the short term, wetland substrates may provide very high phosphorus removal but as adsorption sites are saturated, effectiveness can decline (Faulkner and Richardson, 1989).

Figure 2.8: The Wetland Phosphorus Cycle
(adapted from Mitsch and Gosselink, 1993):



A form of ferric-phosphorus (reductant soluble phosphorus or RS-P) formed under aerobic conditions, releases phosphorus when flooding results in reduction of ferric iron to the ferrous form. Thus, changing redox potential in wetland sediments may result in phosphorus exports in excess of influent loadings, especially if a wetland is constructed with soils high in RS-P (Faulkner and Richardson, 1989). A major removal pathway for phosphorus in riparian and runoff systems is through sedimentation of

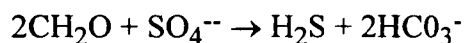
influent clay particles to which phosphorus has adsorbed. Plants then transform the sediment phosphorus into organic forms in living and dead biomass (Mitsch and Gosselink, 1993).

Wetland plants contribute to long-term removal of phosphorus through transformation to organic forms that are consequentially sequestered as plant biomass and as plant detritus in accreting organic soils. The plant uptake mechanisms can be temporary, however, if the plants decompose following senescence and release phosphorus in seasonal exports from the wetland (Faulkner and Richardson, 1989).

2.8 Sulfur Cycle

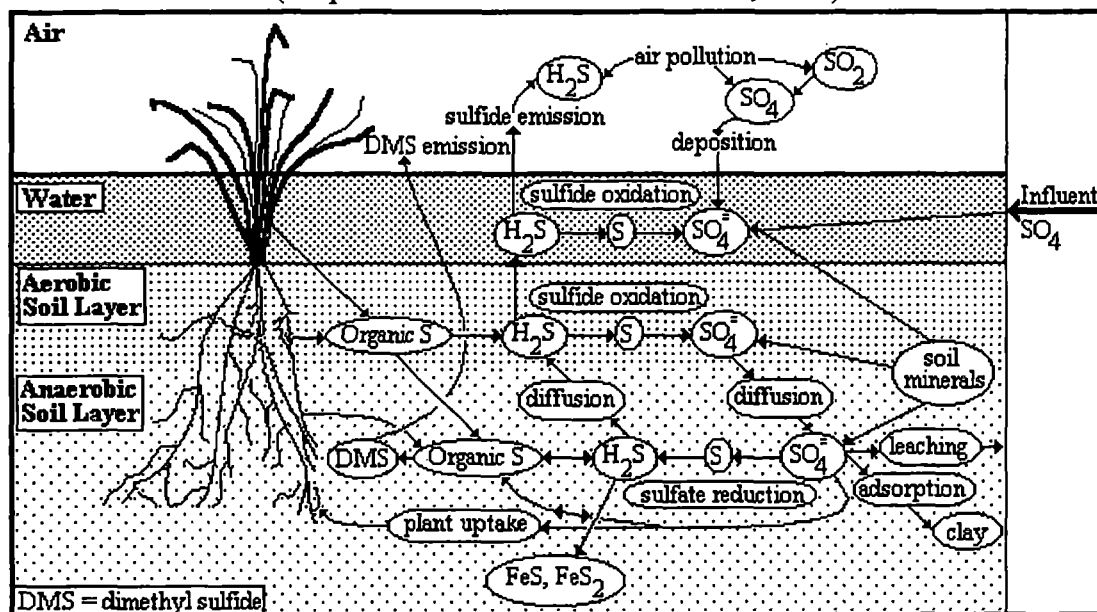
Most of the sulfur in wetlands soils is in organic form and wetlands are effective sulfur sinks and transformers (Faulkner and Richardson, 1989). The sulfur transformations shown in Figure 2.9 are dominated by biologically mediated processes. Sulfur input is typically in the form of sulfate from atmospheric deposition and terrestrial sources (Faulkner and Richardson, 1989). Sulfate input from acid mine drainage (e.g., from oxidation of pyrite, FeS_2) is of particular interest to WWT applications.

A key removal pathway for sulfate is anaerobic reduction to hydrogen sulfide by bacteria (*Desulfovibrio*). This process requires suitable organic substrate which is oxidized with sulfur as the electron acceptor (Hedin, Hyman and Hammack 1988). Hydrogen sulfide is evolved as a gas which can further react to form insoluble sulfides with metal ions. Further reduction yields methyl and dimethyl (DMS) sulfides which are evolved as gasses from the wetland (Mitsch and Gosselink, 1993). The sulfate reduction process yields alkalinity and increases the pH of acid wastewaters according to the relationship (McIntire et al., 1990):



Where treatment of ongoing sulfate inputs is required, it is necessary for plant growth to replace organic substrate decomposed in the anaerobic sulfate reduction process.

Figure 2.9: The Wetland Sulfur Cycle
(adapted from Mitsch and Gosselink, 1993)



It is possible for the products of sulfate reduction to impair wetland viability by creating hydrogen sulfide levels toxic to plants and animals, reducing available sulfur for plant growth and removing available zinc and copper as insoluble metal-sulfides. High levels of soil or wastewater iron reduce impact of hydrogen sulfide through the formation of iron sulfide and pyrite. This common process is evident in the characteristic dark color of anaerobic wetland soils which is often caused by black iron sulfide.

Oxidizing environments can result in the conversion of reduced sulfur to gaseous or soluble forms. Chemoautotrophic and photosynthetic bacteria oxidize hydrogen sulfide to sulfur and sulfites under aerobic conditions (Mitsch and Gosselink, 1993).

The sulfur cycle is important for the design of constructed wetland systems treating low pH water. As shown in Figure 2.9 sulfates can enter the system through atmospheric deposition (directly or indirectly through runoff) or from contaminated waters (e.g., acid mine drainage or AMD). Sulfates and acidity may also be generated by the oxidation of hydrogen sulfide, metal sulfides (e.g., pyrite) and decomposition of organic materials. Creation of anoxic conditions in an organic carbon rich substrate provides the conditions for reduction of the sulfate to hydrogen sulfide and, if metal ions

are present, into metal sulfide precipitates. This sulfate reduction process adds alkalinity and increases pH.

2.9 Carbon Cycle

Organic carbon compounds are created and decomposed in wetland systems. Respiration by flora and fauna oxidizes organic compounds to carbon dioxide and water in aerobic zones. Anaerobic fermentation decomposes organic matter to low molecular weight species such as ethanol and lactic acid. Methanogenic bacteria further decompose organic compounds under very reduced conditions ($E_h = -250$ to -350 mv) to form methane, but only after free oxygen, nitrates and sulfates have been reduced (Mitsch and Gosselink, 1993).

The carbon cycle governs transformation of BOD and decomposition of organic detritus. Anaerobic decomposition of carbon species is directly linked to reduction processes that can remove nitrogen and sulfur from contaminated water. In addition to the linked reactions with nitrogen and sulfur, representative carbon reactions are (Mitsch and Gosselink, 1993)

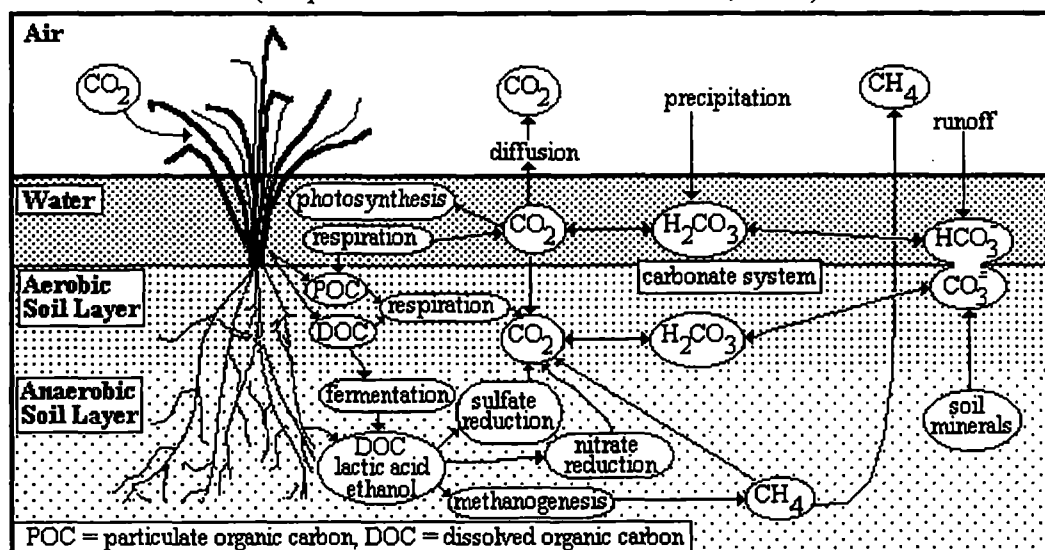
- Aerobic respiration $C_6H_{12}O_6 + 6O_2 \rightarrow 6CO_2 + 6H_2O$
- Fermentation $C_6H_{12}O_6 \rightarrow 2CH_3CH_2OCOOH$ (lactic acid)
 $C_6H_{12}O_6 \rightarrow 2CH_3CH_2OH + 2CO_2$ (ethanol)
- Methanogenesis $CH_3COO^- + 4H_2 \rightarrow CH_4 + 2H_2O$

The carbon cycle is linked to the nitrogen and sulfur cycles as discussed in Sections 2.6 and 2.8 respectively. Under reducing conditions, microbes obtain energy from decomposition of organic carbon with nitrates and sulfates acting as electron receptors. This results in a reduction pathway for the removal of nitrogen as gaseous nitrous oxide or N_2 and removal of sulfur as hydrogen sulfide. The latter reductive pathway further links to removal of metals by the formation of insoluble metal-sulfides.

Figure 2.10 illustrates carbon transformations in wetland systems.

Figure 2.10: The Wetland Carbon Cycle

(adapted from Mitsch and Gosselink, 1993)



The carbon cycle illustrated in Figure 2.10 forms the basis of most constructed systems designed to treat domestic or livestock wastewaters. Shallow aerobic zones provide for the respiration of organic carbon compounds (often reported in terms of biochemical oxygen demand or BOD). Anaerobic processes may also be used, especially for reduction of organic solids in deep water or in closed vessels such as septic tanks. As byproducts of anaerobic processes tend to be odorous, aerobic wetland sections for BOD removal are preferred. Where wetland nutrient removal is important, deeper areas that foster conditions for denitrification, follow the shallow aerobic zones. Carbon is supplied for denitrification by organic detritus from plants, organic soils or remaining BOD in the wastewater.

2.10 Iron, Manganese and other Metals

Metals are removed by sedimentation, adsorption and biochemical transformation. Adsorption and sedimentation have been discussed in preceding sections. Sedimentation removes particulate metals and metal ions adsorbed on sediment surfaces. Contact with peat and soil surface removes metals by adsorption. Chemical or biochemical formation

of insoluble metal species from soluble ions is important in removing metals from the water and in regenerating adsorption sites.

Aerobic oxidation and hydrolysis of iron and manganese remove the metals as oxyhydroxide precipitates. The reactions occur abiotically in alkaline wetlands and through metal-oxidizing bacteria in acid waters (Kleinmann, Hedin and Edenborn, 1991).

Reduction of oxidized precipitates (insoluble ferric iron to soluble ferrous iron) can form more soluble forms when aerobic sediments are flooded and become reducing. The most effective retention mechanism in reducing environments is the formation of insoluble metal sulfides (Faulkner and Richardson, 1989) and is linked to the sulfur cycle (sulfate reduction).

Bioaccumulation of metals by wetlands plants is less significant than the plants' role in providing surfaces for biofilms for organisms responsible for biochemical transformations (Kleinmann et al., 1991). Kleinmann et al. (1991) noted that *Sphagnum* accumulates iron but that it becomes petrified on extended exposure to even moderate iron concentrations. They reported that *Typha* will accumulate iron to lesser extent and is thus more tolerant of iron enriched water. Kalin (1989) reported zinc uptake by attached algal mats as a principle means of biological control of water quality from an abandoned mine site. Algae are noted for very high iron and manganese accumulation, however, the limited algal biomass in conventional WWT designs limits the significance of this metals removal route in most systems (Kleinmann et al., 1991).

High concentrations of reduced metals can render wetland soils toxic. For example, reduced ferrous iron can be oxidized in the aerobic rhizosphere which then can immobilize phosphorus and coat the roots with an impermeable iron oxide barrier (Mitsch and Gosselink, 1993).

2.11 Refractory Organics

Refractory organics, as well as, less stable hydrocarbons are removed from wetland systems through volatilization, adsorption and biodegradation (Reed et al.,

1988). Volatilization occurs across water surfaces and from liquid films (where contaminated water flows across vegetated land surfaces such as "filter strips"). Adsorption typically occurs on organic surfaces.

Volatilization can be described by first-order kinetics where atmospheric concentrations are insignificant. The half-life of a volatile organic in water can be estimated by (Reed et al., 1988, page 62):

$$t_{1/2} = (0.693y) / k_{vol} \quad \text{and} \quad k_{vol} = (B_1H) / (y(B_2 + H)M^{1/2})$$

where

- y = liquid depth (cm)
- $t_{1/2}$ = time until initial concentration is reduced by 50% (hr)
- k_{vol} = volatilization mass transfer coefficient (cm/hr)
- H = Henry's Law constant ($10^5 \cdot \text{atm} \cdot \text{m}^3 / \text{mol}$)
- M = molecular weight of the contaminant
- B_1 = 2.211 for well mixed surface
= 0.2563 for overland flow
- B_2 = 0.01042 for well mixed surface
= 0.000586 for overland flow

The foregoing relationship stems from the first-order kinetic relationship:

$$C_t = C_0 \exp(-tk_{vol} / y)$$

where

- t = time (hr)
- C_0 = initial concentration (mg/l)
- C_t = concentration at time t (mg/l)

The Canadian Water Quality Guidelines (CCME) provides information on potential removal from surface waters by volatilization for a broad range of hydrocarbons, aromatics and halogenated organics. The information suggests that for many species volatilization could be fairly rapid, for example, benzene is reported as having a $t_{1/2}$ of 4.81 hr (at 10°C) in a 1 m water column.

Adsorption relationships can be described in terms of the octanol/water partition coefficient, molecular weight of the contaminant and the fraction of carbon present in the adsorbing system. These can be correlated into a form similar to the first order relationship for volatilization, although empirical coefficients must be developed for the specific system being studied (for more information see Reed et al., 1988, page 64). The

reference notes that adsorption is thought to be the main process for refractory organic removal.

Biological decomposition of adsorbed species (Reed et al., 1988) and atmospheric decomposition of volatilized compounds (CCME) ultimately transforms them to harmless forms. Bacteria species are known that utilize benzene as a sole carbon source (CCME). Acclimated anaerobic microbes are known to dechlorinate poly-chlorinated phenols and methoxybenzenes to simpler chlorophenols given sufficient residence time (Woods, Ferguson and Benjamin, 1989). Toluene, xylene, chlorobenzene and ethylbenzene were found to be aerobically decomposed in dilute concentrations when exposed to sufficient nutrient phosphorus and nitrogen (Bouwer, 1989). The study indicated that the rate of biotransformation proceeded at about one order of magnitude greater rate than volatilization of the relatively low molecular weight aromatics. The study also showed that the rate of decomposition and volatilization for the compounds in ponds was slow (half-lives of 30 to 40 days).

3. HYDROLOGY CONSIDERATIONS IN WWT DESIGN

3.1 Overview

Hydrologic conditions define wetland characteristics. Hydrology is "*probably the single most important determinant of the establishment and maintenance of specific types of wetlands and wetland processes*" (Mitsch and Gosselink, 1993). Hydrology defines the pathways that transport nutrients and energy into, within and out of wetland systems. Hydrology is a key variable that sets the physical, chemical and biological conditions by which water contaminants are removed, transformed, cycled, suspended and dissolved within wetlands systems.

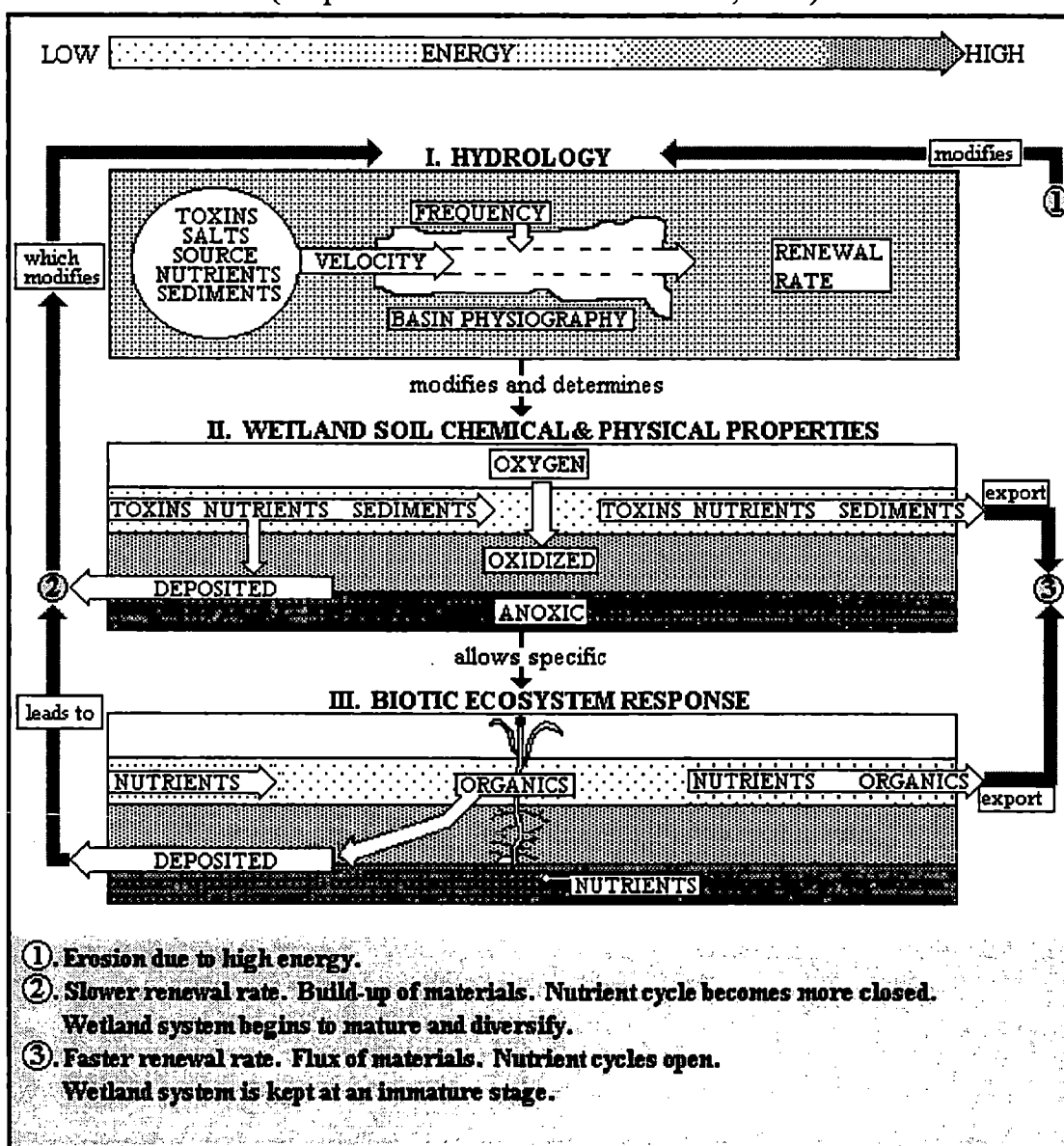
This section introduces the concepts of the wetland water budget and hydroperiod. The discussion is intended to list and briefly explain each component of the water balance. The types of wetlands hydroperiods are listed and implications on design are outlined. Detailed explanation of water flow in porous media, open channel hydrology and atmospheric impacts on wetland water balances are beyond the scope of this document. Evaluation of the parameters, however, is essential to the detailed design of any wetland project.

Wetlands are often transitional sites between upland areas and either flowing or open deep water systems. This dynamic transitional character is exploited by WWT. Such open systems are sensitive to the energy and chemistry impacts of hydrologic changes. Hammer (1992) observed that:

1. Only a relatively few plant species are adapted to submerged or flooded soils, with even fewer able to withstand long inundation and/or deeper water. Short term and/or shallow flooding typically supports higher plant species diversity. Biomass productivity, however, is not necessarily greater in the shallow or intermittently flooded systems.

2. Prolonged inundation can result in nutrient immobilization in reduced sediments. Periodic drying can oxidize the materials and return them to the water column. Such conditions can lead to large increases in biomass (e.g., algae blooms) when nutrients are thus returned to the active wetland nutrient cycles.

Figure 3.1: Model of Wetlands Hydrology
(adapted from Mitsch and Gosselink, 1993)



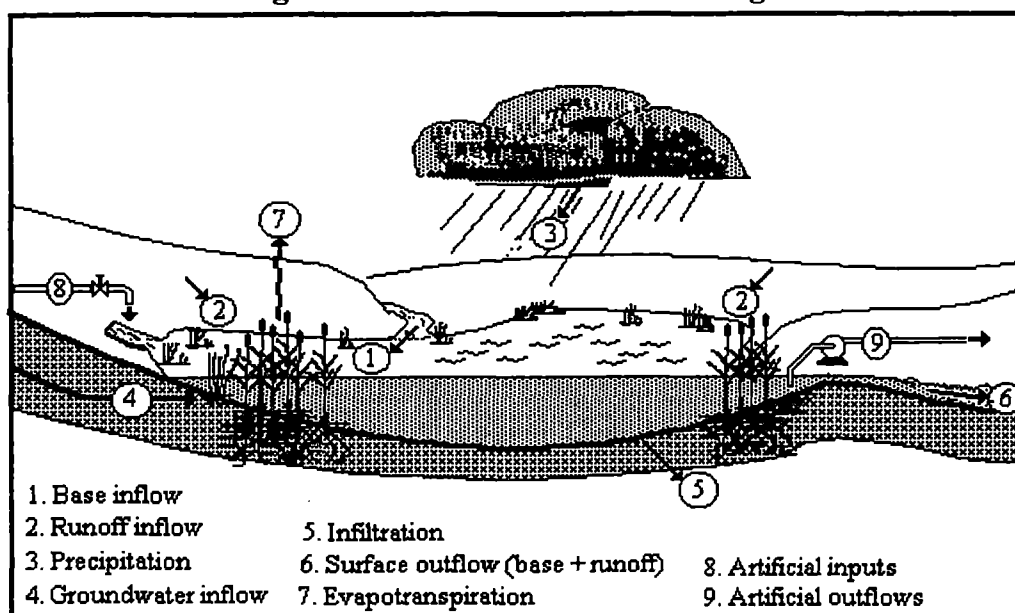
Just as hydrology influences living ecosystem composition, biotic components influence wetland hydrologic characteristics through such mechanisms as peat accumulation, sediment and nutrient retention, modification of evaporation and alteration of flow regimes (Mitsch and Gosselink, 1993, Hammer, 1992). Figure 3.1 illustrates the impacts of hydrology:

Design correlations for constructed wetlands typically focus on water composition and desired effluent quality. Understanding contaminant loads and nutrient status are critical to developing constructed wetlands that meet design goals and support healthy biotic communities. However, hydrology of proposed wetland systems must be well understood before WWT design commences, especially in the case of stormwater wetlands. Key components of the hydrological evaluation must include the water budget (on seasonal and on storm event basis) and the hydroperiod.

3.2 Water Budget

The water budget is a description of the inflows, outflows and changes in wetland water storage. The budget defines the rate of mass and energy transport into and out of wetlands and the rate of level rise or fall. Hydraulic residence time (**HRT**) within the wetland is defined by the budget which in turn defines the extent to which various rate dependent biogeochemical processes can affect concentrations of water constituents. Control of inflows and outflows through diversion and manipulation of storage volume (by changing outlet structure height, for example) are important forcing functions through which human intervention can shape wetlands ecology. Figure 3.2 illustrates the components of the water balance.

Figure 3.2: The Wetland Water Budget



The US SCS (1992) describes the water budget in the following terms:

$$\Delta S / \Delta t = Q_i - Q_o = \text{change in wetland storage per change in time}$$

and

$$Q_i = P + R_i + B_i + G_i + P_i + T_i$$

$$Q_o = ET + R_o + B_o + G_o + P_o + T_o$$

where S = storage of water in the wetland
 $= S_s + S_p$, S_s = surface water storage
 S_p = substrate water storage

t = time

Q_i = water inflow rate

Q_o = water outflow rate

P = direct precipitation on wetland surface

R_i = wetland drainage basin storm and melt runoff inflow

B_i = base or stream flow entering wetland

G_i = groundwater (seepage, springs) inflow

P_i = artificial water input (irrigation flows pumped in)

T_i = tidal inflow (where applicable)

ET = evapotranspiration from wetland surface

R_o = stormwater outflow

B_o = base outflow

G_o = infiltration or percolation outflow to groundwater

P_o = water pumped out of wetland

T_o = tidal outflow (where applicable)

Mass balances for waste constituents can be derived from the water budget. By including appropriate concentrations in the inflow and outflow streams plus parameters for storage, creation and decomposition within the wetland the foregoing relationship can be modified to describe contaminant loadings, accumulations, generation and exports. As water is the transport medium for nutrients, sediments and pollutants, the water budget defines the mass balances for the system.

Atmospheric Input/Losses: Precipitation can be evaluated from local data (Environment Canada, 1980) and must include assessment of seasonal variations, peak events, droughts, as well as, normal or average amounts. Precipitation input is the direct rain or snow fall on the wetland

Water loss by evapotranspiration is a major factor in prairie wetlands. For example, the average (1951 - 1980) Calgary lake evaporation for July of 154 mm, exceeds average July precipitation, 65.4 mm, by a considerable margin (Environment Canada, 1980). Evaluation of actual wetland evapotranspiration is difficult and estimating values for design is potentially complex. While transpiration may intuitively be expected to increase losses from wetlands, the plants retard evaporation by shading, reducing wind currents near the surface and permitting higher humidity zones to develop adjacent to the air-water interface (Kadlec, 1989). Hammer (1992) reported that most studies have found evapotranspiration from wetlands to be 30% to 90% of nearby open water evaporation. As a generalization, Hammer (1992) noted that investigations support wetland evapotranspiration as 80% of open water Class A pan evaporation (lake evaporation is 1.4 times Class A pan evaporation) For planning purposes it is suggested that evapotranspiration be approximated as lake evaporation (Kadlec, 1989, Hammer, 1992). Continuous modeling, incorporating solar input, winds, temperature and precipitation histories applicable to the site is required for detailed system design unless base flow is expected to substantially exceed evaporation losses.

Groundwater Inflow/Outflow: Wetlands may intercept aquifers and be supported by seeps or springs within the wetland complex. Such inflows are difficult to evaluate. Infiltration loss to groundwater systems can be significant for porous

substrates. Sedimentation of silt, clay and organic matter within most wetlands will reduce substrate permeability and reduce infiltration rates. Generally, subsurface losses are much less than surface and evapotranspiration out flows (Hammer, 1992).

Watershed Runoff: Runoff must be related to size and perviousness of the drainage area and the intensity of precipitation events. Procedures for evaluating runoff are given in Hodgson, Lewis and Bright (1987) for the Province of Alberta. For smaller watersheds methods such as the US Soils Conservation Service "Curve Number" methods can be used (US SCS, 1986):

$$Q_r = (P - 0.2S)^2 / (P + 0.8S)$$

where Q_r = runoff (in)
 P = rainfall (in)
 S = watershed storage or retention (in) = $(1000/CN) - 10$
 CN = runoff curve number related to watershed perviousness
 for sandy clay loam soils - $CN = 98$ for paved roads
 - $CN = 74$ for parks

Runoff volume is the runoff (Q_r) multiplied by the area of the drainage basin. The relationship describes runoff from a single event. The range of runoff inputs to the wetland must be estimated from the frequency, duration and intensity of the full distribution of annual precipitation events which will require use of a "continuous" computer simulation. Continuous simulations involve watershed evaluation based on historical data (typically 20 to 30 years of hourly data). The illustrated method is limited to small land areas and more sophisticated computer models are required for large or complex watersheds (Hodgson et al., 1987). Where available, historical records or studies of precipitation - runoff volume measurements will provide the best indication of runoff inputs. With judgment, data from analogous watersheds can be used to determine water budgets for study area wetlands following adjustment to reflect differences in drainage basin areas (Usher, 1993, pers. comm.).

Base Inflows/Outflows: In addition to runoff inflows which arrive in either sheet or channelized forms, base stream flows may also support the wetland between runoff events. Such flows need to be assessed and included in the water budget.

Outflow will be related to flow hydraulics within the wetland, the nature of wetland discharge systems and the nature of temporary wetlands water storage (US SCS, 1992). Wetlands typically perform an important pulse control in that runoff inflow peaks are attenuated by the wetland and downstream flows are stabilized (Loucks, 1990).

Artificial Subsidies/Withdrawals: Wetlands may receive artificial inputs of water. In southern Alberta, wetlands have been created in irrigation return flow systems and as irrigation subsidized waterfowl habitat. Wetlands are water reservoirs and artificial withdrawals must also be accounted for. Notably, stormwater wetlands have been used as sources of irrigation water (Palmer and Hunt, 1989).

3.3 The Hydroperiod

The water budget describes a dynamic balance between inflows and outflows that results in rise and fall of wetland water levels. The seasonal variability of inflow, outflow and storage is described as the hydroperiod. The hydroperiod describes water level and availability throughout the year and defines the plant communities that will develop in the wetland. Non-tidal hydroperiod descriptions are shown in Table 3.1.

Table 3.1: Description of Wetland Hydroperiod Types
(adapted from Mitsch and Gosselink, 1993)

Permanently Flooded	flooded throughout the year in all years
Intermittently Exposed	flooded throughout the year except in extreme drought
Semipermanently Flooded	flooded during most growing seasons
Seasonally Flooded	flooded for extended periods during growing season, but usually no surface water by end of growing season
Saturated	standing water rarely present but extended soil saturation during growing season
Temporarily Flooded	temporarily flooded during growing season with water table usually well below surface
Intermittently Flooded	soil surface usually exposed with surface water present only for unpatterned periods

Often the hydroperiod will be defined in terms of probability of certain water levels or duration of inundation (e.g., for intermittently exposed or semipermanently flooded wetland types). In the case of created wetlands, such description helps designers evaluate the type of habitat that can be created and the plant species that can be expected to survive (Usher, 1993, pers. comm., Houck, 1993, pers. comm.).

3.4 Constructed Wetland Implications

Hydrological conditions define the type of wetland ecosystem that can be expected to be sustained without intervention or conversely define the extent of interventions required to achieve a desired wetland system. The water budget can be altered by diverting runoff into or around the wetland, managing landuse within the drainage basin, altering base flows (stream diversion) into or out of the wetland, subsidizing with artificial inputs (irrigation water) and changing storage volume within the wetland through construction of deep pools, levees or level control structures (US SCS, 1992).

The literature is most explicit that a thorough understanding of the water budget and hydroperiod is essential to successful wetland projects (Kadlec, 1989, Hammer, 1992, Schueler, 1992, Mitsch and Gosselink, 1993). The most useful data will be obtained from real time measurements on the actual watershed in question. Data can also be estimated by qualified hydrological engineers using climatological statistics and computer simulations. Alternately the information can be scaled from records for an analogous watershed in the study area (the scaling should be done by an experienced hydrologist).

3.4.1 Hydroperiod Effects

Changing water levels described by the hydroperiod will affect the biogeochemistry of wetlands. Flooded systems typically develop the aerobic water column, thin aerobic soil layer and anaerobic substrates as discussed in Section 2 (e.g., Figure 2.1). Periodic exposure of the wetland soils can result in shift of reducing zones to

oxidizing conditions and conversion of compounds to potentially more soluble forms. Such chemical transformations coupled with physical changes such as drying of sediments and accumulation of dry organic matter during exposed periods can make the contaminants available for resuspension when the wetland is flooded. This is best exemplified by the poor or variable water quality performance reported for "dry" stormwater detention ponds (Pope and Hess, 1988, Schueler, Kumble and Heraty, 1992).

Hydroperiod has major effects on wetland plants. Plants have specific and differing hydrological conditions for germination and differing tolerance to flooding once established (US EPA, 1993). Bulrush transplants are sensitive to over-topping with water (Thullen and Sartoris, 1993) but can tolerate flooding and level changes once established. Periodic inundation enhances species diversity and density relative to constant dry or flooded conditions (US EPA, 1993). The author has observed that emergent plants such as cattails (*Typha sp.*) and bulrush (*Scirpus sp.*) are common in shallow more permanently flooded sections of Calgary area wetlands. Upper shore or wetland systems that are intermittently flooded appear to be vegetated with sedges (*Carex sp.*) and rushes (*Juncus sp.*) giving way to grasses (*Hordeum jubatum*, *Calamagrostis sp.*), willows (*Salix sp.*) and/or poplar (*Populus sp.*) with reducing exposure to flooded conditions. Where dense stands of emergent plants are desired to enhance WWT removal performance in prairie climates, it appears that maintenance of a more permanent water level is required.

The decline of western riparian cottonwood forests illustrates the impact of hydroperiod on vegetation. River impoundment has altered flooding events which in turn has reduced accretion of floodplain soil, limited creation of new germinating sites (point bars) and has accelerated water table recession after high water events. Poplars depend on moist, open sites for seed germination and seedlings must maintain root contact with the water table, thus impoundment induced hydroperiod alteration can reduce the rate of tree replacement (Rood and Heinze-Milne, 1988). Failure of the poplars to regenerate causes the biologically diverse riparian forests to decline into grasslands (Rood and Mahoney, 1990). Notably these observations will be important to designers wishing to establish and sustain similar woodlands as runoff filter strips or buffers to WWT systems.

3.4.2 Wetland Design Information from Hydrological Models

It will be necessary to carefully model wetland water balance and hydroperiod as part of detailed design, especially for dry, cold prairie climates. Modeling is critical for evaluation of stormwater wetlands and should be used for design of sewage and other wastewater systems.

The existence of a base flow (e.g., known continuous wastewater influent source or permanent watershed stream) reduces the risk associated with wetland design. In such instances where base flow is reasonably expected to substantially exceed evapotranspiration and infiltration, outlet structures can be readily designed to maintain and control wetland water levels. Sewage and wastewater treatment systems should have runoff diverted away from the constructed wetland thus only direct precipitation and evapotranspiration need be considered in addition to the influent and effluent flows. Even for such hydraulically simple systems, engineering simulators will have value in determining the water balance. Additionally, most of the sophisticated software is capable of evaluating sedimentation and will have value in examining system performance expectations (e.g., effectiveness of sediment forebays).

Stormwater runoff wetland systems are much more complex hydraulically, especially in the prairies. Significant runoff events can be separated by long periods of no or limited precipitation. In the absence of base flow or groundwater support, stormwater wetlands should be expected to dry out or at least suffer major water level fluctuations. These conditions need to be assessed by engineering modeling and through physical evaluation of the watershed in question.

It is the author's observation that typical drainage basin analysis done for flood control purposes fail to provide needed information such as base flow rates, runoff volumes from small events and evaluation of storm event size versus cumulative runoff. Stormwater studies often evaluate flows and performance of detention facilities to relatively infrequent major events (e.g., 1:2, 1:5, 1:25, 1:50 and 1:100 year events). As the evaluations are done to ensure storm sewers, overland flow paths and detention ponds have sufficient capacity, infiltration and evapotranspiration are often ignored (in so doing

a more conservative or safer design results in terms of flood prevention). Such engineering considerations are essential to ensure adequate capacity in stormwater wetlands and to ensure peak flows do not damage the system. Stability and viability of wetlands, however, depends on weekly or monthly conditions.

A common, if potentially complex, engineering evaluation approach will be important for design of runoff systems. Most hydraulic models provide for continuous simulation, that is, meteorological data collected over several years is “run” on a model of the study watershed. Output from the model is used to evaluate adequacy of storm systems and to predict stage height frequencies in streams and ponds associated with the runoff system. One or more events in the continuous simulation may produce more severe conditions, than the 1:100 year design storm. Data used in the model is typically hourly precipitation and temperature data from the local weather station. In Calgary, for example, this data set spans thirty years, thus the evaluations involve the processing of large amounts of data through the engineering model.

Output routines for continuous simulations will need to provide a variety of “new” information relative to that used for design of flood prevention measures. As is discussed more fully in Section 5.5, stormwater wetlands are often designed to treat about 90% of all runoff. Runoff from infrequent large events is either bypassed around the system or passed through the system with insufficient residence time for the desired level of treatment. Continuous simulation must be used to determine the “design event” that accounts for the majority of runoff for the purpose of defining wetland volume. Table 3.2 suggests the type of information required for selection of stormwater wetland treatment volume.

Hydrological evaluations including evaporation subroutines will be necessary to assess fluctuations in water level, the frequency that the system dries up and the duration of typical droughts. Such information could be displayed graphically as stage height versus time for a design case year. This information is necessary for the selection of vegetation. For example, bulrushes may be suitable for systems that are usually flooded

but grasses such as reed canary grass (*Phalaris arundinacea*) may be more tolerant of extended dry periods.

Table 3.2: Suggested Hydrological Model Output for Stormwater Wetland Design

Return Interval	Cum. % of Precip.	Rainfall Volume (watershed mm)	Runoff Volume (m ³)	Peak Inflow Rate (m ³ /s)
7 days				
14 days				
Monthly				
Bi-monthly				
Semi-annually				
Annually				
Two-year				
Five-year				

4. WETLAND AND NATURAL WASTEWATER TREATMENT PROCESSES

4.1 Overview

Wetland wastewater treatment depends on natural soil and aquatic processes for improving water quality. These processes define a related suite of technologies that are based on removal and transformation of wastewater contaminants by vegetation, soil, microorganisms and to some extent by larger animals (WPCF, 1990). The processes proceed at "natural" rates and derive energy from decomposition of compounds within the system or from solar input (direct or indirect result of photosynthesis). Natural wastewater processes are roughly divided into soil based and aquatic systems with WWT forming a subset of the latter (WPCF, 1990).

Understanding the range of natural wastewater treatment systems not only provides insight on WWT technology but also provides the basis for combining natural processes to optimize pollutant removal. Several applications discussed in Section 5, although centered around wetland concepts, incorporate components related to processes discussed in this section such as overland flow or facultative lagoons. In several cases, the design literature for specific applications such as stormwater systems does not refer to related natural wastewater treatment design correlations. Where target contaminants such as BOD or nutrients are involved, the information in this section may suggest enhanced design approaches to improve performance.

This section will firstly overview natural processes related to WWT and then more fully discusses the types of WWT.

4.2 Soil Based Natural Wastewater Treatment Processes

Soil based systems are classified by effluent discharge point, flow path and wastewater application rate. Influent is applied below soil grade in leach fields and adsorption beds, however, wastewater is surface applied in slow rate (SR), rapid

infiltration (**RI**) and overland flow (**OF**) systems (WPCF, 1990). All the technologies can be used for sewage treatment with SR and RI related to stormwater infiltration systems and OF to runoff filter strips or riparian buffer zones. System sizing is based on what the WPCF (1990) terms as the limiting design parameter (**LDP**) concept.

4.2.1 Subsurface Systems

Subsurface Wastewater Infiltration: The typical subsurface system is the septic tank leach field or subsurface wastewater infiltration system (**SWIS**). Area requirements for SWIS systems can be estimated from (WPCF, 1990):

$$A = 1.5Q / k$$

where A = leach field area (m²)
 Q = design wastewater rate (m³/day)
 k = soil permeability (m³/m²·day)

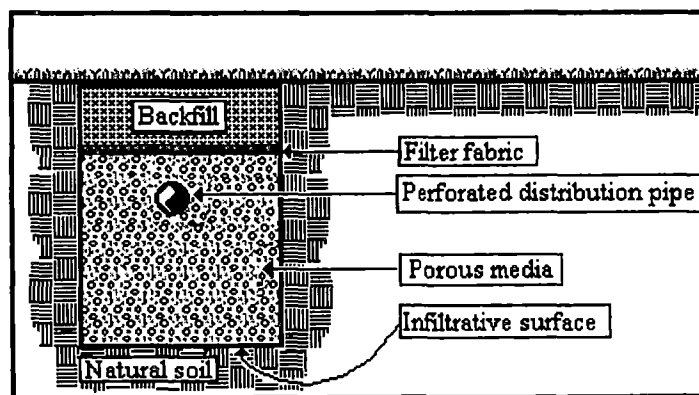
A septic tank provides pretreatment to remove settleable solids, oil and grease with a capacity defined by (US EPA, 1980):

V = 2 to 3 times Q	Q < 2.8 m ³ /day
V = 1.5Q	2.8 m ³ /day < Q < 5.7 m ³ /day
V = 4.26 + 0.75Q	5.7 m ³ /day < Q < 56.8 m ³ /day

where V = capacity (m³) of septic tank (at upper sludge level)
 Q = daily wastewater flow (m³/day)

Septic tank effluent flows by gravity or is pumped into perforated distribution pipes buried in the soil as illustrated in Figure 4.1.

Figure 4.1: Subsurface Infiltration System



Water treatment potential is high, with sandy, unsaturated soils capable of greater than 95% BOD, 10% to 40% nitrogen, 85% to 95% phosphorus and greater than 99.9% fecal coliform removal in 0.6 m of infiltration (WPCF, 1990). Design LDP is soil hydraulic conductivity and representative loading rates are given in Table 4.1:

Table 4.1: Hydraulic Loading Rates for Subsurface Infiltration
(source: WPCF, 1990)

Based on:	
-rate on horizontal infiltrative surface	
-domestic sewage	
Soil Texture (1 m min. thickness)	Hydraulic Loading Rate (cm/day)
Gravel, very coarse sand	not recommended
Coarse to medium sand	4.0
Fine sand, loamy sand	3.2
Sandy loam, porous loam	2.1
Loam, silt loam	2.5
Clay loams	1.0
Clays	0.5
Clay loams with shrink-swell potential	not recommended

In addition to unsuitable soil types, SWIS are limited by shallow depth (less than 0.6 to 1.2 m) to bedrock and high water tables (US EPA, 1980). In such situations artificial mounds have been constructed to provide sufficient infiltration zones (see US EPA, 1980 page 239 for design information). The Alberta Government has published a summary of regulatory and Canadian code requirements that includes design procedure information for septic fields and mounds (Alberta Labour, 1990).

The capital costs of conventional septic fields are estimated between \$ 15 and \$30 per m² (1988 US dollars) excluding land (WPCF, 1990).

Buried Wastewater Filters: A related approach is the use of buried wastewater filters constructed of sand or peat. Such systems are typically drained as illustrated in Figure 4.2. Studies of buried sand filters report reduction of septic tank effluent BOD and suspended solids to less than 15 mg/l, however, nitrate in the treated effluent ranged from 17 to 32 mg/l (US EPA, 1980, Table 6-4).

Three basic types of sand filters are described by the US EPA (1980). The illustration is based on a once-through buried filter as illustrated in Figure 4.2. Such systems can be protected against freezing weather and are used in Canada. The second type is a once through free access filter where influent is applied to the sand surface. The exposed surface permits ready maintenance and inspection of the sand bed and higher hydraulic loadings are used. The third type is a recirculating filter consisting of a free access sand bed and a system for recycling treated effluent. Recirculation ratios range from 3:1 to 5:1 with the latter preferred (US EPA, 1980). The complexity of the recycle tank and pump is offset by smaller required sand beds. The latter two systems will require insulated covers in northern climates. It is possible to design buried filters for recirculation, although specific procedures were not reviewed by the author. Sand filter design criteria are provided in Table 4.2.

Figure 4.2: Subsurface Sand Filter
(adapted from US EPA, 1980)

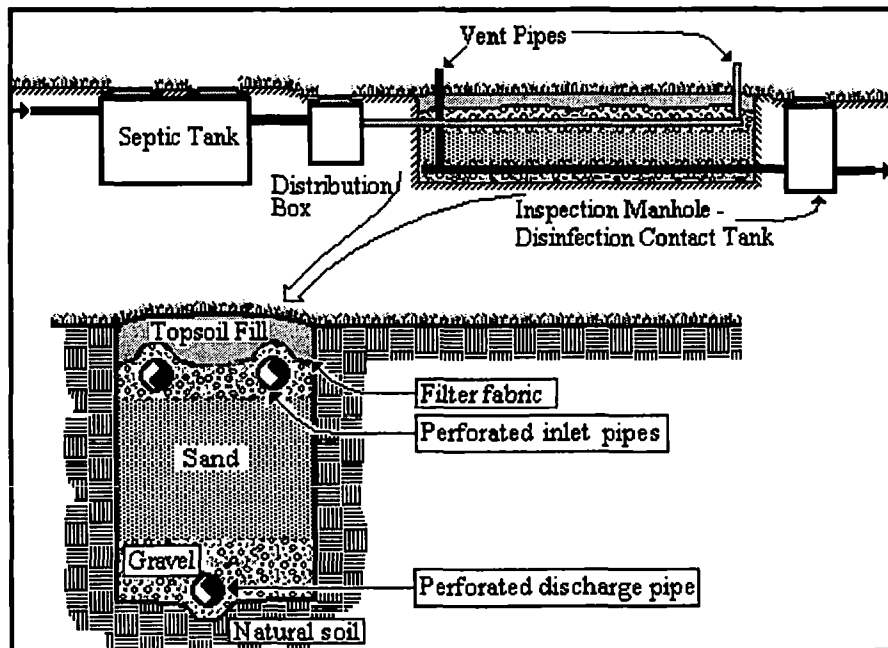


Table 4.2: Sand Filter Design Criteria
(adapted from US EPA, 1980)

	Buried Sand Filter	Free Access Sand Filter	Recirculating Sand Filter
Pretreatment	Sedimentation (septic tank)	Sedimentation (septic tank)	Sedimentation (septic tank)
Hydraulic Loading	<0.04 m/d (all year) <0.08 m/d (seasonal)	0.08 - 0.20 m/d (septic tank) 0.20 - 0.40 m/d (aerobic)	0.12 - 0.20 m/d
Media particle size Uniformity. Coef. Depth	Washed sand (<1% organic) 0.50 - 1.00 mm <4.0 (<3.5 preferred) 61 - 91 cm	Washed sand (<1% organic) 0.35 - 1.00 mm <4.0 (<3.5 preferred) 61 - 91 cm	Washed sand (<1% organic) 0.30 - 1.50 mm <4.0 (<3.5 preferred) 61 - 91 cm
Distribution Bedding Venting	Perforated pipe 25 cm washed gravel gravel 1.9 - 6.3 cm dia. Downstream end	Surface troughs, sprinkler -- --	Surface troughs, sprinkler -- --
Underdrains Slope Bedding Venting	Perforated pipe 0.5 - 1.0 percent 25 cm washed gravel gravel 0.6 -3.8 cm dia. Upstream end	Perforated pipe 0.5 - 1.0 percent 25 cm washed gravel gravel 0.6 -3.8 cm dia. Upstream end	Perforated pipe 0.5 - 1.0 percent 25 cm washed gravel gravel 0.6 -3.8 cm dia. Upstream end
Cover	topsoil (<15 cm) 300 cm < depth <600 cm*	Insulated cover required	Insulated cover required
Dosing	Flood entire filter during dosing cycle at least twice per day	Flood surface to 5 cm at least twice per day	Flood filter surface to 5 cm or pump 5-10 min. every 30 min or empty recirc. tank in less than 20 min.
Recirculation Tank	--	--	Volume > Daily flow
*Based on design topsoil cover depths for septic drain fields (Alberta Labour, 1990)			

The US EPA manual does not deal with peat based systems although facilities have been constructed. Hydraulic loadings are reported as 4.1 cm/day for a subsurface peat system (Brooks, 1987). The US EPA manual does, however, note that anthracite, garnet, ilmenite, activated carbon and mineral tailings have been used in the filters. The media must not be water soluble and must be durable. Clay, loam, limestone or organic matter in the media may, in time, lead to clogging problems (US EPA, 1980).

The systems require little maintenance and operational input and depend on filtration, sedimentation and biochemical activity. Biological growth within the filter,

ranging from bacteria to annelid worms transforms wastes in the influent. Dosing is critical to successful operation of the filters and it is recommended the beds be flooded at least twice per day. Maintenance for free access filters includes quarterly raking of the sand surface to break up any crusts that form and occasional media replenishing or replacement. In addition, regular inspection and servicing of septic tanks and pumps will be needed. Buried filters can be “rested” for a period of time to allow break down of clogging caused by organic matter. New or inactive beds should not be started during the winter (US EPA, 1980). In order to facilitate resting of the filters if they become clogged, it would appear prudent to design larger systems with redundant beds.

4.2.2 Slow Rate Land Treatment (SR)

The SR or wastewater irrigation process involves controlled application of pretreated (at least with primary settling) wastewater to a vegetated surface. Shallow groundwater, climate, type of vegetation and soil properties all affect the design of SR systems. Typical hydraulic loading rates vary between 0.5 to 6 m/yr. The systems depend in part on vegetation for treatment mechanisms, are operationally limited to growing seasons and wastewater storage is needed for northern climates (Reed et al., 1988, WPCF, 1990).

Based on a six month operational period (growing season), required land area can be initially estimated as (WPCF, 1990):

$$A = 0.03Q$$

where A = area required (ha)
 Q = average daily wastewater flow (m³/day)

Treatment effectiveness is reported as 94% to 99% for BOD, 66% to 94% for nitrogen and 99% for pathogens (WPCF, 1990). Soil adsorption and bacteria oxidation reduce BOD with typical systems operating at 10 kg/ha-day loading rates. Crop uptake, biological nitrification-denitrification, ammonia volatilization and soil storage are responsible for nitrogen removal. Phosphorus is rapidly adsorbed in soil systems with subsequent precipitation with iron, aluminum or calcium. Plant uptake accounts for 10% to 25% of applied phosphorus removal. Pathogens are removed by adsorption followed

by die-off (WPCF, 1990). Design guidelines have been developed by the Province of Alberta (Alberta Environment, 1984) and LDP correlations for hydraulic loading, phosphorus removal and nitrogen treatment are discussed in detail by the WPCF (1990).

SR systems are usually stand alone entities but could have application as either post or pre treatment processes in an integrated WWT system design. Wetlands could be used for effluent "polishing" where SR treatment rates exceed natural infiltration - vegetation evapotranspiration and drainage occurs.

4.2.3 Rapid Infiltration Systems (RI)

RI involves application of wastewater to basins with permeable soils. Treatment mechanism are similar to SR concepts except that vegetation does not play a role. Cyclic loading is used to restore aerobic conditions in the infiltration zone. Land area requirements can be estimated by (WPCF, 1990):

$$A = 0.0064 Q$$

$$\begin{array}{ll} \text{where } A & = \text{RI treatment area (ha)} \\ Q & = \text{daily wastewater flow (m}^3\text{/day)} \end{array}$$

Design criteria depend on influent quality and most importantly on soil hydraulic conductivity. Hydraulic mounding may limit application rates and must be considered in design. Depth to bedrock or groundwater should be at least 5 m unless underdrains are installed (WPCF, 1990). A BOD:nitrogen ratio of 3:1 or greater is needed to provide organic carbon for the denitrification reaction (Reed et al., 1988). Pretreatment is important as the systems depend on long term permeability of the substrate and plugging of the soil system can lead to early failure (WPCF, 1990).

Percolate quality, when it enters critical aquifers or discharges into surface waterbodies, defines the limits of potential RI applications. Although the systems can meet drinking water standards, it is suggested installations be designed for lateral flow with percolate emergence as subflow in nearby surface waters (WPCF, 1990).

Performance and loading rate information are shown in Table 4.3.

Table 4.3: Typical RI System Performance

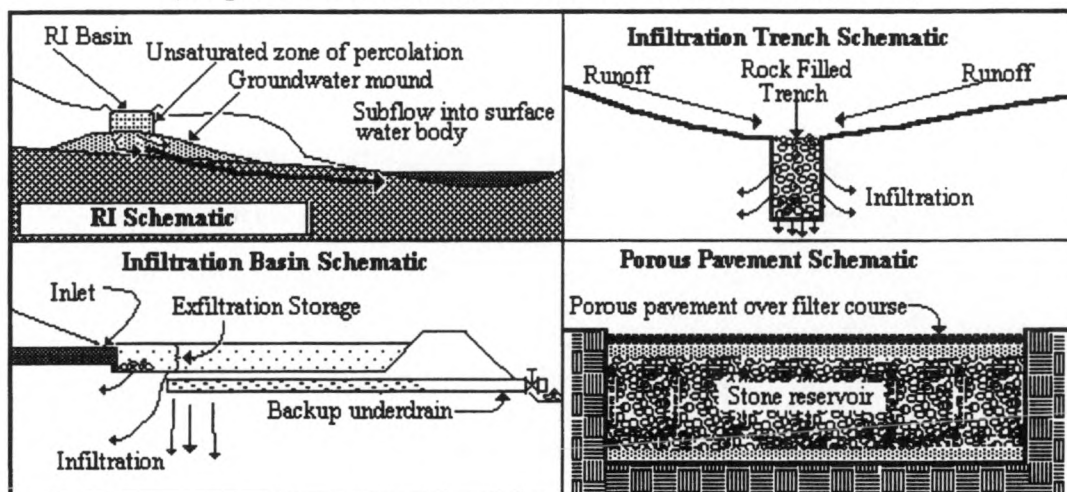
(source: WPCF, 1990)

Parameter	Loading (kg/ha day)	Removal %	Factors Influencing Better Performance
BOD	45 - 177	86 - 100	good design
Nitrogen	3.4 - 41	10 - 93	pretreatment, BOD:N, loading rate
Phosphorus	1.0 - 13.2	29 - 99	travel distance
Fecal coliform	--	2 to 6 logs	soil texture, travel distance, resting time
Suspended solids	33.6 - 112	100	rest cycles, surface maintenance

RI systems described in sewage treatment literature (Reed et al., 1988, WPCF, 1990) are closely related to stormwater infiltration trenches, infiltration basins and porous pavements used to manage runoff. The later features are designed to route stormwater into groundwater systems and away from surface flows. The stormwater systems are compared with RI in Figure 4.3:

Figure 4.3: Comparison of RI with Stormwater Infiltration Systems

(adapted from Reed et al., 1988 and Schueler et al., 1992)

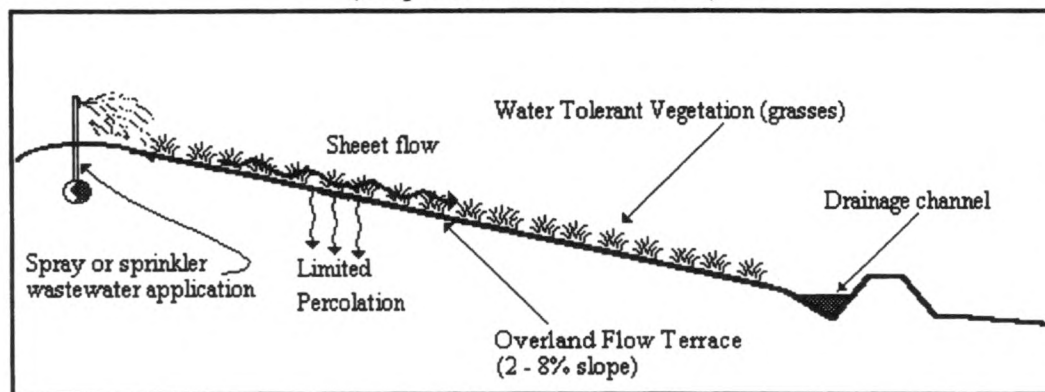


Reviews of infiltration stormwater systems indicates an unacceptable failure rate (the majority of studied systems had failed) from plugging and poor maintenance (Scheuler, 1992). RI systems require surface scarification to maintain infiltration rates and have been reported as problematic if improperly designed (WPCF, 1990).

4.2.4 Overland Flow Systems (OF)

OF technology has particular relevance for WWT, especially for design of integrated runoff control and water quality improvement systems. Grass covered or forested "filter strips" as well as grassed swales are stormwater management techniques closely related to OF processes. Meadows which incorporate OF system concepts are part of agricultural wastewater treatment applications discussed in Section 5. For those reasons, design details of OF will be discussed in more detail than those for SWIS, SR and RI processes. Figure 4.4 illustrates components of an OF system:

Figure 4.4: OF System Schematic
(adapted from WPCF, 1990)



OF systems use sheet flow over vegetated relatively impermeable soils. OF systems are designed only as discharging systems unlike SWIS, SR and RI systems that can be designed to handle inflows completely by percolation into groundwater and evapotranspiration. The systems are constructed on clay and clay loam soils, require at least 1 m depth to bedrock or groundwater and are built on slopes of 2% to 15%.

Scoping level land area requirements, based on six month treating season (cold season storage will be required), can be estimated by (WPCF, 1990):

$$A = 0.018 Q$$

where A = RI treatment area (ha)
Q = daily wastewater flow (m³/day)

OF systems are typically designed with field lengths of 30 to 60 m (depends on influent loading and target effluent quality) with optimum slopes of 2% to 6% (Reed et

al., 1988). Application is typically done in periods of 6 to 12 hours/day, 5 to 7 days/week (Reed et al., 1988). The wet:dry ratio (time irrigated : time system off) is usually less than 1:1 with lower ratios (e.g., longer relative dry periods) recommended for ammonia removal (WPCF, 1990). The extended time provides both for ammonia volatilization and nitrification.

OF systems can be designed to treat screened primary sewage to produce advanced secondary effluent with BOD and suspended solids reduced to 15 mg/l (Reed et al., 1988). Table 4.4 summarizes expected performance for municipal wastewater OF treatment:

Table 4.4: OF System Performance

Contaminant	Typical Effluent & Performance	Reference
BOD	reduced to 15 mg/l in effluent up to 95% BOD removal	Reed et al., 1988 WPCF, 1990
Suspended Solids	reduced to 15 mg/l in effluent reduced to 10 mg/l (not for algae)	Reed et al., 1988 WPCF, 1990
Nitrogen	up to 90% removal of ammonia low removal of nitrate N up to 80% removal of ammonia	Reed et al., 1988 Reed et al., 1988 WPCF, 1990
Phosphorus	20 - 60 % removal typical 90 - 98% removal with alum addition	WPCF, 1990 WPCF, 1990
Metals	Cd, Ni, Cu, Zn removal 84 - 94%	WPCF, 1990
Pathogens	not effective for fecal coliforms up to 85% removal of enteric virus	WPCF, 1990 WPCF, 1991
Volatile Organics	removal can be significant	WPCF, 1990

In order to refine sizing estimates, more detailed design correlations are used to establish application rates and slope length which in turn can produce more accurate estimates of land area. The following relationships for application rate and hydraulic loading are firstly evaluated and the results are used to define organic loading rate. Area can be calculated from application rate and organic loading rate correlations with the larger calculated area selected as the design basis.

The hydraulic loading rate, L_w , is described by (Reed et al., 1988):

$$L_w = [qP(100\text{cm/m})] / Z$$

where L_w = hydraulic loading rate (cm/day)
 q = application rate per unit width ($\text{m}^3/\text{hr}\cdot\text{m}$)
 P = application period (hr/day)
 Z = slope length (m)

The application rate, q , is described by (Reed et al., 1988, WPCF, 1990):

$$(C_z - C_r) / C_o = A \exp[-(KZ)/q^n]$$

where C_z = BOD at down slope point Z (mg/l)
 C_r = design BOD residual at base of slope (mg/l)
 $\quad\quad\quad = 5 \text{ mg/l}$
 C_o = influent BOD (mg/l)
 A, K and n are empirical constants

Both Reed et al. (1988) and WPCF (1990) provide solutions for the foregoing correlation in the form of a family of curves relating slope length to BOD reduction at constant application rates. The iterative solution is a matter of selecting a application rate that produces a practical field length and ultimately surface area. The graphical solution is reported to be valid for screened raw and primary treated sewage but not for high strength ($> 400 \text{ mg/l BOD}$) wastewaters (Reed et al., 1988). The figures in the two references are approximated by:

for $q = 0.10 \text{ m}^3/\text{hr}\cdot\text{m}$,	$\log[(C_z - 5) / C_o] = -0.43 - (0.0257 Z)$
for $q = 0.16 \text{ m}^3/\text{hr}\cdot\text{m}$,	$\log[(C_z - 5) / C_o] = -0.40 - (0.0244 Z)$
for $q = 0.25 \text{ m}^3/\text{hr}\cdot\text{m}$,	$\log[(C_z - 5) / C_o] = -0.28 - (0.0235 Z)$
for $q = 0.37 \text{ m}^3/\text{hr}\cdot\text{m}$,	$\log[(C_z - 5) / C_o] = -0.14 - (0.021 Z)$

Organic loading rate, believed to be limited to $100 \text{ kg / ha}\cdot\text{day}$ by the oxygen transfer capability of the thin wastewater film, is described by the relationship (Reed et al., 1988):

$$L_{\text{BOD}} = 0.1 L_w C_o$$

where L_{BOD} = BOD loading rate ($\text{kg/ha}\cdot\text{day}$)
 L_w = hydraulic loading rate (cm/day)
 $\quad\quad\quad = qPW_m/Z$
 W = application slope width (m)
 m = 100 cm/m (conversion factor)

Land area requirement is a function of flow, application rate, slope length and application period, as well as limiting organic loading rate. Land area can be estimated as the greater of the following relationships (from Reed et al., 1988):

$$A_s = (QZ) / (10000qP)$$

and

$$A_s = 0.1QC_o / L_{BOD}$$

where A_s = OF surface area (ha)
= WZ

Q = design flow rate to OF site (m³/day)
= $365 Q_d / T_p$

Q_d = design average wastewater flow rate (m³/day)

T_p = number of treating days
(150 - 180 days for cold climates)

Designs derived from the correlations should be tested against guidelines presented in Table 4.5:

Table 4.5: OF System Design Guidelines
(source: WPCF, 1990)

Parameter	Pretreatment Level		
	Screening	Primary	Secondary
Application rate (m ³ /m hr)	0.2 - 0.4	0.2 - 0.4	0.25 - 0.6
Slope length (m)	30 - 45	30 - 45	30 - 45
Application period (hr)	8 - 12	8 - 12	8 - 12
Drying period (hr)	16 - 12	16 - 12	16 - 12

While the correlations presented here pertain primarily to sewage, it is anticipated they could be used to evaluate runoff treatment filter strip designs by ensuring adequate treatment areas are provided based on hydraulic loading and priority pollutant design considerations. The additional design factors may serve to improve the effectiveness of runoff filter strips which are reported by Schueler et al. (1992) to be inconsistent.

4.3 Aquatic Systems

Aquatic wastewater systems include lagoons, aquatic plant systems and wetlands wastewater treatment processes. The latter includes free water surface (FWS), subsurface

flow (SFS) and peatland or natural wetlands approaches which will be discussed in Section 4.4. All aquatic systems take advantage of the physical and biogeochemical processes described in Section 2 to remove and transform wastewater contaminants. To date, design procedures have been developed primarily based on sewage treatment (such as Crites et al., 1988, Reed et al., 1988, Watson et al., 1989, WPCF, 1990) but applications for runoff water quality improvement (Hartigan, 1988, Livingston, 1988, Schueler, 1992), acid mine drainage (Hedin and Nairn, 1992) and agricultural wastes (Hammer et al., 1991) have also been developed.

The design procedures in Reed et al. (1988) and WPCF (1990), although primarily focused on sewage, are particularly complete and consistent and will form the basis of this section. Specific modification of design procedures for non-sewage applications will be discussed in more detail in Section 5.

4.3.1 Lagoons

Lagoons or stabilization ponds include sewage treatment aerobic, facultative, partial mix and controlled discharge lagoons, as well as runoff "wet" ponds. Aerobic ponds are restricted to warmer climates (Reed et al., 1988, WPCF, 1990) and will not be discussed further. Runoff wet ponds are primarily sized on hydraulic considerations, however, where removal of target contaminants is desired, the design correlations discussed for facultative ponds can have value.

Facultative Lagoons: Facultative lagoons with depths of 1.2 to 2.5 m rely on aerobic stabilization in upper water layers and anaerobic decomposition in deeper water and sediments for transformation of pollutants (WPCF, 1990). Oxygen is supplied both by surface aeration and algae photosynthesis. Lagoon systems may be non-discharging (where annual evaporation exceeds precipitation), controlled discharge and continuously discharging. Where the lagoon systems serve small communities in temperate climates controlled discharge or non-discharge systems are often found.

Sewage treatment lagoons are subject to prescriptive guidelines in Alberta (Alberta Environment, 1988 and 1991). Minimum requirements for controlled discharge

systems are based on a 60 day residence time treatment cell followed by a one-year capacity storage cell. Evaporative systems require at least two 1.5 m deep cells with a 60-day treatment cell plus an evaporation pond with 3 to 5 years capacity (Alberta Environment, 1991). The typical controlled discharge system is illustrated in Figure 4.5:

Figure 4.5: Typical Alberta Sewage Lagoon System
(adapted from Alberta Environment, 1991)

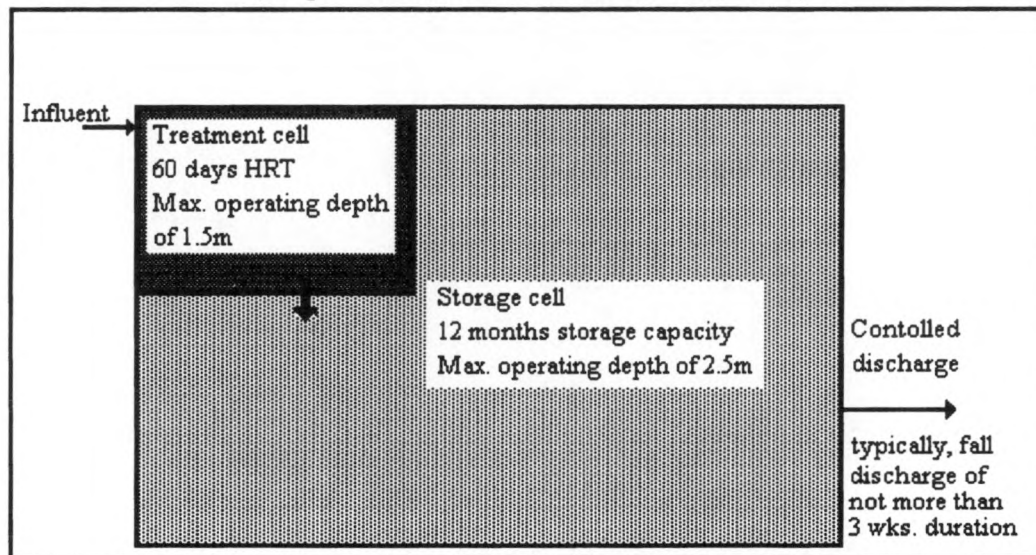


Table 4.6 presents four empirical correlations for BOD removal discussed in Reed et al. (1988), WPCF (1990) and US EPA (1983). The relationships are reported to produce satisfactory results, although difficulty in applying them may result from defining the coefficients or from the complexity of the relationships.

Short circuiting as well as, spring and fall overturn (when thermal stratification of pond is destabilized by changing ambient temperatures) can result in high effluent contaminant loadings not predicted by the correlations which otherwise can reasonably estimate performance. Of the correlations in Table 4.5, Reed et al. (1988) noted that the plug-flow model adequately describes facultative ponds.

The simplest approach to rational design is based on organic loading (BOD) with 11 - 22 kg / ha·day recommended for cold climates such as the northern prairies (Reed et

al., 1988, WPCF, 1990). The references further recommend first cell loading be less than 40 kg / ha-day and advise total system retention times of 120 to 180 days.

Table 4.6: Facultative Pond Design Correlations

(Reed et al., 1988 and WPCF, 1990)

GLOYNA EQUATION $V/Q = t = (3.5 \times 10^{-5}) L_a \theta^{(35-T)} f f'$ where V = pond volume (m³) Q = influent rate (l/day) L_a = influent BOD (mg/l) θ = temperature coefficient = 1.085 T = pond water temperature (°C) f = algal toxicity factor = 1.0 for domestic waste f' = sulfide toxicity factor = 1.0 if sulfate < 500 mg/l	COMPLETE MIX MODEL $C_n / C_o = [1 / (1 + k_c t_n)]^n$ where max. primary cell BOD, (C_e)_{max} = 55 mg/l depth defined by: (C_e)_{max} = 700 / (0.6d + 8) C_n = effluent BOD (mg/l) C_o = influent BOD (mg/l) k_c = complete mix first order rate (1/days) $k_{cT} = k_{c35} (1.085)^{(T-35)}$ $k_{c35} = 1.2 \text{ days}^{-1}$ t_n = hydraulic residence time (days) n = number of equal sized cells in series d = depth (ft)														
PLUG FLOW MODEL $C_e / C_o = e^{(-k_p t)}$ where C_e = effluent BOD (mg/l) C_o = influent BOD (mg/l) k_p = rate constant = k₂₀(1.09)^(T-20) t = hydraulic residence time (days) <table border="1" data-bbox="396 1351 674 1698"> <thead> <tr> <th colspan="2">Rate Constants (k₂₀)</th> </tr> <tr> <th>BOD Loading (kg/ ha d)</th> <th>k</th> </tr> </thead> <tbody> <tr> <td>22</td> <td>0.045</td> </tr> <tr> <td>45</td> <td>0.071</td> </tr> <tr> <td>67</td> <td>0.083</td> </tr> <tr> <td>90</td> <td>0.096</td> </tr> <tr> <td>112</td> <td>0.129</td> </tr> </tbody> </table>	Rate Constants (k ₂₀)		BOD Loading (kg/ ha d)	k	22	0.045	45	0.071	67	0.083	90	0.096	112	0.129	WEHNER - WILHELM EQUATION $C_e / C_o = [4a e^{(1/2D)}] \div [(1+a)^2 (e^{(a/2D)} - (1-a)^2 e^{(-a/2D)})]$ where C_e = effluent BOD (mg/l) C_o = influent BOD (mg/l) $a = (1 + 4ktD)^2$ k = first order rate constant (1/days) t = hydraulic residence time (days) D = dispersion number (dimensionless) $= H / vL = Ht / L^2$ H = axial dispersion coefficient v = fluid velocity L = particle travel path length • requires trial and error solution • see Figure 4.1 Reed et al. (1988) or Figure 7.1 WPCF (1990)
Rate Constants (k ₂₀)															
BOD Loading (kg/ ha d)	k														
22	0.045														
45	0.071														
67	0.083														
90	0.096														
112	0.129														

Water temperature for the correlations can be estimated by (Reed et al., 1988):

$$T_w = (A/T_a + QT_i) / (Af + Q)$$

where T_w = pond water temperature (°C)
 T_a = ambient air temperature (°C)
 T_i = influent water temperature (°C)
 A = pond surface area (m²)
 Q = wastewater flow rate (m³/day)
 f = factor = 0.5

Pathogen removal in lagoon systems (and other aquatic and wetland processes) occurs through die-off, sedimentation, adsorption and predation (Reed et al., 1988). Multiple-cell pond systems, where three cells are used in series and where detention times are based on BOD removal (or at least 20 days HRT), are reported to be effective in meeting most bacteria and virus discharge limits without disinfection (WPCF, 1990). Reed et al. (1988) suggests that fecal coliform removal can be estimated by the relationship:

$$C_f / C_i = 1 / (1 + tk_T)^n$$

where C_i = influent fecal coliform (number per 100 ml)
 C_f = effluent fecal coliform (number per 100 ml)
 t = cell detention time (days)
 n = number of same volume cells in series
 k_T = rate constant (1/days)
 $= 2.6(1.19)^{(T_w-20)}$
 T_w = mean pond temperature (°C)

Because short circuiting within lagoons is typical, the residence time, t , must be determined from tracer studies or estimated as about 50% of design (Reed et al., 1988).

Performance capabilities notwithstanding, lagoon effluent suspended solids can be high due to algae and plant detritus from biological sources within the pond. Levels of over 100 mg/l are possible during warm seasons (Reed et al., 1988). Where the solids must be removed from effluent streams, devices such as sand or rock filters, microstrainers and downstream polishing (aquaculture or WWT) must be used. Design of the filters is covered in engineering texts such as Ramahlo (1983) or Metcalf and Eddy (1991).

Where specific phosphorus discharge limits exist, the design manuals (Reed et al., 1988, WPCF, 1990) recommend chemical addition (alum at 150 - 225 mg/l or, ferric chloride at 20 mg/l). Batch additions of the chemicals to intermittent discharge lagoons are reported as cost effective means to limit phosphorus to less than 1 mg/l in the effluent (WPCF, 1990).

Correlations have been developed to describe nitrogen removal with two models discussed in the design references (Reed et al., 1988, WPCF, 1990):

$$N_e = N_o \exp[-k_T(t + 60.6(\text{pH} - 6.6))]$$

and

$$N_e = N_o / [1 + t(0.000576T - 0.00028)e^{(1.080 - 0.042T)(\text{pH} - 6.6)}]$$

where N_e = effluent total nitrogen (mg/l)
 N_o = influent total nitrogen (mg/l)
 k_T = rate constant (1/days) = $0.0064 (1.039)^{(T - 20)}$
 T = pond water temperature (°C)
 t = detention time (days)
 pH = pH of near surface bulk liquid = $7.3 e^{0.0005 \text{ ALK}}$
 ALK = expected effluent alkalinity (mg/l)

Stabilization ponds can be very effective in reducing nitrogen with elimination of up to 95% possible (WPCF, 1990).

Design correlations for BOD, pathogens and nitrogen have been discussed in the foregoing. Where effluent limits for all three are regulated, design calculations need to identify the limiting design parameter or LDP that requires the largest treating volume. Sizes predicted by the correlations should provide ample conditions for sedimentation provided channeling can be controlled.

Deep water cells such as storage lagoons, wet ponds and sedimentation pools are often integrated with wetlands applications. The foregoing lagoon performance estimation procedures should be used to account for pollutant removal in the cells which will reduce needed wetland area.

Controlled Discharge Ponds: Controlled discharge ponds are facultative systems where effluent is stored and released intermittently. The sewage lagoons defined by Alberta guidelines fit this model. Where HRT is less than 120 days the facultative

correlations can be used (Reed et al., 1988, WPCF, 1990). Where longer detention periods are used the two references recommend design factors shown in Table 4.7:

Table 4.7: Controlled Discharge Lagoon Design Factors
(after Reed et al., 1988 and WPCF, 1990)

Parameter	Design Factors
Overall organic loading	22 to 28 kg / ha·day
Liquid depth	less than 2 m in first cell less than 2.5 m in subsequent cells
HRT	minimum 6 months storage above 0.6 m water depth or storage greater than the period of ice cover
Number of cells	at least three cells piped for series <i>and</i> parallel flow
Discharge procedure	cell to be emptied is isolated from inflow and tested for water quality (e.g., BOD, suspended solids, dissolved oxygen) if quality is suitable and weather favorable the contents can be discharged to a residual level of 45 to 60 cm

Partial-mix Aerated Ponds: Partial-mix ponds involve mechanical aeration to increase oxygen availability for biological oxidation (WPCF, 1990). The systems are not designed to keep all solids suspended as in conventional activated sludge sewage processes. The partial-mix concept can accelerate BOD removal and reduce pond area requirements. The concept can be applied as a retrofit to facultative ponds where increasing organic loads or tighter effluent quality regulation exceeds the capability of the simple lagoons.

As this report is primarily interested in natural or passive wastewater treatment systems, correlations for partial-mix systems will not be discussed. Design procedures can be found in US EPA (1983), Reed et al. (1988) and WPCF (1990) and involve first order kinetics similar to those discussed for facultative ponds.

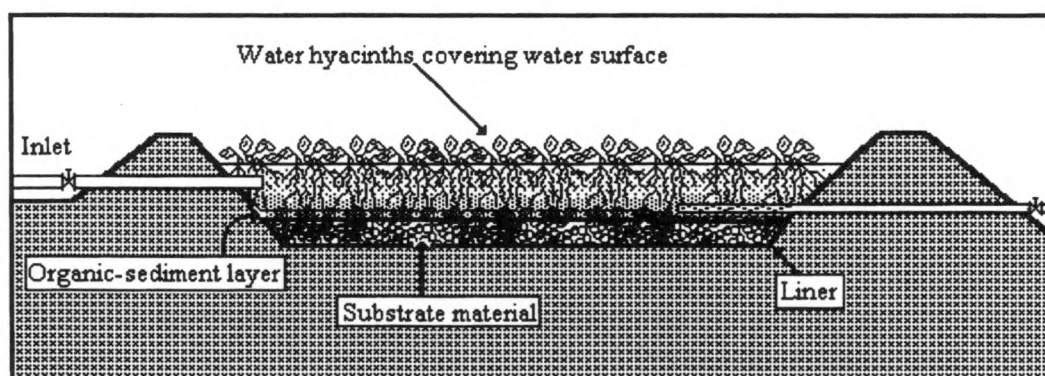
4.3.2 Aquatic Plant Systems and Aquaculture

Aquatic systems are similar to wastewater treatment lagoons except that aquatic macrophytes are intended to replace algae (Crites et al., 1988) and pollutant removal may be augmented by animals in the system (Reed et al., 1988, Shijun and Jingsong, 1989).

The most familiar version of the technology is the shallow water hyacinth pond as illustrated in Figure 4.6.

Two types of floating plant systems have been significantly developed and are based primarily on water hyacinth (*Eichhornia crassipes*) and duckweed (*Lemna sp.*, *Spirodela sp.*, and *Wolffia sp.*) respectively (Reed et al., 1988, Crites et al., 1988, WPCF, 1990). Pennywort (*Hydrocotyle umbellata*), which is not a true floating plant, has also been suggested as having potential as it is more tolerant of cool climate (Crites et al., 1988).

Figure 4.6: Aquatic Plant Treatment Schematic



Submerged macrophytes have also been suggested as having potential but their use is restricted by sensitivity to being shaded out by water turbidity (incoming suspended solids and algae that develop in the pond) and to anaerobic conditions (Crites et al., 1988). Plants such as water weed (*Elodea sp.*), pondweed (*Potamogeton sp.*), milfoil (*Myriophyllum heterophyllum*) and coontail (*Ceretophyllum demersum*) may work for polishing nutrient removal from treated wastewater but pilot test reports have not been very positive (Reed et al., 1988). Observations of Calgary area wetlands and wet ponds suggests that submerged macrophytes are components of the systems but whether sufficient plant density could be sustained throughout the wastewater treating season (growing season) to improve treatment beyond that of simple facultative ponds is not known. The referenced literature suggests that submerged plant based systems will probably have lower potential than emergent plant based WWT systems.

Water Hyacinth Systems: Water treatment in hyacinth ponds follows concepts discussed for facultative ponds but is augmented by the presence of the plants. The dense leaf canopy limits algae growth in the water and the root structure in the water column provides surfaces for biofilms which sorb and decompose wastewater constituents. Notably the plant cover reduces pond turbulence and improves settling relative to open water (Reed et al., 1988). Plant uptake of nutrients, metals and organics is significant in hyacinth systems (Shijun and Jingsong, 1989).

Design factors are shown in Table 4.8 and preliminary estimates of project area for hyacinth treatment systems can be estimated by the following (WPCF, 1990):

Secondary Treatment

$$A_{fh} = 0.0095Q$$

where A_{fh} = area for facultative hyacinth pond (ha)
 Q = design daily wastewater rate (m^3/day)
 Organic loading < 30 kg/ha·day
 50 day HRT, Temperature > 10°C

Tertiary Treatment

$$A_{ht} = 0.00071Q$$

where A_{ht} = area for tertiary hyacinth pond (ha)
 Organic loading < 50 kg/ha·day
 6 day HRT, Temperature > 10°C

Major problems with hyacinth systems are the plant's vulnerability to cold (they thrive between 20 and 30 °C and are intolerant of freezing) and mosquitoes that can reproduce in open water between the leaves (Crites et al., 1988). The technology, unless built in heated greenhouses, will not be applicable to Canadian conditions. However, understanding of the basic design concepts may have value if local analogs for the hyacinth can be found.

Table 4.8: Water Hyacinth Design Considerations
(after Crites et al., 1988, Reed et al., 1988 and WPCF, 1990)

BOD Removal	Secondary	Tertiary	Nitrogen Removal
Loading (kg/ha·d)	50	< 50	$N_e / N_o = e^{-kt}$
Depth (m)	< 1.5	< 0.9	or
HRT (days)	>40	6	$L_N = 760 / (1 - N_e/N_o)^{1.72}$
Influent	untreated	secondary	where N_e = effluent nitrogen (mg/l)
Temperature	>10°C	>20°C	N_o = influent nitrogen (mg/l)
No. of Cells	2 sets of 3	2 sets of 3	k = rate constant
Effluent BOD (mg/l)	< 30	< 10	(function of plt. density)
			= 0.218 - 0.590 (1/days) @ 27°C
			= 0.033 - 0.184 (1/days) @ 14°C
			t = residence time (days)
			L_N = hydraulic loading (m ³ /ha·day)
Phosphorus Removal			Operational Factors
$L_p = 9353(P_e - 0.778P_o) / (P_o - P_e)$			Plant harvesting monthly to annually
where L_p = hydraulic loading (m ³ /ha·day)			<u>Sludge clean out</u>
P_e = effluent phosphorus (mg/l)			primary cell annually
P_o = influent phosphorus (mg/l)			secondary cells 2 - 3 years
			seasonal systems annually

Duckweed Systems: Duckweed treatment processes improve water treatment largely through maintenance of an anaerobic water column and by controlling algae through restricting light penetration. The densely vegetated water surface and anaerobic water column also prevents survival of mosquito larvae (Crites et al., 1988, Reed et al., 1988, Buddhavarapu and Hancock, 1991). The cover of the pond surface and nutrient uptake are the primary roles of the plants in wastewater treatment (Buddhavarapu and Hancock, 1991). The plants provide minimal impact from biofilm activity due to the small and short roots (Reed et al., 1988).

Preliminary estimates of project area for duckweed systems can be calculated by (WPCF, 1990) with more detailed criteria present in Table 4.9:

$$A_{dw} = 0.002Q$$

where A_{dw} = area for duckweed polishing project (ha)
 Q = design daily flow rate (m³/day)

Reed et al. (1988) suggest facultative pond design correlations be used with nitrogen removal modeled by:

$$N_e / N_o = e^{-kt}$$

where N_e = effluent nitrogen (mg/l)
 N_o = influent nitrogen (mg/l)
 k = rate constant which is a function of plt. density (1/days)
 = 0.074 for Lemna at 73 kg/ha (dry wt.) and 27°C
 = 0.011 for Lemna at 131 kg/ha (dry wt.) and 27°C
 = 0.028 for Lemna at 40 kg/ha (dry wt.) and 14°C
 = 0.012 for Lemna at 67 kg/ha (dry wt.) and 14°C
 t = residence time (days)

Table 4.9: Design Parameters for Duckweed Ponds
 (after Reed et al., 1988 and WPCF, 1990)

Design Parameter	Value	
Type of wastewater influent	facultative pond effluent	
BOD ₅ loading (kg/ha·day)	22 - 28	
Hydraulic loading (m ³ /ha·day)	22 - 28	
Water depth (m)	1.5 - 2.0	
HRT (days)	20 - 25	
Water temperature (°C)	> 7	
Configuration for plug flow (L:W)	15:1	
	For Secondary Treatment	For Nutrient Removal
Harvest schedule	monthly	weekly
Expected Effluent Quality		
BOD ₅ (mg/l)	< 30	< 10
Suspended Solids (mg/l)	< 30	< 10
Nitrogen (mg/l)	< 15	< 5
Phosphorus (mg/l)	< 6	< 1 - 2

Duckweed systems are constructed according to procedures for facultative stabilization ponds. Some form of grid plus ongoing labor is required to ensure the plants evenly cover the pond surface. The systems are more tolerant of cold weather, working down to 7°C. Below that temperature the plants sink (an over wintering adaptation) and are not necessarily destroyed by freezing (Buddhavarapu and Hancock, 1991). The plants

are common in Calgary area wetlands and a successful system has been demonstrated for tertiary sewage treatment as far north as Devils Lake, North Dakota.

The North Dakota project reported that the harvested duckweed accounted for 73% of phosphorus removed. Phosphorus was reduced from 4 mg/l to 0.1 mg/l and BOD from 40-60 mg/l to 3 mg/l during a 47 day HRT (Buddhavarapu and Hancock, 1991).

4.4 Wetlands Wastewater Treatment Systems

4.4.1 WWT Overview

Wetland systems for wastewater treatment not only provide effective, low cost pollutant removal but also enable development of aesthetic and wildlife benefits. Wetlands based pollution technology reflects a "conceptual change" from wastewater as a liability to using water pollution control as an opportunity for the production of useful resources (Crites et al., 1988).

The physical, biological and biogeochemical processes discussed in Section 2 enable wetlands to be natural nutrient recycles and water purifiers. In their natural state, wetlands can act as effective pollutant and nutrient removal systems. A Canadian definition of wetland, however, was suggested as (Tarnocai, 1979 as quoted in Mitsch and Gosselink, 1993):

Wetland is defined as land having the water table at, near or above the land surface or which is saturated for a long enough period to promote wetland or aquatic processes as indicated by hydric soils, hydrophytic vegetation and various kinds of biological activity which are adapted to the wet environment.

This definition suggests that a wide range of natural water treatment processes are wetlands related. OF and lagoon processes, in particular, are closely related to the WWT systems that will be discussed in this subsection. The integration of pools and meadows with marshes for specific applications is discussed in Section 5.

Both natural and artificial wetlands have been used for wastewater treatment, however, specifically designed and constructed systems face fewer regulatory constraints and typically perform better than natural areas of the same size (Reed et al., 1988). Depending on the extent and purpose of human intervention wetlands can be referred to as:

- **Natural** wetlands that have not been altered by human intervention.
- **Restored** or reclaimed wetlands developed on sites where natural wetlands once existed.
- **Created** wetlands on sites where wetlands did not exist and which often have the intent of replacing natural wetlands lost elsewhere.
- **Constructed** wetlands with the primary intent of managing water quality of stormwater, sewage or other wastewaters.

Further discussion in this reports centers on constructed wetlands for wastewater treatment. Two principle types of wetlands technology are currently in use and are described in the literature (Reed et al., 1988, Crites et al., 1988, WPCF, 1990, Reed, 1991):

- **Free water surface (FWS)** systems where wastewater flows through shallow beds (15 - 30 cm deep) of emergent plants. These can take a wide variety of forms from single marsh-pond-marsh designs to systems comprising many long narrow vegetated trenches.
- **Vegetated submerged bed (VSB)**, subsurface flow system (SFS) or root zone method (RZM) where water flows through the porous rooting substrate of emergent wetland plants but is not permitted to flow above grade (e.g., no free water surface or overland flow). The beds can be laid out as single wetlands or set up in parallel trenches.

Wastewater has also been successfully treated in natural free water surface wetlands and in peatland systems where wastewater flows through peat bogs that adsorb contaminants. Peat systems may have features of FWS and VSB systems but depend on

peat substrates for adsorption, filtration and ion exchange of contaminants. Peatland systems maybe originally forested but the literature suggests that cattails and rushes will replace willows, swamp birch and spruce once eutrophic wastewaters are introduced (Kadlec and Alvord, 1989).

Rough estimates of project areas for FWS and VSB systems are provided below (WPCF, 1990) and more detail procedures are listed in the following subsections:

$$A_{fws} = 0.0082Q$$

and

$$A_{vsb} = 0.0027Q$$

where A_{fws} = project area for FWS wetland (ha)
 A_{vsb} = project area for VSB wetland (ha)
 Q = design daily flowrate (m³/day)

This section will provide design relationships for FWS and VSB systems and briefly discuss natural wetland and peatland concepts.

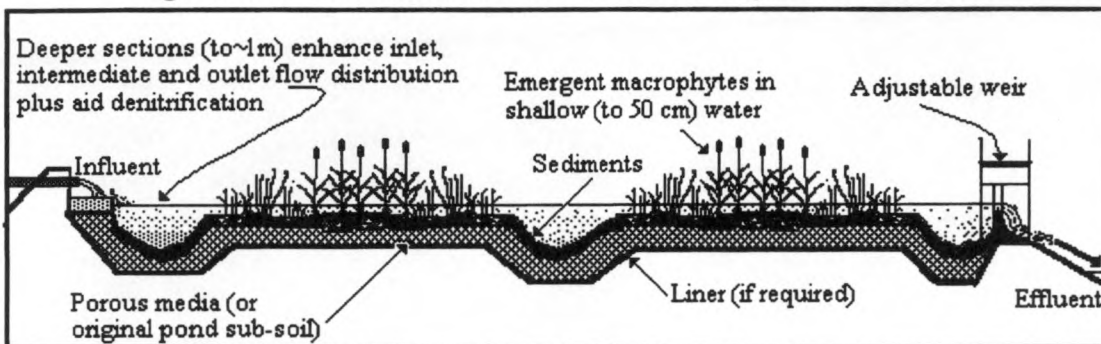
4.4.2 Free Water Surface WWT

FWS systems augment lagoon contaminant removal processes by slowly flowing wastewater through stands of dense emergent vegetation. The biomass of the emergent plants provides surfaces for biofilms which adsorb and decompose pollutants (Reed, 1991) and the plants create quiescent conditions which aid sedimentation and filtration (Watson et al., 1989). A broad range of removal mechanisms, as listed in Table 2.1, are involved in FWS systems. In the US some 56 FWS system process 95 mgal/day of runoff and wastewaters (Reed, 1991) and in the order of 500 systems are in use to control mine drainage water quality (Cooper, Borek, Brickett and Edenborn, 1993). Figure 4.7 illustrates the concept.

Design of FWS systems can be based on contaminant loading factors, first order reaction kinetic correlations and application specific conditions. Sewage treatment applications are key drivers for the first two approaches. Design of stormwater wetlands, for example, relies more on hydrology and basin storage factors. The loading factor and

kinetic correlations will be discussed in this section with specific application designs described in Section 5.

Figure 4.7: Free Water Surface Treatment System Schematic



Much of the literature on FWS system design (Crites et al., 1988, Watson et al., 1989, WPCF, 1990) refers to or is taken from the findings of Reed et al. (1988). That reference (Reed et al., 1988) is the source for the relationships described in Table 4.10 unless otherwise noted. In addition to the correlations, design criteria for FWS systems have been discussed as summarized in Table 4.11.

The correlation for pathogen removal discussed in Section 4.3.1 can be used to estimate performance of FWS systems (Reed et al., 1988). Designs based on BOD removal should be sufficient for suspended solids removal, provided dense vegetation is maintained near the outlet to control algae growth (WPCF, 1990).

Phosphorus removal can occur through sedimentation, adsorption and vegetation uptake (provide the plant material is harvested) with 31% to 80% reported (WPCF, 1990). The level of phosphorus removal will depend on specific system chemistry and soil substrate. Where phosphorus removal occurs by adsorption, for example, effectiveness can be expected to decline as adsorption sites become saturated. Significant clay content with iron and aluminum will enhance phosphorus removal in wetland soils (Reed et al., 1988).

Table 4.10: FWS Design Correlations
(after Reed et al., 1988 except as noted)

BOD Removal $C_e / C_o = A \exp[-0.7K_T(A_v)^{1.75} LWdn / Q]$ where C_e = effluent BOD (mg/l) C_o = influent BOD (mg/l) A = fraction BOD not settled out in inlet zone (0.52 is used to estimate A) K_T = rate constant (1/days) $= 0.0057(1.1)^{(T-20)}$ T = water temperature (°C) (approx. average ambient or use pond temperature correlation) A_v = specific surface area for microbial activity (15.7 m²/m³ is used as an estimate) W = system width (m) L = system length (m) d = system depth (m) n = system porosity (fraction) (0.75 is used based on dye studies) Q = average hydraulic loading (m³/day) $= (Q_{in} + Q_{out}) / 2$	Nitrogen Removal (after WPCF, 1990) NH₃ $\ln(C_e) = [0.688 \ln(C_o)] + [0.655 \ln(HLR)] - 1.107$ $A = (0.01Q) / \exp[1.527 \ln(C_e) - 1.05 \ln(C_o) + 1.69]$ Total Nitrogen $C_e = (0.193C_o) + (1.55HLR) - 1.75$ $A = (0.01Q) / (0.645C_e - 0.125C_o + 1.129)$ where HLR = hydraulic loading (cm/day) = $0.01Q / A$ Q = flow rate (m³/day) C_e = effluent N or NH₃ (mg/l) C_o = influent N or NH₃ (mg/l) A = required treatment area (ha) The regression correlations do not contain temperature corrections, thus do not account for cold climate effects and should be used for preliminary estimates only.
Detention Time for BOD Removal $t = [\ln(C_o) - \ln(C_e) - \ln(A)] / 65K_T$ where t = HRT (days) other parameters as above based on n and A_v numerical values noted above	Area for Nitrogen Removal (after Hammer and Knight, 1992) $A = 1.8312C_oQ / 1160.6$ where variables and units are per above

Plants commonly used in FWS systems include cattail (*Typha sp.*), reed (*Phragmites communis*), rush (*Juncus sp.*), bulrush (*Scirpus sp.*) and sedge (*Carex sp.*) all of which are viable throughout the world (Reed et al., 1988). Cattail, bulrush, rush and sedge are common to Calgary area wetlands. Systems such as the cold climate FWS system for Minot, North Dakota (the systems serves a population of 35,000) have been developed with sequential pond (60 cm depth) and marsh (15 cm and 30 cm depths) components. The 15 cm deep zones use cattail (*Typha latifolia*), the 30 cm deep sections are planted with bulrush (*Scirpus validus*) and the 60 cm deep sections with the

submerged macrophytes pondweed (*Potamogeton pectinatus*) and *Vallisneria americana* (Hammer and Knight, 1992).

Table 4.11: FWS Design Criteria
(factors based on sewage treatment)

Factor	Recommended Value	Reference
Organic loading (kg BOD / ha·day)	< 112	Reed et al., 1988
	100 - 110	WPCF, 1990
	< 100	Hammer, 1990
Nitrogen Loading (kg / ha·day)	TN < 60	WPCF, 1990
	NH ₃ < 3 - 5	Hammer and Knight, 1992
Aspect Ratio L:W	parallel channels of > 10:1	Herskowitz, 1986
	> 2:1	WPCF, 1990
	4:1 (with redistrib. per Fig. 4.7)	Hammer and Knight, 1992
HRT (days)	7 - 14	Herskowitz, 1986
	5 - 10	WPCF, 1990
Hydraulic Loading (cm/day)	2.5 - 5	WPCF, 1990
Configuration	multiple cells in series and parallel	WPCF, 1990
Water Depth (cm)	<10 cm (warm months)	Reed et al., 1988
	<45 cm (cool months)	
	< 50	WPCF, 1990

Hammer and Knight (1992) observed that the high L:W ratios like those suggested by the Listowel, Ontario study (Herskowitz, 1986) to control short circuiting can be reduced to 4:1 by the pond-marsh-pond design illustrated in Figure 4.7. The deeper zones redistribute flow and provide for nitrification/denitrification processes. Notably the 4:1 (versus 10:1 or greater) aspect ratio greatly reduces berm lengths and system costs (Hammer and Knight, 1992).

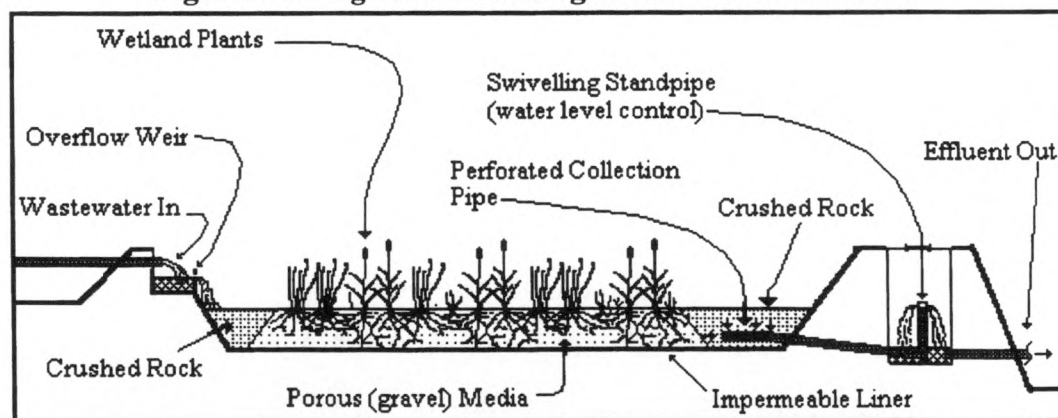
The capital cost (Lakshman, 1983) of the wetland treatment at \$0.59/m³ per day compared favorably for costs of conventional tertiary treatment (\$0.97/m³ per day) for small town installations (1983 dollars). Operating costs of conventional tertiary treatment were estimated to be three to eight times the cost of the wetland (@ \$0.04/m³, 1983 \$).

Reed (1991) reported costs of FWS systems to range between \$2,000 and \$65,000 per acre with a mean of \$22,000/acre. FWS systems are typically twice as large as VSB beds (of the same hydraulic and contaminant loading) but are 75% cheaper to develop on a unit area basis (Reed, 1991). FWS are the more common approach to wetlands treatment and are reported as less prone to the fouling that is a major problem with VSB processes.

4.4.3 Vegetated Submerged Bed WWT

VSB systems route wastewater through a porous media that supports emergent wetland plants. Contaminants are removed through media filtration of solids, microbial oxidation, plant uptake, particulate sedimentation and sorption of dissolved species on media and biofilms as described in Table 2.1. VSB systems offer several potential advantages over lagoons, OF, SR, RI and FWS systems because flow is below media surface. This feature limits risk of odors, public exposure and insect vectors while greater “active” media surface area minimizes land requirements relative to FWS wetlands (Reed, 1993). Well designed systems are reported as capable of reducing nitrogen species by 90% (Rogers, Breen and Chick, 1991) and BOD by up to 96% (Conley, Dick and Lion, 1991). The process is illustrated in Figure 4.8:

Figure 4.8: Vegetated Submerged Bed WWT Schematic



Optimum VSB systems have water levels controlled at the media surface with a downward flow from inlet to effluent collection pipes (Rogers et al., 1991). This ensures contact between the contaminants and active features (adsorption surfaces, microbes and plant roots) of the system and eliminates the possibility of water bypassing directly to the

effluent. This design also restricts growth of algae (which creates turbidity, suspended solids and BOD in effluent), controls odors and prevents mosquito problems.

Emergent hydrophytes of the *Scirpus* (bulrush), *Typha* (cattail) and *Phragmites* (reed) genus are reported as the principle plants of constructed VSB systems (Conley et al., 1991). Ideal emergent plants should have extensive roots and rhizomes to facilitate rapid propagation and large root surface area for contaminant sorption, decomposition and absorption. The literature (Conley et al., 1991) suggests that some plants may have toxic reactions to certain pollutants (relatively few instances noted), thus, the system designer should conduct appropriate tests with actual wastewaters as part of the design process. Wetland plants are vulnerable to excessive water velocities which should be minimized by adequate bed sizing and care in the layout of influent diffusion systems so that channeling of water in the treatment bed does not occur.

Although most VSB systems have been designed as secondary or tertiary sewage processes, some have been used to treat primary wastewaters including storm runoff (Esry and Cairns, 1989). Some control of suspended non-volatile influent solids is advisable as VSB beds will fail when the media pore volume in the first metre of depth becomes plugged (Conley et al., 1991). Root growth and decay work to maintain porosity by opening up fines clogged hydraulic pathways and VSB systems are anticipated to have useful lives similar to conventional treatment plants when bulk inlet sediments are controlled (estimated in the order of 100 years by Conley et al., 1991).

Laboratory material balance studies (Rogers et al., 1991) show that a substantial portion of the pollutant nitrogen feed is incorporated into the biotic material of the hydrophytes and indicates similar incorporation for phosphorus. Species such as *Scirpus validus* are known (Rogers et al., 1991) to transport oxygen to their roots and, as a result, oxygenate root zone areas. The available oxygen and high surface area for biofilms supports bacteria which consume and break down or precipitate nitrogen, phosphorus and BOD.

In the US EPA study by Reed (1993), VSB systems were found to reduce BOD and suspended solids below 20 mg/l with first order plug flow relationships providing

reasonable estimation of performance for up to two days HRT. The study, however, reported that nitrogen, ammonia and phosphorus removal of most systems was limited. Only where roots from dense stands of *Typha* or *Scirpus* fully penetrated the full depth of the VSB was ammonia removal effective. Similar results were found for total nitrogen. The only system found to be effective in phosphorus removal was constructed of aluminum and iron oxide rich gravel, thus, phosphorus removal was thought to be more a function of precipitation with the metals than a function of the wetland. In such cases the wetland will have a finite capacity for phosphorus removal.

Design of VSB systems rely on first order kinetics for BOD removal but must also evaluate system hydraulics and oxygen transfer. Design correlations presented in the literature (Crites et al., 1988, Reed et al., 1988, WPCF, 1990, Conley et al., 1991, Reed, 1993) are all fairly common. The relationships are summarized in Table 4.12. The information in Table 4.12 can be applied through the following design steps (Reed, 1993):

1. Define the vegetation, bed depth and media to be used. Determine porosity and hydraulic conductivity of the media (field tests or laboratory procedures). Assume an initial bed temperature (20°C summer and 6°C winter is suggested for small systems).
2. Estimate required surface area (A_s) from the equation in Table 4.12.
3. With consideration of site topography and geometry, select an initial aspect or length to width (L:W) ratio within the range noted in the table. Calculate length and width based on the estimate of A_s .
4. Calculate the maximum flow possible through the bed using the Darcy equation in Table 4.12 using “effective” gradient and conductivity (e.g., 0.1S and $k_s/3$).
5. If the calculated flow is less than design, revise L:W and repeat steps 3 and 4.
6. Evaluate system energy balance and heat loss (if a cold climate application) to assess validity of original bed temperature assumptions.

Repeat calculations as needed to reach a converged solution. Verify design with factors listed in Table 4.14.

Most operating VSB systems are not effective for ammonia removal which occurs through biological nitrification followed by denitrification (Reed, 1993). Oxygen necessary for conversion of ammonia to nitrate is supplied by plant transfer to roots. Thus ammonia conversion can only occur in sections of the VSB with fully developed root systems and in many of the systems reviewed by Reed (1993) the roots did not fully penetrate the full bed depth. Calculations for ammonia removal can be based on 7.5 g/m³·d oxygen transfer to fully developed root zones and 5 mg oxygen per 1 mg of ammonia (Reed, 1993). The reference suggests that BOD₅ must be reduced to about 20 mg/l before nitrification will start. The proposed procedure for estimating treatment area for ammonia removal is discussed below. Reed (1993) notes that the procedure is supported by limited research information and should be used with caution.

1. Calculate residence time (HRT) and wetland area required to reduce influent BOD₅ to 20 mg/l based on formulas given in Table 4.12.
2. Calculate oxygen availability per unit surface area based on expected depth of developed root zone:

$$O_a = d_r (7500 \text{ mg/m}^3 \cdot \text{d})$$

where O_a = available oxygen (mg/m²)
 d_r = root zone depth (m)

3. Define oxygen requirements:

$$O_r = 5000 Q_i (C_o - C_e)$$

where O_r = required oxygen (mg)
 Q_i = inlet flowrate (m³/d)
 C_o = influent NH₃ concentration (mg/l)
 C_e = desired effluent NH₃ concentration (mg/l)

4. Calculate area required for nitrification:

$$A_n = O_r / O_a$$

where A_n = required VSB area for nitrification (m²)

5. Add area determined in steps 1 and 4 to define total size of the VSB system.

Nitrogen removal calculations can also be estimated by the formula in Table 4.12, as well as by correlations presented for FWS systems (WPCF, 1990). Reed (1993) suggests that the correlations will result in areas at least double that estimated based on oxygen transfer. The FWS nitrogen correlations, however, may serve to provide order-of-magnitude estimates of ammonia removal in oxygen deficient VSB systems (Reed, 1993).

The oxygen based approach suggests that ammonia removal will be minimal following fall senescence in cold climate applications. This is further supported by the kinetic correlations which predict extreme area requirements for ammonia treatment as water temperature approaches freezing (the rate constant approaches zero). Thus, while the VSB systems are reported as effective for year-round use in cold climates, winter storage with summer treatment will be required if receiving water bodies are sensitive to ammonia inputs.

WPCF (1990) suggests that the aspect ratio (L:W) can be greater or less than 1:1 depending of flow distribution and treatment goals as much of the treatment occurs near the influent area. Berm cost minimization will favor a 1:1 aspect and the suggested maximum water velocity of 8.6 m/day will restrict high L:W ratios. Porosity and hydraulic conductivity should be evaluated for the specific media to be used, however, typical values are given in Table 4.13.

Table 4.12: Design Correlations for VSB Systems

(after Crites et al., 1988, Reed et al., 1988, WPCF, 1990, Conley et al., 1991 and Reed, 1993)

Hydraulic Loading Flow is subsurface and follows Darcy's Law: $Q = k_s A_c S$ where Q = average flow (m^3/day) $= [(Q_{in} + Q_{out}) / 2]$ k_s = hydraulic conductivity ($m^3/m^2 \cdot day$) A_c = cross-sectional area (m^2) = dW W = bed width (m) d = substrate depth (m) S = hydraulic gradient (dH/dL) or slope (m/m) -gravel media ≤ 4 cm -maximum velocity of less than 8.6 m/day is necessary to avoid root disruption thus: $S = 8.6 / k_s$ -to provide safety factor, designs should be based on $1/3$ k_s and 10% of available hydraulic gradient.	Organic Loading First order kinetic relationship for BOD: $C_e / C_o = \exp[-K_T A_s dn / Q]$ where C_o = influent BOD (mg/l) C_e = effluent BOD (mg/l) ≥ 5 mg/l (practical limit of VSB systems) Q = average flow (m^3/day) A_s = surface area (m^2) d = bed depth (m) n = bed porosity (as a fraction) K_T = rate constant (1/days) $= K_{20} (\theta)^{(T-20)}$ K_{20} = first order plug-flow rate constant $= 1.104$ per Reed (1993) ← suggested $= 0.7$ per Conley et al. (1991) $\theta = 1.06$ T = water temperature ($^{\circ}C$) note: $A_s dn / Q = t = \text{HRT (days)}$						
Aspect Ratio & Slope Aspect - $0.4: < L:W < 3:1$ Slope - Flat to slightly sloping base with adjustable outlet to control water level over entire bed depth.							
Ammonia Removal (Bavor et al. as cited in Reed, 1993) $A_s = [Q(\ln(C_o/C_e))] / (K_T dn)$ where C_o = influent NH_3 (mg/l) C_e = effluent NH_3 (mg/l) K_T = rate constant (1/days) $= K_{20} (1.03)^{(T-20)}$ $K_{20} = 0.107 d^{-1}$ other variables as above	Required Surface Area $A_s = Q[\ln(C_o/C_e)] / (K_T dn)$ variables and units per BOD relationship above Design Depth design depth is a function of rooting characteristics of selected vegetation with 60 cm the most common depth used, <table> <tr> <td>Typha</td> <td>$d = 30$ cm</td> </tr> <tr> <td>Scirpus</td> <td>$d = 76$ cm</td> </tr> <tr> <td>Phragmites</td> <td>$d = 60$ cm</td> </tr> </table>	Typha	$d = 30$ cm	Scirpus	$d = 76$ cm	Phragmites	$d = 60$ cm
Typha	$d = 30$ cm						
Scirpus	$d = 76$ cm						
Phragmites	$d = 60$ cm						

Table 4.13: Porosity and Hydraulic Conductivity Values
(after Reed, 1993)

Material	Effective Size (D₁₀) mm	Porosity (n)	Hydraulic Conductivity (k_s) m/d
Coarse Sand	2	32	1,000
Gravelly Sand	8	35	5,000
Fine Gravel	16	38	7,500
Medium Gravel	32	40	10,000
Coarse Rock	128	45	100,000

Although the correlations listed in Table 4.12 are the preferred design approach certain design factors have been suggested. The factors listed in Table 4.14 can be used for screening estimates and to verify whether designs predicted by the correlations are reasonable.

The literature (Conley et al., 1991, Reed, 1993) supports use of a porous gravel substrate for the aquatic plants. Systems have been constructed with native soils as the media but the reduced hydraulic conductivity impacts performance or requires larger treatment areas than gravel beds for a given water volume or pollution loading. Soil based systems appear to be chosen to minimize construction costs, particularly where gravel is scarce and expensive or where the treatment system has been designed to utilize an existing wetland with only minor modification.

Table 4.14: VSB Design Considerations
(after WPCF, 1990)

Factor	Recommended Value
Organic loading (kg BOD / ha·day)	80 - 120
Nitrogen Loading (kg / ha·day)	TN < 60
HRT (days)	5 - 10
Hydraulic Loading (cm/day)	6 - 8
Configuration	multiple cells in parallel
Water Depth (cm)	30 - 90
Min. Size (ha/1000 m ³ /d)	1.2 - 1.7

VSB processes depend on a balance of large adsorption surface area and the filtering effect of the rooting media for pollutant removal with adequate permeability to

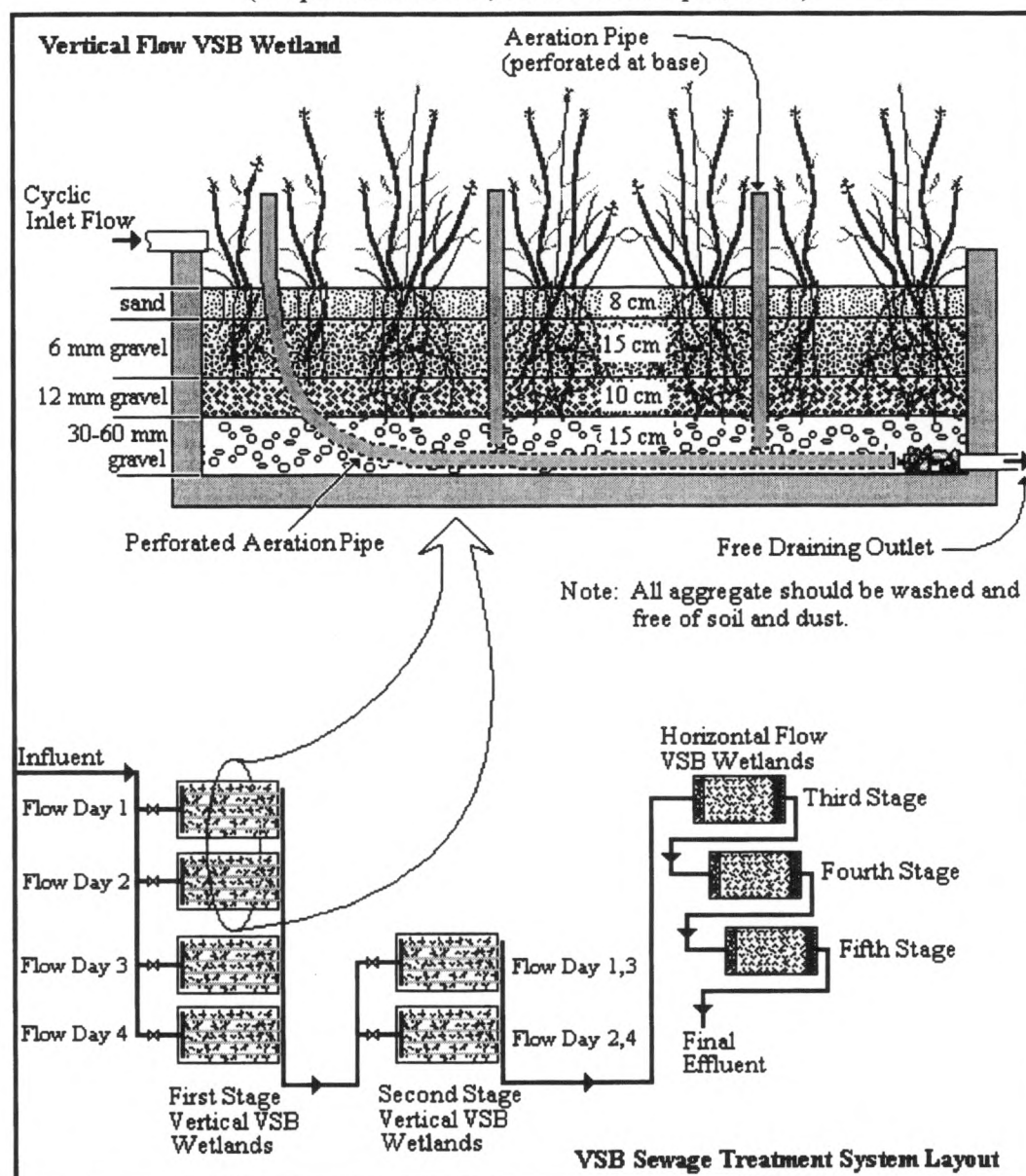
sustain subsurface flow. Finer soils and clays afford more relative surface area for adsorption and tighter matrix for better filtration. Conversely, high permeability of the media is desired to improve hydraulic loading. Coarse media are able to handle a wider range of flows than fine soils and are less prone to over-surface flow. In practice, most soil based systems are unable to maintain sufficient permeability and report poor performance because of overland flow or root zone bypassing (Conley et al., 1991).

Reed (1993) reports that several failures (loss of substrate permeability with resulting overland flow) of early VSB systems may be attributable to construction practices rather than inherent flaws in the concept (e.g., plugging from wastewater constituents). It is essential that clean media be used and that trucks and construction equipment not contaminate the media with dust or soil.

Provision should be made to raise and lower water levels in the VSB system (adjustable level outlet pipe). Short duration flooding of the surface is a technique employed to control weeds that develop on the substrate. Gradually lowering the water level in the fall can be used to encourage deeper penetration of roots. The annual level reductions are reported as necessary for at least three years (in the case of *Phragmites*) to fully develop deep root zones and may be necessary through out the life of the project (Reed, 1993).

An alternative concept closely related to horizontal flow VSB involves sequenced vertical down flow through porous beds planted with wetland plants (usually *Phragmites australis*). Systems consist of several beds in parallel that are alternately flooded and drained resulting in alternating reducing and oxidizing conditions in the bed (Brix, 1993). This greatly enhances nitrification because the draining cycle results in oxygenation of soil pore space, countering problems noted with oxygen transfer in continuous horizontal systems. The concept is illustrated in Figure 4.9.

Figure 4.9: Vertical Flow VSB System
(adapted from Brix, 1993 and Cooper, 1993)

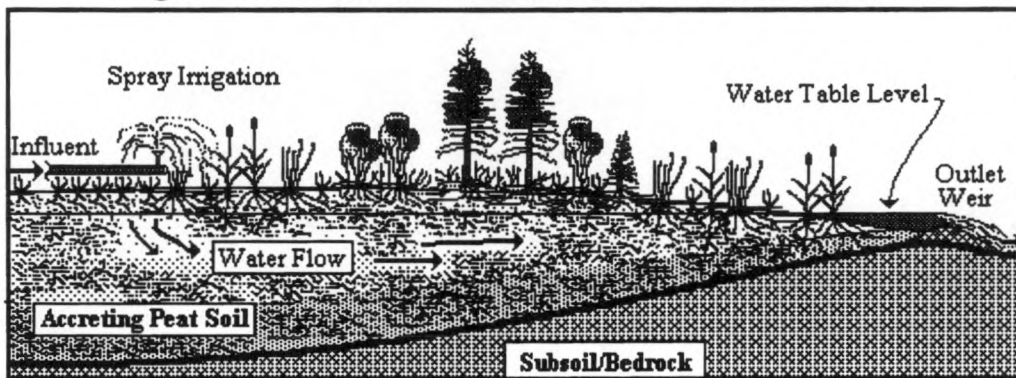


A survey of some 98 VSB systems in the US (which process 16.5 mgal/d) indicates VSB development costs range from \$72,000/acre to \$100,000/acre (Reed, 1991). The mean of \$87,000/acre is high compared to the average FWS system cost of \$22,000/acre. Unit system costs per m^3 of wastewater capacity of \$163/ m^3 for VSB and \$206/ m^3 for FWS, however, reflect the higher treatment effectiveness of VSB systems (Reed, 1993).

4.4.4 Peatland and Natural Systems

Natural wetlands systems have been used successfully for wastewater management. Such systems vary from relatively large prairie pothole wetlands such as Frank Lake, Alberta (Haworth-Brockman and Smallwood, 1990, Usher, 1993, pers. comm.) to systems with considerable peat content like Houghton Lake, Michigan (Kadlec and Alvord, 1989). Treatment in the systems may combine facets of lagoon, FWS and VSB processes. Figure 4.10 as well as Figure 4.7 illustrate natural WWT systems.

Figure 4.10: Peatland Wastewater Treatment Schematic



Peatland Systems: In the Northern US and Canada peat bogs represent a potential treatment resource. Peatland treatment of mining wastes has been used in Saskatchewan (Bres, 1992). A peat wetland has been in use for eleven years in Michigan (Kadlec and Alvord, 1989) to treat secondary sewage water. The performance of the Houghton Lake, Michigan system is impressive in that, during the May to September growing season, it processes 10,000 m³/day of secondary sewage. After passing through an area of only 60 hectares (10% of the area of the natural peatland system in which the project was located) water quality is restored to background levels for the wetland.

Peatland systems are typically natural features into which storm runoff or pretreated sewage is flood or spray irrigated as illustrated in Figure 4.10. Mechanisms of nutrient removal in peatlands are similar to those discussed for the VSB process. The Michigan study reported a 98% reduction in dissolved organic nitrogen and a 97% reduction in total phosphorus. These nutrients are incorporated into new biomass, sorbed

onto and constituent of slowly decaying sediment detritus and decomposed into gases and water.

With proper design, peatlands can be established to minimize free water surface. This feature maximizes contact of the wastewater with adsorptive surfaces of the peat and emergent plant roots. The high surface area of the peat substrate promotes effective, adsorption of pollutants and decomposition of some contaminants, as well as their incorporation into biomass and accreting soils.

A significant feature of peatlands is their water storage capacity (Heliotis, 1989). Under normal climatic conditions the water table exists at some level below the peat surface. Because of deep bed depth and porosity of the material, ample flow area exists below the water table level for suitable hydraulics without overland flow of the influent stream. The height of the water table below the surface of the peatland then permits surge retention of influent fluctuations potentially to a greater extent than may be feasible to design into a VSB system. However, as peatland systems have relatively larger area and are often integrated into regional drainage patterns, seasonal hydrological cycles and precipitation loading factors must be incorporated into the design to avoid flooding and overland flow of the wastewater.

The sustainability of the treatment system depends to some degree on the rate that new adsorption sites can be generated (accreting peat generated from living biomass) in relationship to the contaminant loadings on the system. Although apparently successful, the Houghton Lake project is forecast to have a useful life of about 30 years for phosphorus removal (Girts and Knight, 1989). Studies reveal moving fronts of ammonia and phosphorus within the peatland which indicate influent loadings exceed the long-term assimilation capacity of the fen.

In addition to concerns about the long term sustainability of the process, issues arise concerning the impact of wastewater on the ecology of natural peatlands. The Michigan study (Kadlec and Alvord, 1989) reported that above ground biomass quadrupled in the affected zone and shifted in species composition from sedges (*Carex*) to *Typha* in response to nutrient enrichment from the wastewater. Wetland shrubs, *Betula*

pumila and *Chamaedeaphne calyculata* increased in biomass three fold, however, root biomass decreased with consequential mortality. At the same time accretion of peatlands soils increased from 2 or 3 mm/year to 10 mm/year. The Saskatchewan study (Bres, 1992) also reported initial adverse impacts on larger plants from spray irrigation of mining wastewater. Girts and Knight (1989) questioned the enrichment (and consequent alteration) of a natural peatland for only a finite period of treatment. As will be noted for natural WWT systems, the potential negative impacts of wastewater on natural ecosystems such as peatlands warrant careful evaluation of desirability of such approaches.

Natural WWT Systems: Kent (1987) reports that natural wetlands treatment of septic tank effluent from a school near St. Paul, Alberta reduced BOD by 82%, nitrogen by 85% and phosphorus by 90% during summer months. Notably the WWT system has been in use for over 40 years and involves a wetland with no surface outflow. The wetland is part of the Blue Quills National Wildlife Area.

Design factors are summarized in Table 4.15. Design of natural wetland WWT systems requires evaluation of soils, hydrology and topography (WPCF, 1990 suggests surface surveys with contour intervals of 3 cm). HRT can be estimated by (WPCF, 1990):

$$t = \phi V / Q$$

where t	= hydraulic residence time (days)
ϕ	= void fraction or porosity
V	= wetland volume at design stage (m ³)
Q	= average flow rate (m ³ /d)

Negative impacts or significant changes in at least parts of the wetlands are reported as a result of wastewater introduction (Kent, 1987, Kadlec and Alvord, 1989) and design manuals recognize issues related to such use of natural wetlands (WPCF, 1990). Introduction of wastewater alters the hydrology of the wetland, changes the nutrient status and may introduce toxins. These factors combine to alter the vegetation of the system and consequently the fauna in ways that are not necessarily predictable or desirable. Constructed wetlands are usually considered part of wastewater systems and

are regulated in terms of effluent quality. Discharges to natural wetlands, however, are likely subject to quality regulations which may require a higher level of pretreatment than for constructed wetland processes.

Table 4.15: Design Considerations for Natural Wetland Systems
(after WPCF, 1990)

Parameter	Design Consideration
Minimum size (ha/ 1000 m ³ /d)	5 - 10
Maximum water depth (m)	0.5 (related to vegetation tolerance)
Aspect ratio (L:W)	2:1
Minimum HRT (days)	14
Maximum hydraulic loading rate (cm/day)	1 - 2
Configuration	multiple influent discharge sites
Maximum loading (kg / ha-day)	
BOD	4
Total Nitrogen	3

Treatment wetlands developed with Ducks Unlimited in Alberta typically use treated sewage to augment the hydrology of created, restored or natural wetlands in non-discharging systems (Haworth-Brockman and Smallwood, 1990, Usher, 1993, pers. comm.). **It is the author's opinion that natural wetlands should be used with great caution and only after adequate experience with constructed systems has been developed.** This conclusion is based on the author's literature review and the complexity of assessing hydrology of natural wetlands let alone forecasting potential wastewater impacts.

5. POTENTIAL APPLICATIONS FOR WWT

Wetlands and related natural wastewater treatment systems have been demonstrated for a variety of significant applications. The design correlations discussed in Section 4 were derived from sewage treatment applications which are probably the most developed WWT use. This section will discuss potential applications of wetlands technology and describe relevant application specific design criteria.

The following subsections will describe applications, briefly discuss relevant operating sites, summarize performance, detail application specific design consideration and, where information exists, provide data on capital and operating costs. Applications reviewed include:

- sewage treatment
- agricultural wastewater and runoff
- industrial wastewater
- landfill leachate
- acid mine drainage
- urban runoff

As discussed in Section 1.4 problems associated with urban runoff are significant and water quality treatment is not typically incorporated into Alberta urban stormwater systems. The use of wetlands ecotechnologies for controlling non-point source pollution from urban, agricultural and industrial lands is expected to be the most important application in Western Canada. For this reason wetland applications for stormwater management will be discussed in some detail in Section 5.5.

5.1 Sewage Treatment Wetlands

WWT has application for secondary - tertiary treatment of municipal wastewater for smaller communities where lagoon or septic system effluents are adversely affecting receiving waterbodies. Constructed wetlands can be used to control organic (BOD),

suspended solid and nutrient contaminants in primary treated municipal wastewater or stabilization lagoon effluents. The design correlations for lagoon, OF, aquatic plant, VSB and FWS systems discussed in Section 4 have been developed primarily for sewage treatment applications. This subsection will discuss operations experience and performance expectations for sewage treatment WWT systems.

5.1.1 Municipal Sewage Applications

In a report for the US Environmental Protection Agency, Reed (1991) identified 56 FWS and 98 VSB wetlands in use for municipal wastewater. Capacities of the systems ranged up to 75,000 m³/day with average FWS system size of 4,000 m³/day and mean VSB capacity of 1,100 m³/day. The Minot, North Dakota FWS system provides final treatment for aerated lagoon effluent for a community of 35,000 (Hammer and Knight, 1992). The Minot system provides seasonal treatment at reported rates up to 29,000 m³/day with lagoon storage during winter months.

In Canada, significant research has been done on cold climate applications of the ecotechnology in Ontario (Herskowitz, 1986, Miller, 1989) and Saskatchewan (Lakshman, 1983). A natural wetland FWS system has been operating near St. Paul, Alberta for over 40 years (Kent, 1987). Non-discharging wetlands have been developed by Ducks Unlimited, notably at Frank Lake which receives treated wastewater from High River, Alberta. All of these projects have demonstrated that WWT is a viable technology for sewage treatment in our climate.

Sewage treatment systems should incorporate redundancy with several cells in series and parallel (WPCF, 1990). Systems tested in Listowel, Ontario (Herskowitz, 1986), Saskatchewan (Lakshman, 1983) and others reported by the US Environmental Protection Agency (Crites et al., 1988) used parallel trenches with high L:W ratios. Some provide for recycle or step feed (influent introduced along length). This type of design approximates a plug flow reactor and provides ready maintenance access along the length of each cell.

Hammer and Knight (1992) suggest marsh-pond-marsh systems with alternating shallow emergent plant zones of 15 - 30 cm and deeper pond zones of 60 cm or more. The arrangement supports nitrification - denitrification processes and the deeper sections redistribute flow to prevent bypassing of areas of the wetland. Such designs have an L:W of 4:1 or less which lowers construction costs relative to the high L:W trench approaches (berm costs are minimized when L:W is 1:1). The Minot, North Dakota system is designed with four parallel cells (177 m wide and 745 m long) to provide municipal wastewater polishing for aerated lagoon effluent (Hammer and Knight, 1992).

Hydrology of sewage treatment systems is usually controlled by diverting runoff away from the system and sealing the wetland bottom with impervious soil or a synthetic liner. The water balance is thus reduced to inflows, outflows, direct precipitation and evapotranspiration. High rates of evapotranspiration can result in high effluent concentrations despite absolute contaminant reduction in the wetland (Reed et al., 1988).

Reed et al. (1988) warns that management of algae formed in the storage lagoons is required as FWS wetlands may not be effective for its removal. The discussion in Section 5.2 for agricultural systems provides insight on algae control by wet meadows (an OF modification).

Hammer (1991) reports that WWT should be 1/10 to 1/2 as expensive to construct and operate as conventional systems. He reported the Benton, Kentucky WWT system to treat primary lagoon effluent cost \$260,000 (in 1986) compared to a \$2,500,000 estimate in 1972 for a conventional process. Secondary and tertiary WWT treatment for communities of 500 and 1000 ranged from \$212,000 to \$366,000 and operate for less than \$10,000/year (Hammer, 1991).

Land requirements, nuisance and safety concerns are issues with FWS sewage treatment applications. Even in semi-rural areas land can be expensive and scarce. The Minot, North Dakota (Hammer and Knight, 1992) and the Humbolt, Saskatchewan (Lakshman, 1983) FWS system designs require wastewater to be stored during cold months with design wetland feed rates sufficient to draw down storage during the ice-free season. The necessary lagoon presents a second source of odors, safety issues and land

requirements. It is possible that use of a year-round VSB system or sand filter to pretreat effluent entering the storage lagoon could reduce problems. It is questionable whether the winter storage lagoons could be fully eliminated in any application requiring control of effluent ammonia levels, however. Thermal balances would have to be carefully done and the concept piloted before wide scale application. As domestic septic drain systems typically do not freeze in prairie climates, it is postulated that sufficient heat from influent and from anaerobic decomposition could result in viable year-round VSB systems.

Design correlations for sewage treatment are given in Section 4. Appropriately designed sewage WWT systems can be expected to achieve performance presented in Table 5.1.

Table 5.1: Typical Performance of Sewage Treatment Wetlands

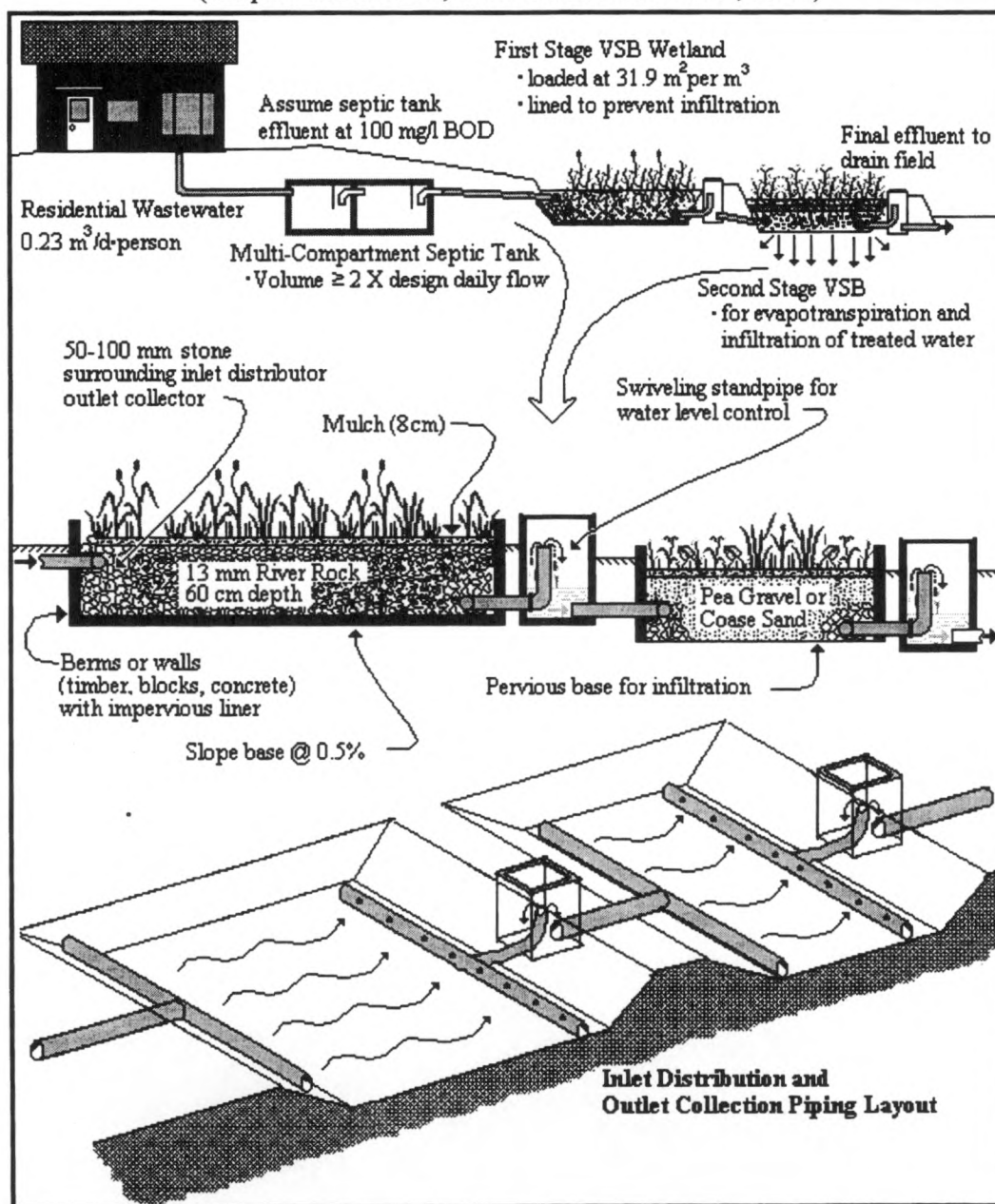
Contaminant	Removal Efficiency (%)	Comment	Reference
BOD	70 - 90	less for low influent BOD	WPCF, 1990
Ammonia	70 - 90	loading < 10 kg/ha day HRT 3 to 5 days	WPCF, 1990
Total Nitrogen	75 - 95	loading < 10 kg/ha day HRT > 5 days	WPCF, 1990
Phosphorus	30 - 50 0 - 90	variable, function of soil variable	WPCF, 1990 Watson et al., 1989
Metals (Cu, Zn, Cd)	> 97	one pilot project	Crites et al., 1988
(Fe)	median 96	study of 10 constr. WWT	Watson et al., 1989
(Mn)	median 83	study of 10 constr. WWT	Watson et al., 1989
Pathogens	82 - 100 90 - 99	California WWT studies	Watson et al., 1989 Gersberg et al., 1989
TSS	> 70% 60 - 90	HRT > 5 days for natural wetlands	WPCF, 1990 Crites et al., 1988

5.1.2 On-Site WWT Systems

In addition to systems for small communities, WWT designs have been developed for on-site treatment of residential wastewater (Reed, 1993). These designs typically

employ small VSB systems to treat septic tank effluent prior to release into watersheds or into poorly drained soils. Components of the system are illustrated in Figure 5.1.

Figure 5.1: On-Site Constructed Wetland for Domestic Wastewater
(adapted from Reed, 1993 and Steiner et al., 1993)



Design procedures have been published by the Tennessee Valley Authority (Steiner et al, 1993) and the US EPA (Reed, 1993) with the latter procedure involving

greater detail. The following design process is primarily based on the method of Reed 1993).

1. Initial Assumptions:

- Design flow of $0.23 \text{ m}^3/\text{d}\cdot\text{person}$
- BOD_5 in septic tank effluent $\geq 100 \text{ mg/l}$
 - range of concentration reported at 140-200 mg/l (Metcalf and Eddy, 1991)
- Nitrogen in septic tank effluent (Metcalf and Eddy, 1991):
 - total N 25-60 mg/l
 - ammonia 20-60 mg/l
- Design wetland effluent $\text{BOD}_5 \leq 10 \text{ mg/l}$.
- Design receiving water body total nitrogen near outfall $\leq 1.0 \text{ mg/l}$ (Alberta Environment, 1977). CCME recommends receiving waterbody maximum ammonia at 2.5 mg/l for 0°C (pH=7.0) and 1.49 mg/l for 20°C (pH=7.0) to protect aquatic life (the values decrease to 0.49 and 0.32 mg/l receptively at pH=8.5).
- Washed 1.25-2.5 cm gravel VSB media
 - hydraulic conductivity of $1500 \text{ m}^3/\text{m}^2\cdot\text{d}$
 - porosity of 0.38
- Summer temperature of 20°C and winter bed temperature of 6°C .

2. Determine VSB surface area using $K_{20} = 0.828 \text{ d}^{-1}$ (1.104 d^{-1} derated 25% as a safety factor) and the area equation in Table 4.12.

3. If an aspect ratio (L:W) of 2:1 or less is possible, Darcy's Law calculations should not be necessary for a 0.6 m deep VSB. For higher aspect ratios or shallower beds, the hydraulic loading should be evaluated with the Darcy's Law formula in Table 4.12. For flat

bottomed VSBs assume a hydraulic gradient of 0.25% for the calculations.

4. The design will treat for BOD but Reed (1993) advises nitrogen control (reduction of total N to 10 mg/l) will require doubling of HRT (to about 6 days in summer and 10 days in winter). Again the correlations in Table 4.12 can be employed for design checks. In the absence of receiving water body mixing calculations, designs have been based on total nitrogen in the effluent of 2-5 mg/l.
5. For northern prairie climates, heat loss calculations should be done to evaluate risk of winter freezing. Freeze protection of the level control chambers and additional mulching for insulation of the VSB may be required.
6. On sites with suitable percolation properties and where permitted, the wetland can be constructed in two sections (each with length $L/2$). The first bed should be lined (20-30 mil PVC or polyethylene sheet) to prevent infiltration. The second bed provides final treatment and percolation of treated wastewater to minimize surface discharge or load on drain fields.
7. Bed(s) should be constructed with a level to slightly sloping base and level surface. A 0.6-1.0 m section of washed stone (2.5-10 cm) should be placed around the 10 cm diameter inlet distribution and effluent collection manifolds. These manifolds should have evenly spaced 2.5 cm diameter holes. Clean, washed media must be used and protected with an 8 cm layer of mulch.
8. Local wetland plants (*Typha*, *Scirpus*) should be used in the first bed (Steiner et al., 1993), however, Reed (1993) recommends *Phragmites*. If a second infiltration cell is used, ornamental wetland species can be planted.

The design is appropriate for systems up to 40 m³/d and should provide good control of suspended solids and BOD provided adequate provision is made for winter conditions. It is expected that senescence of vegetation, snow cover and freezing of the upper surface of the wetland will limit cold weather oxygen transfer. This means that nitrification of ammonia will be minimal in winter. If ammonia or total nitrogen concentrations in effluent are a concern for protection of aquifers or surface waters, some means to store the effluent during the winter may be required.

5.2 Agricultural Runoff and Wastewater Treatment

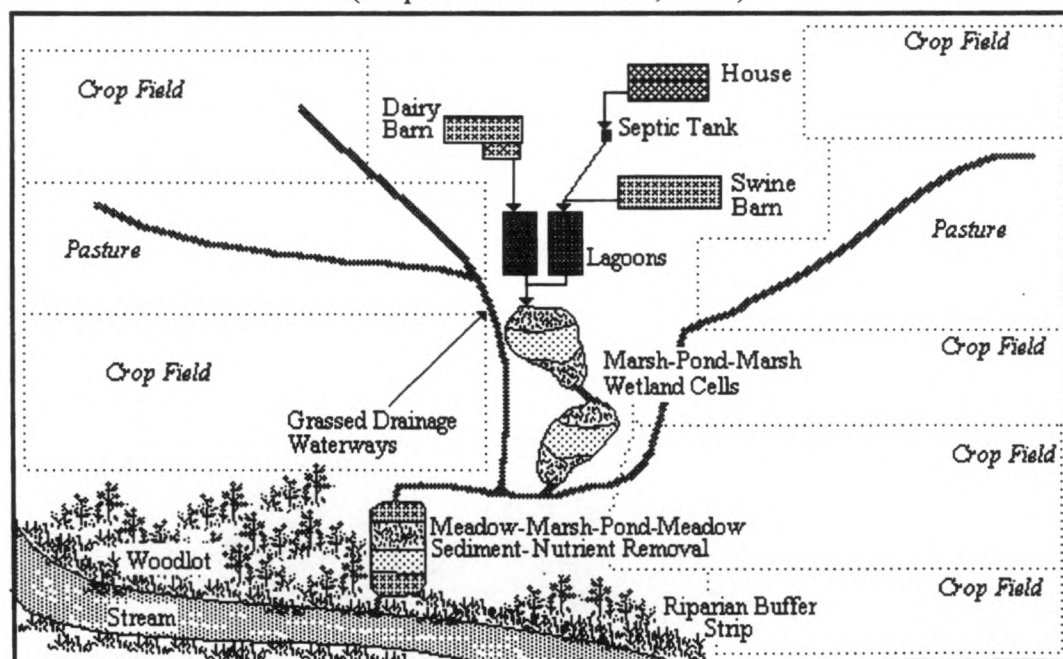
Nonpoint source (NPS) water pollution has become recognized as a major impediment to water quality improvement with agricultural practices related to tillage and animal wastes identified as key issues (various US EPA studies cited in Hammer, 1989). Trends to intense animal rearing practices concentrate wastes in small areas that are subject to deliberate or precipitation initiated flushing. These trends are in contrast to free range livestock practices that distribute wastes over larger areas increasing opportunity for on-site recycling and overland flow removal of materials entrained in runoff. Riparian vegetation, which filters runoff water, has been removed to increase agricultural acreage (Hammer et al., 1991). Crop production practices and destruction of natural buffers along water bodies can result in pollution by agri-chemicals and high sediment loads. Process complexity as well as capital and operating costs make conventional wastewater treatment technologies prohibitive for all but the largest agricultural operations (Hammer, 1989). WWT coupled with other natural processes present affordable means to control agricultural pollution.

Agricultural operations such as cattle feedlots, intensive swine operations and poultry farms can produce substantial amounts of highly contaminated water as illustrated in Table 5.2:

Table 5.2: Typical Agricultural Wastewater Composition

Contaminant		Swine Barn	Swine Lagoon	Treated Dairy	Raw Livestock	Municipal
		Lagoon	Effluent	Effluent	Wastewater	Wastewater
BOD	(mg/l)	110.8	2900	800 - 13,800	2000 - 4000	200 - 300
Total P	(mg/l)	48.8	122	3.5	75 - 150	5 - 20
Ammonia	(mg/l)	84	475	10.7	300 - 500	9 - 30
Fecal Colifoms	(#/100ml)	817,500	--	41,200	--	--
Fecal Strep.	(#/100ml)	118,750	--	23,700	--	--
Source		Hammer et al., 1991	Hammer, 1989	Costello, 1989	Hammer, 1989	Hammer, 1989

Agricultural water pollution can be reduced by restoration of vegetated buffers between fields and water bodies. The buffers act as filter strips (discussed in Section 5.5.2) and involve contaminant removal processes of OF treatment systems explained in Section 4.2.4. Routing of drainage within agricultural operations in grassed swales (also discussed in Section 5.5.2) and as sheet flow over vegetated areas will aid in pollutant removal.

Figure 5.2: Schematic of an Integrated Agricultural Wastewater System
(adapted from Hammer, 1991)

Marsh-pond-marsh FWS wetlands can be used not only to treat specific effluents (dairy or swine barn wastes) but also can be integrated into farm drainage networks. Figure 5.2 illustrates the concept of integrated agricultural water pollution control.

The riparian buffer strip and grassed drainage swales indicated in Figure 5.2 function similar to OF treatment. They assist in control of sediments draining from tilled areas and filtering of algae and solids not removed by marsh - pond units. Swale slope and flow width should prevent erosive velocities and should favor shallower over deeper water.

The livestock wastewater treatment wetland cell and the final wastewater - drainage system are composed of three elements, as described by Hammer (1991):

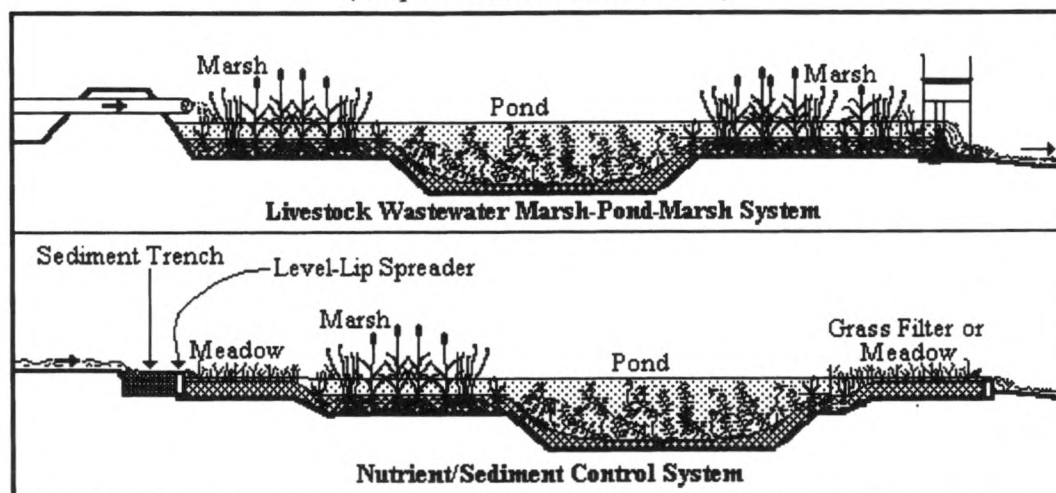
Pond: The pond sections aid BOD reduction and are significant for nitrification - denitrification. Water depth is suggested as 0.5 to 1 m with duckweed (*Lemna sp.*) on the surface, algae in the water column and submerged pondweeds, the later preferably with linear filiform leaves (*Potamogeton sp.*, *Ceratophyllum sp.*, *Elodea sp.*, *Vallisneria sp.*).

Marsh: The marsh treats BOD, suspended solids and pathogens but is not as effective as the pond or meadow in ammonia transformation. Dense stands of cattail (*Typha sp.*), bulrush (*Scirpus sp.*), reed (*Phragmites sp.*), rushes (*Juncus sp.*) and/or sedges (*Carex sp.*) in 10 to 20 cm of water are desired features.

Meadow: The meadow is modeled on OF concepts with reed canary grass (*Phalaris arundinacea*), sedges (*Carex sp.*) or other water tolerant grasses. In areas where significant dry periods are expected more tolerant grasses such as *Hordeum jubatum* could be considered (*Hordeum jubatum*, however, is undesirable and potentially harmful as animal forage). Flow is distributed across the width with sheet flow of 1 to 5 cm depth. In true OF mode flow would be alternated between at least two meadows to allow beds to dry and oxidize. The meadow component is important for nitrification/denitrification and removes suspended solids, especially algae that forms in the pond and FWS marsh.

The layout of the wetland components in the agricultural WWT systems is shown in Figure 5.3.

Figure 5.3: Cross-Section of Agricultural WWT Systems
(adapted from Hammer 1991)



A lagoon (or lagoons) should be provided for winter storage, initial reduction of BOD and phosphate and removal of gross solids. The lagoon should be designed to remove about half of the BOD and suspended solids (Hammer, 1991) using procedures described in Section 4.2.1. Livestock wastewater loading can be estimated from parameters in Tables 5.2 and 5.3 with a 10% - 20% allowance for waste hay and feed.

Table 5.3: Livestock Waste Production
(after Hammer, 1989)

	Animal Size (kg)	Waste Production (g/day)			Volume (m ³)
		BOD	Nitrogen	Phosphorus	
Dairy Cows	455	773	186	33	0.05
Swine	91	180	204	68	0.03
Poultry	1.8	6	1.3	0.5	0.0001

Hammer (1991) suggests that wetlands treatment areas should be sized so that BOD₅ loading is less than 100 kg/ha·day and ammonia loading is less than 10 kg/ha·day. However pilots for swine wastewater have been based on 3 kg ha·day (Hunt et al., 1993) and it has also been suggested that ammonia concentrations above 100 mg/l can be toxic to wetland plants (Payne, McCasky and Eason, 1992). Both the latter references suggest

dilution of influent as needed to avoid excessive wetland loading. Dilution can be accomplished by routing an upland base flow through the system or by recycling treated effluent. Correlations for FWS systems shown in Table 4.10 can be used for marsh sections and procedures discussed in Section 4.2.4 for OF processes can be used for estimating meadow performance. Hammer (1991) suggests design relationships for tertiary livestock wastewater treatment as listed in Table 5.4.

Table 5.4: Design Factors for Tertiary Livestock Wastewater Treatment
(after Hammer, 1991)

Component	BOD Loading (kg/ha day)	BOD Removal (%)	N Loading (kg/ha day)	N Removal (%)
Lagoon	--	design for 50 (2)	--	20
Marsh	100	to < 30 mg/l	10	30
Pond	-- (1)	--	40	60
Meadow	-- (1)	--	20 (3)	90

	Configuration	L:W
Lagoon	1 -2 in series/parallel	avoid extremes (4)
Marsh	min. 2 to provide for maintenance	3-5:1
Pond	1 is adequate	avoid extremes (4)
Meadow	one for continuous flow at least two for OF batch process	3-5:1

Notes:

1) Functionally both pond and meadow sections follow the marsh which removes most of the BOD.

2) Design per facultative lagoon procedures.

3) For meadows operated as OF systems (periodic drying cycle) loading can be 30 - 40 kg/ha day.

4) Low L:W results in wide systems that can be subject to short circuiting.

High L:W results in very high loading in inlet zones and potentially high velocities in the wetland.

The design factors listed in Table 5.4 are more than supported by examples in the literature. Table 5.5 summarizes results reported for a swine system in Alabama and a dairy operation in Ireland. The Alabama system was designed along the lines of the foregoing discussion, however, the Irish system utilized a peat wetland that was part of a lake bed. Both received primary treated wastewater (lagoon and anaerobic channels respectively).

Table 5.5: Agricultural WWT Contaminant Removal Effectiveness

	Swine Farm Effluent Treatment			Dairy Farm Effluent Treatment		
	Influent	Effluent	Removal	Influent	Effluent	Best Removal
Contaminant	(mg/l)	(mg/l)	(%)	(mg/l)	(mg/l)	(%)
BOD	63.7	6.1	90.4	200 - 24,000	9.0 - 20.6	99.95
Total P	25.8	6.2	75.9	--	--	--
Ortho - P	--	--	--	20	2.8	91.3
Ammonia	54.7	3.5	93.6	19.7	1.3	95.4
TKN	69.8	6	91.4	--	--	--
Nitrate	--	--	--	40	1.5	95.4
Fecal Coliforms*	175,164	1,040	99.4	905	0	
Fecal Strep.*	76,727	1,192	98.4	1,100	40	
Location	DeKalb County, Alabama			County Roscommon, Ireland		
Reference	Hammer et al., 1991			Costello, 1989		
* CFU/100ml						

The foregoing discussion and design parameters provided in Table 5.4 define systems for point source livestock wastes. Hammer (1991) also describes nutrient/sediment control systems (lower illustration in Figure 5.3) for the control of runoff NPS pollution. The non-point source agricultural system consists of a sediment basin, a rock filled distribution trench that ensures sheet flow over a meadow which routes water to a marsh-pond system. Water is discharged from the marsh-pond across a meadow or forested filter strip. The systems appear to be sized as 2% of drainage basin area and could consist of single units or several smaller wetlands sited to protect waterbodies. The runoff nutrient/sediment systems are reported to remove 90% - 100% of BOD and suspended solids, 85% - 100% of phosphorus and 80% - 90% of nitrogen from potato field runoff in northern Maine (Hammer, 1991).

The Ontario Ministry of Agriculture and Food (OMAF) is participating in the development of agricultural wastewater and runoff treatment wetlands. Four Ontario wetland systems were constructed in 1993 and at least three more are planned for 1994 (Toombs, 1994). The wetlands are sized based on FWS kinetic correlations (Section 4.4.2) and consist of sediment pretreatment ponds, marshes (removal of BOD, pathogens and suspended solids), ponds (BOD reduction, nitrification - denitrification) and wet meadows (algae removal, nitrification - denitrification).

Additionally, OMAF participated in three runoff vegetated filter strip projects in 1993 with five more planned for 1994. The filter strips are designed with slopes of 0.5 to 4 percent, are 90- to 260 m long and are typically 9 m or more wide. Toombs (1994) reports that vegetated filter strips in the US and Quebec reduce suspended solids, phosphorus, ammonia and total nitrogen by over 99%.

An Alabama swine waste system was developed in 1988 at a cost of \$12,800 for a treatment area of 5,700 m² that served a 500 animal operation. Notably the cost included \$3,500 to remove an old lagoon (Hammer et al., 1991). Reported nutrient/sediment control system construction costs range from \$13,500 for 8 ha and \$23,000 for 67 ha watershed or \$50 - \$60 / ha·year (amortized capital plus ongoing maintenance) for the contributing watershed (Hammer, 1991).

US experience with WWT for agricultural water pollution points to its high potential as an affordable technology for agricultural operations. In southern Alberta potential applications will include management of feedlot runoff and wastewater streams and improvement of irrigation return flow water quality. The significant number of projects undertaken by OMAF illustrates the applicability of the technology in Canada. It is the author's opinion that pilot testing of WWT for this purpose in western Canada should have priority.

5.3 Industrial Wastewater and Landfill Leachate WWT

WWT has potential application for treatment of industrial wastewaters and work site runoff streams. A wide variety of applications are documented including a surprising number of Canadian and other cold climate applications.

This section will review experiences reported but will not discuss design details of the applications. Industrial WWT systems typically are specific to the applications and related design procedures were not discussed in the literature reviewed. It is anticipated that initial evaluations of land area and required HRT could be derived from correlations in Section 4 for many waste streams where BOD, suspended solids or nutrients are problems. Removal of persistent organics or toxic metals will require specific

investigation beyond the scope of this report, however, some insight on metals removal is given in Section 5.4.

5.3.1 Pulp Mill Effluent

Thut (1989) described a Weyerhaeuser - National Space Technology Laboratory study conducted in Mississippi on secondary treated bleached kraft pulp mill effluent. The laboratory study compared contaminant removal of monotypic plantings of cattail (*Typha latifolia*), reed (*Phragmites australis*) and cordgrass (*Spartina cynosuroides*) with a non-vegetated control cell (all cells were 1.2 m x 3.6 m x 0.6 m deep with 0.4 m of 10-20 mm rock substrate). The vegetation was found to enhance ammonia and phosphorus removal relative to the control but the WWT systems did not effectively remove color constituents. Table 5.6 summarizes results.

Table 5.6: WWT for Treated Pulp Mill Effluent
(after Thut, 1989)

Contaminant	Influent Conc. (mg/l)	Removal Efficiency (%) for 24 Hour Retention			
		Control*	Cordgrass	Cattail	Reed
TSS	5 - 8	46	42	58	47
BOD	10 - 15	45	44	49	36
Ammonia	1.5 - 3.0	24	75	79	79
Organic - N	2.5 - 3.5	38	36	33	27
Phosphorus	0.5 - 0.8	8	18	41	26
*Rock substrate of 10 - 20 mm marl 0.6 m deep with no vegetation.					
Same marl substrate used for vegetated cells					

The table demonstrates that WWT can have impact even at dilute wastewater contaminant concentrations. Thut (1989) reported that untreated black liquor was applied over a period of two weeks (simulating a spill of pulp treating solution) without effecting the plants. Consequently, it is possible that tests with more concentrated effluents may demonstrate a stronger relationship between wetlands vegetation and BOD removal.

5.3.2 Industrial Site Runoff

An experimental constructed FWS wetland has been developed by Imperial Oil in Chilliwack, British Columbia for final treatment of hydrocarbon (diesel) contaminated

site runoff. The system was constructed in 1992 and evaluation is ongoing (Stecko, 1993, pers. comm.). The three section cattail wetland has a surface of 1000 m², a volume of 250 - 450 m³ and depth of 0.25 - 0.45 m.

Because of very low influent hydrocarbon loading, "spiked" diesel additions were made to test the performance of the wetland. Addition of sufficient diesel to raise the wetland to a theoretical 10 ppm resulted in a peak first section effluent concentration of 2.1 ppm with minimal discharge from the 16 hour HRT total system. Notably the regulatory requirement is 5 ppm. Subsequent tests continue to demonstrate the effectiveness of the wetland and work is being done to define removal mechanisms.

5.3.3 Petroleum and Oilsands Wastewater Management with WWT

Both Syncrude and Suncor have WWT research programs in place at Fort McMurray, Alberta. The Syncrude research is aimed at using wetlands to stabilize tailings (Boerger and McKinnon, 1993). Initial results are favorable with *Typha* and *Rumex* reported as establishing easily. The plants may stabilize the clay fines sludge in the tailings and provide an opportunity to convert the tailings pond to a major wetland.

Suncor is investigating wetlands as a means to control quality of tailings pond leachate and runoff (Bishay et al., 1993). The Suncor project consists of nine lined trench type WWT cells developed in 1991. Cattail (*Typha latifolia*) and bulrush (*Scirpus validus*) were planted at four clumps per m² with plants obtained from local donor sites. Full coverage of the 50 m x 2 m wide (bottom width) channels was achieved in one growing season. Testing was done at retention times of 12 - 18 days (hydraulic loading of 1.7 - 4.2 cm/day). Reported results show 88% - 98% of nitrite-nitrate nitrogen removed along with 38% - 98.6% of phosphorus. Total extractable hydrocarbons (TEH) were reduced by 35%-75%.

Both projects demonstrate the ability to establish treatment wetlands in cold climates and show WWT potential for treating petroleum industry wastewaters. Suncor is continuing work to evaluate the fate of removed contaminants and to define optimal design parameters.

The Amoco petroleum refinery at Mandan, North Dakota uses a complex of wetlands to treat process wastewater prior to discharge into the Missouri River (Litchfield and Schatz, 1989). The system involves pretreatment in an API separator followed by a 6 ha aerated lagoon from which water is pumped to a series of six ponds created by damming natural drainage channels. Cattail, bulrush and other plants have naturally established in down stream ponds. An additional five ponds were developed for wildlife habitat and to provide holding capacity during periods of high runoff or system upsets. Discharge from the system averaged 2,500 m³/day in 1987. Table 5.7 summarizes effectiveness of the refinery WWT system.

The decision to develop the ponds at a \$250,00 cost was based on evaluation of alternatives such as carbon adsorption and activated sludge. Alternate wastewater treatment upgrade systems were estimated at between \$1 million and \$3 million. The wastewater system and wildlife areas cover 267 ha of the 389 ha refinery site and have become a significant habitat area for fish, waterfowl and animals. The refinery has received major environmental awards for the WWT project.

Table 5.7: Mandan Refinery WWT Performance
(after Litchfield and Schatz, 1989)

Contaminant	WWT Influent (kg/day)	WWT Effluent (kg/day)	Reg. (NPDES) Limit (kg/day)
BOD	79.4	12.4	197.7
COD	346.7	101.0	1477.6
Ammonia	16.9	2.6	131.8
Sulfides	0.2	ND	1.3
Phenol	0.2	0.01	1.5
Oil-Grease	21.4	1	59.9
TSS	106.1	11.7	137.8

5.3.4 Landfill Leachate WWT

Water leaching from landfill sites can contain excessive concentrations of BOD, arsenic, ammonia, manganese, nitrate, benzene, iron and other pollutants (Staubitz et al., 1989, Trautmann et al., 1989). Conventional treatment processes can be capital and energy intensive, however, wetland systems incorporating features to remove metals,

nutrient and organics can be substantially less expensive than conventional alternatives. WWT was estimated to save a New York town \$21,000/year on costs to treat 3.8 m³/day of landfill leachate (Trautmann et al., 1989).

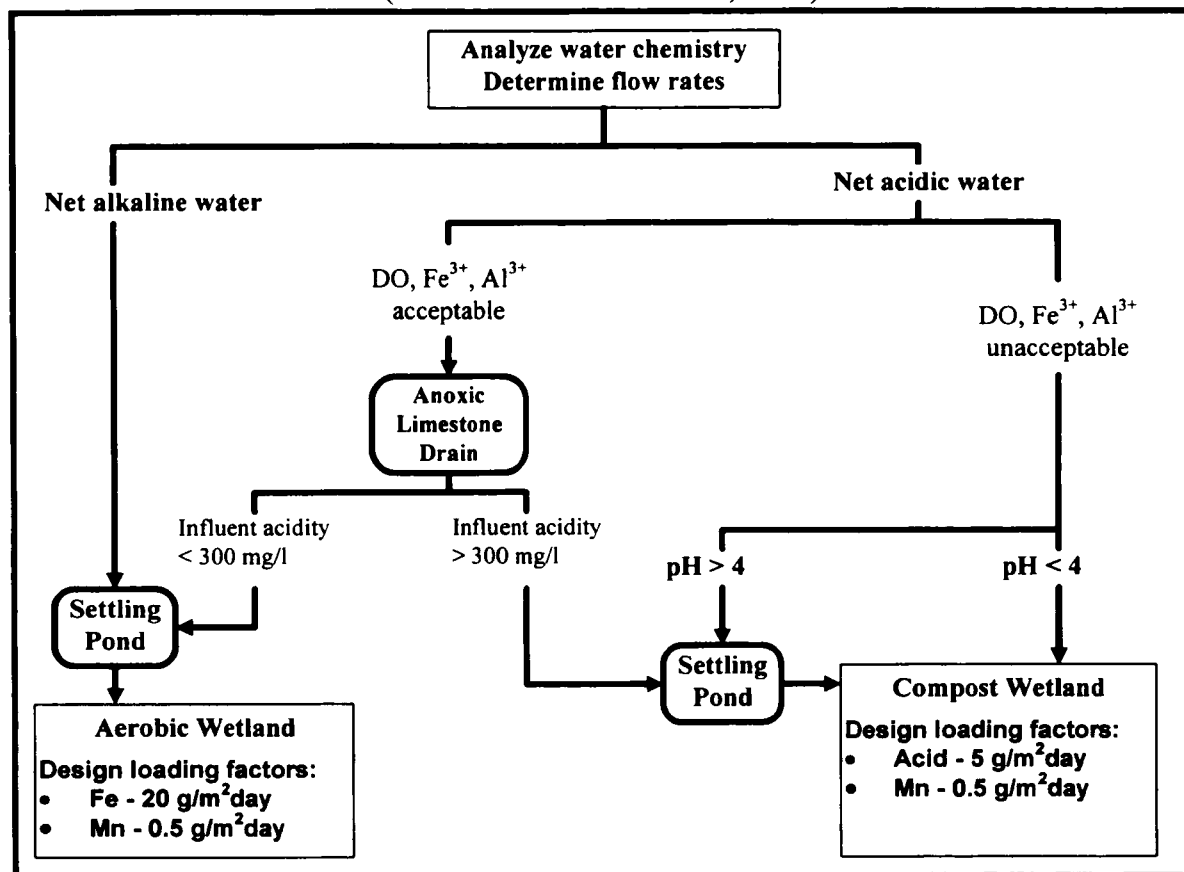
A typical approach to deal with landfill drainage is to collect it and pipe or truck the water to conventional sewage plants (Staubitz et al., 1989, Trautmann et al., 1989). Especially as sites are abandoned (and reclaimed), it is desirable to deal with leachate in low capital, maintenance and operation cost systems such as WWT. Designs discussed by both Staubitz et al. (1989) and Trautmann et al. (1989) are based on VSB technology. Pilot systems, located in New York State, were designed for 15 day HRT (hydraulic loading of 0.3 m³/day·m² cross sectional area) in reed (*Phragmites australis*) vegetated, 0.6 m deep gravel VSB systems (Staubitz et al., 1989).

5.4 Acid Mine Drainage (AMD) and WWT

Oxidation of sulfur containing materials (e.g., pyrite) exposed by mining produces drainage and runoff that is acidic with elevated concentrations of contaminants such as iron, manganese, aluminum, suspended solids and sulfate. It has been estimated that AMD has adversely affected over 17,000 km of the Appalachian region rivers and streams alone (Hammer, 1987). In the US, over 500 wetlands systems have been installed for control of mine drainage (Cooper et al., 1993). The systems are used to mitigate acid waters and associated metal contamination from mine sites and tailings piles. This subsection will discuss the basic AMD wetland design approach proposed by the US Bureau of Mines.

Wetlands remove solids by settling and metals by hydrolysis (aerobic acid producing reactions), reduction (include sulfide formation) and adsorption. Acidity can be reduced by limestone dissolution and sulfate reduction. The pathways are described in Section 2. The US Bureau of Mines (1992) has summarized AMD design procedures and sizing factors as illustrated in Figure 5.4.

Figure 5.4: AMD Wetland Design Flowchart
(after US Bureau of Mines, 1992)



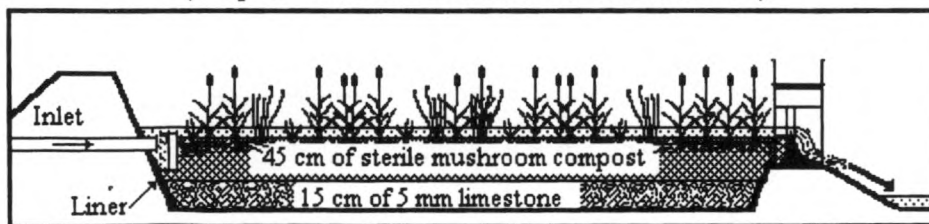
AMD should be analyzed for pH, dissolved iron, alkalinity and acidity with the latter two expressed as mg/l CaCO_3 equivalent. If it is necessary to increase alkalinity with a limestone drain, ferrous iron (Fe^{2+}), dissolved aluminum and dissolved oxygen (DO) need to be assessed. Ferric iron (Fe^{3+}) is inferred as the difference between total dissolved iron and ferrous iron (US Bureau of Mines, 1992).

Dissolved iron will precipitate from net alkaline water under aerobic conditions due to formation of oxyhydroxide precipitates (Kleinmann et al., 1991). The reactions will increase acidity of the water thus require buffering provided by net alkaline conditions. The aerobic iron removal route is an essentially abiotic process. Designs incorporating oxygenating features such as waterfalls or riprap ditches will enhance system performance (US Bureau of Mines, 1992). Shallow aerobic AMD wetlands are typically planted with cattails.

Anoxic limestone drains (ALD) are beds of crushed (2-4 cm) limestone (preferably with high CaCO_3 versus MgCO_3 or $\text{CaMg}(\text{CO}_3)_2$ content) which are sealed from the atmosphere and designed to intercept mine water before contact with air. Ferric iron, aluminum and dissolved oxygen in the mine water will cause failure of the ALD due to hydrolysis and precipitation of the two metals. The precipitates will coat the limestone and prevent dissolution needed to increase alkalinity. The US Bureau of Mines (1992) notes that where water quality is appropriate ALDs will produce alkalinity cheaper than compost wetlands. Figure 5.4 provides the design decision flow chart by the US Bureau of Mines.

Alkalinity is generated in compost wetlands by anaerobic oxidation of organic materials using sulfur as the electron acceptor (sulfate reduction as described in Section 2.8). Additional alkalinity is generated from CaCO_3 in the substrate (where CaCO_3 can be constituent of the substrate, added to the media or provided by an underlying crushed limestone bed). Spent mushroom compost, which typically contains limestone, has been found to provide good results (Kleinmann et al., 1991). Compost beds should be 30 - 45 cm deep. FWS wetlands can be expected to generate alkalinity at 2-12 $\text{g/m}^2\cdot\text{day}$ (US Bureau of Mines, 1992). While the systems are typically FWS type, residence times must be sufficient for mass transfer between aerobic surface water and anaerobic substrate. Plant growth in the system is important for biofilm surfaces as well as for an ongoing source of organic material for sulfate reduction processes. Figure 5.5 illustrates AMD compost wetland concepts.

Figure 5.5: AMD Compost Wetland Schematic
(adapted from Kolbash and Romanski, 1989)



Adsorption, ion exchange and complexation with organic materials are important removal mechanisms for metals. Eggar and Lapakko (1989) developed an AMD pilot

based on at least 50 cm deep peat beds vegetated with native vegetation including sedges (*Carex sp.*) and grasses (*Calamagrostis sp.*) in 5 cm water depth or cattails in 15 cm of water. They found that much of the metal removal occurs in the top 20 cm of peat and estimated 4000 tonnes of peat in a layer 20 cm deep could treat a seep (1200 m³/day, 1.7 mg/l Ni) for 90 years. Saturation of adsorption, ion exchange and complexation sites with time can lead to decline in effectiveness of AMD systems where those processes form a significant component of metals removal.

Egar and Lapakko (1989) reported that conventional treatment plant costs for the 1600 m³/day mine drainage from a northwest Minnesota site were estimated at \$8.5 million capital and \$1.2 million per year operating expense (OPEX). Passive approaches incorporating infiltration reduction, alkaline treatment and wetlands were estimated at \$4.0 million capital and \$40,000/year OPEX.. Kolbash and Romanski (1989) discussed a compost wetland that removed 50% of iron from coal mine waste pile drainage that was constructed for \$12,500. Hammer (1987) reported that a system that treated 5 m³/day to reduce iron levels from 80 mg/l to 2 mg/l was constructed for \$28,000. This compared favorably with the estimated \$10,000/year cost for treating chemicals to achieve similar results.

5.5 Wetlands Integration with Urban Runoff Control Systems

Non-point source (NPS) pollution is a major factor in degradation of natural water bodies (Baker, 1991). Section 5.2 demonstrated how WWT techniques can significantly reduce agricultural contamination of rivers and lakes. This section will show how wetlands technologies also provide a multi-use opportunity to deal with urban runoff problems.

Urban runoff best management practices (BMPs) provide both quantity and quality management of runoff and are based on natural wastewater treatment processes (discussed in Section 4). BMPs include:

- Facilities to reduce runoff by enhancing runoff infiltration including infiltration trenches and basins, as well as porous pavements.

- Landscape designs, such as grassed or forested filter strips, to slow flow and remove runoff contaminants.
- Facilities to attenuate peak flows through detention or retention of runoff including dry ponds, wet ponds and wetlands.
- Systems designed to permit sedimentation, removal and transformation of runoff contaminants such as wetlands and wet ponds.

No one BMP is a panacea for stormwater management and the systems can only be rationally evaluated and applied within the context of a drainage basin plan. Watershed hydrology and water quality issues must be assessed, objectives for water quantity and quality management developed and the potential of various BMP combinations to meet the objectives tested by computer simulation. Review of approaches to watershed evaluation and water quality management, however, are complex and beyond the scope of this document.

Wetlands technology applied to urban runoff may be the most significant potential application of WWT in western Canada. The technology is relatively low cost, is passive (requiring minimum operational and maintenance input) and provides opportunities to create multi-purpose urban amenities. The following sections will overview design of runoff BMPs, most of which are based on, or are enhanced by, wetlands ecotechnology.

5.5.1 Infiltration Systems

Infiltration trenches, porous pavements and infiltration basins attempt to direct runoff into ground water systems. The RI process discussed in Section 4.2.3 explains contaminant removal by infiltration systems which are illustrated in Figure 4.3. The systems are only suited to smaller drainage areas and are singularly reported to be prone to failure (Schueler et al., 1992). Failures typically result from inappropriate maintenance and plugging of infiltrative surfaces with runoff solids.

Porous pavements typically serve small drainage areas such as parking lots and individual buildings but may not be suitable for cold climates due to impacts of deicing chemicals (Schueler et al., 1992). Schueler et al. (1992) reported a 75% failure rate

within five years due to poor construction, clogging and inappropriate maintenance. Problems notwithstanding, design factors for infiltration units are shown in Table 5.8. Pollutant removal rates for infiltration systems are summarized in Table 5.9. The rates incorporate reduction of contaminant removal due to system bypassing during extreme runoff events.

Schueler (1987) discussed infiltration basin designs that incorporate features of dry ponds so that runoff above design infiltration capacity is detained before being discharged to surface or storm sewer flow systems. Basins that do not have this feature bypass excess runoff from large storm events.

Based on Schueler et al. (1992) infiltration systems in general do not appear to be attractive runoff BMPs even though initial contaminant removal effectiveness can be quite high.

Table 5.8: Infiltration System Design and Performance
(after Schueler, 1987 and Schueler et al., 1992)

Factor	Infiltration Trenches	Infiltration Basins
Soil Permeability	> 1.3 cm/hr	> 1.3 cm/hr
Slope	site < 20%, watershed < 5%	watershed < 20%
Depth to bedrock/water table	0.9 m	0.9 m
Watershed size	< 2 ha	< 6 ha
Pretreatment	sediment removal (e.g., filter strips)	sediment removal suggested (e.g., sediment forebay)
Storage volume size	Rule 1: 40-50% of runoff* Rule 2: 65-75% of runoff*	Rule 1: 40-50% of runoff* Rule 2: Runoff from 1" storm Rule 3: 2-year storm runoff
Relative cost (US \$ in 1987) (V_s = storage volume in ft^3)	Cost = $26.6 V_s^{0.63}$	Cost = $10.7 V_s^{0.69}$
Performance	life < 5 years	60% to 100% failure in 5 yrs
* sized to capture runoff from storm events that account for shown percentage of total annual runoff		

Table 5.9: Infiltration System Removal Rates
(after Schueler, 1987)

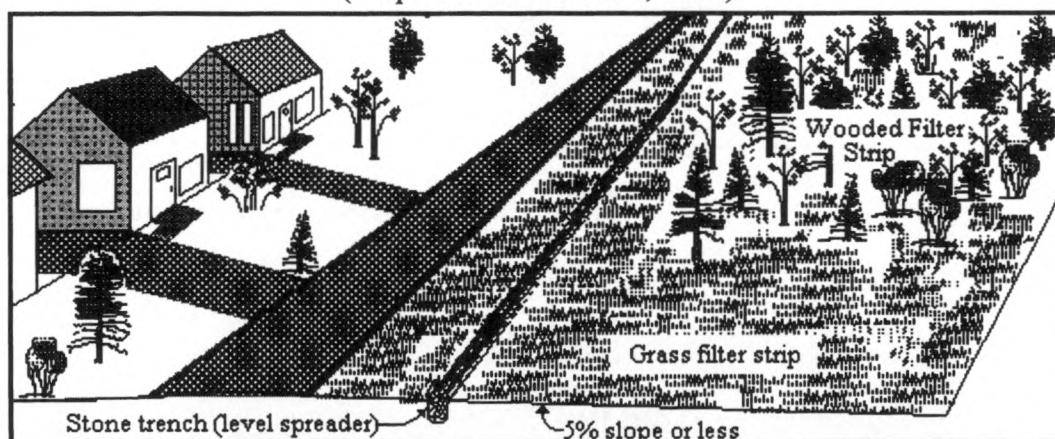
Contaminant	Infiltration Trench Removal (%)	Infiltration Basin Removal (%)	Porous Pavement Removal (%)
Sediment	99	75 - 99	82 - 95
Total P	65 - 75	50 - 75	65
Total N	60 - 70	45 - 70	80 - 85
Metals	95 - 99	75 - 99	98 (Pb), 99 (Zn)
BOD	90	70 - 90	82 (COD)
Bacteria	98	75 - 98	--

5.5.2 Biofiltration with Grass Strips, Swales or Urban Forest Buffers

Biofiltration techniques include filter strips with grass or urban forest vegetation, meadows and grass swales. Literature on BMPs (Schueler, 1987, Schueler et al., 1992, BC Environment, 1992) provides information on filter strips and grassed swales as discussed below, however, information on meadow components of WWT systems is given in Section 5.2 and Table 5.4. Biofiltration BMPs appear to have particular application for development adjacent to rivers and lakes and, with naturalized vegetation, should provide urban park amenity.

Grassed or Forested Filter Strips: Filter strips can be effective to remove sediments and act as buffers between developed areas and waterbodies or as pretreatment steps for other BMPs (Schueler, 1987, US EPA, 1990, Schueler et al., 1992, BC Environment, 1992). Filter strip concepts are illustrated in Figure 5.5. Design and performance factors for filter strips are summarized in Table 5.10.

Figure 5.6: Filter Strip Concepts
(adapted from Schueler, 1987)



Although not specifically addressed in the literature on BMPs, performance correlations for OF systems (Section 4.2.4) should apply to runoff contaminant removal. Filter strip design objectives will include careful flow distribution aimed at establishing sheet flow during runoff events. Ongoing maintenance should be expected to correct channeling that develops through settling, erosion or sediment build up. As illustrated in

Figure 5.6 wooded filter strips, in particular afford opportunities for enhanced urban aesthetics and wildlife habitat.

Table 5.10: Filter Strip Design and Performance Factors

Factor	Value	Reference
Strip length	min. 6 m, 15-30 m better, 30-90 m optimum	Schueler, 1987
Strip width	min. 15 m	Schueler et al., 1992
Strip slope	< 5%	Schueler, 1987
Contributing drainage area	< 2 ha	Schueler et al., 1992
Flow distribution	rock filled trench at constant elevation	Schueler, 1987
Slope bypass control	berms perpendicular to top edge every 15-30 m	Schueler, 1987
Vegetation	forested preferred over grass grass height 15-30 cm	Schueler et al., 1992 Schueler et al., 1992
Relative Cost*		
- grass seeding	\$3,600/ha	Schueler, 1987
- conifer seedlings	\$250/ha (manual planting + weed control)	Schueler, 1987
- nursery plantings	\$2,500-\$12,400/ha (stock + manual planting)	Schueler, 1987
Performance		
- TSS	20-100% (based on design manuals) 28% (one study, many with poor maint.)	BC Environment, 1992 Schueler et al., 1992
- COD	0-80% (based on design manuals)	BC Environment, 1992
- Pb, Zn	20-100% (based on design manuals)	BC Environment, 1992
- Nutrients (N, P)	0-60% (based on design manuals)	BC Environment, 1992
* "Relative cost" based on Schueler (1987) for Washington, D.C. area. Costs derived from these factors can be used for comparison with those of other BMP's estimated from similar "relative costs" presented in this report. The costs will not be accurate Canadian estimates.		

Grassed Swales: Swales are shallow vegetated channels that route and/or store runoff while providing some filtration and sedimentation. The concept is shown in Figure 5.7.

Figure 5.7: Grassed Swale Illustration
(adapted from Schueler, 1987)

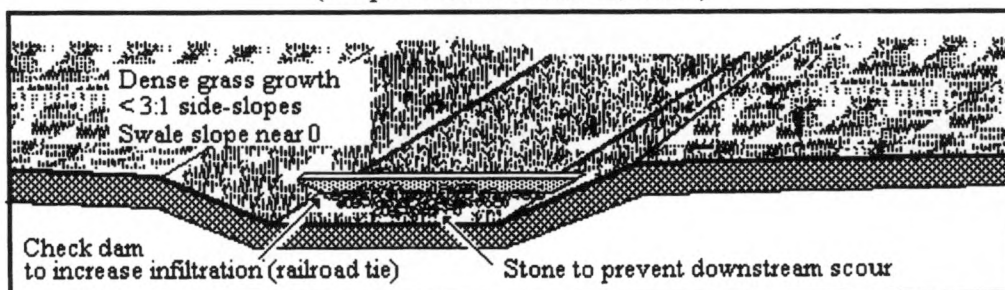


Figure 5.7 illustrates an "enhanced" swale which uses small check dams to increase runoff storage and settling (Schueler, et al., 1992). Swales provide biofiltration and can be an alternative to piped conveyance systems. Runoff depth and velocity are key factors affecting swale performance. Table 5.11 discusses swale design and performance factors.

Drainage area size for grassed swales are limited by maximum water velocity considerations (Schueler et al., 1992). Mowing and sediment removal are required maintenance procedures, however, convincing adjacent landowners not to cut the grass too short is an issue. Grass should be left sufficiently long to provide filtering action and close clipping typical of residential lawns reduces the effectiveness of the BMP (BC Environment, 1992).

Table 5.11: Grassed Swale Design and Performance Factors

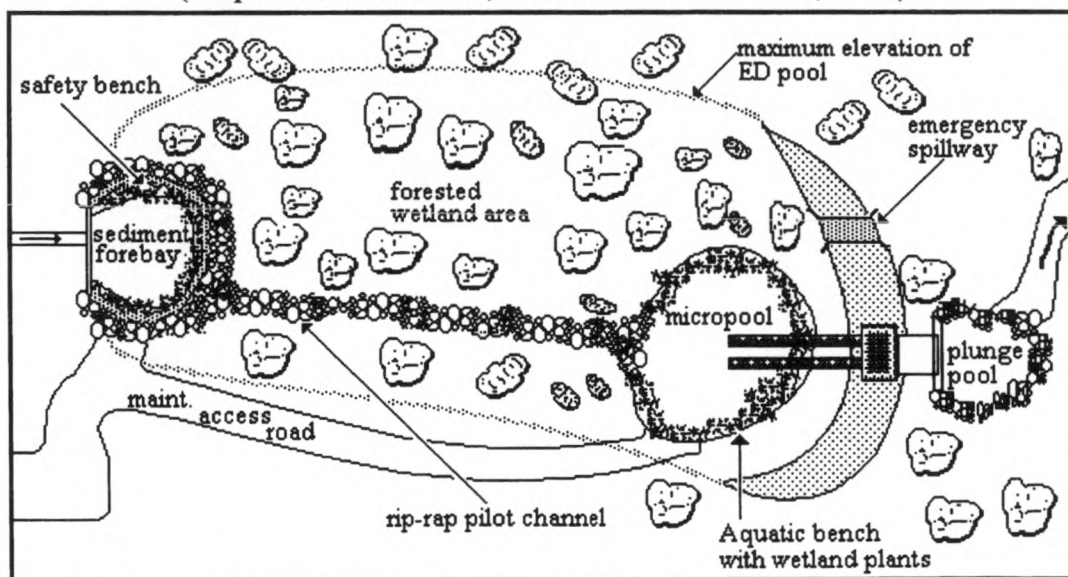
Factor	Value	Reference
Swale bottom slope	as close to 0% as possible	Schueler, 1987
Swale side slope	shallow not greater than 3:1	Schueler, 1987
Subsoil permeability	> 1.3 cm/hr	Schueler, 1987
Runoff velocity	< 0.45 m/s	Schueler et al., 1992
Swale length	> 60 m	Schueler et al., 1992
Vegetation	dense cover of water tolerant grass	Schueler, 1987
- grass height	design water depth + at least 5 cm	BC Environment, 1992
Dry weather flow	avoid -- reduces pollutant removal	Schueler et al., 1992
Desirable features	plunge pools, check dams, pocket wetlands	Schueler et al., 1992
Relative cost*	<u>for swale 4.5 m wide with 3:1 side slopes</u>	
- seeding/straw mulching	\$15/ running m	Schueler, 1987
- sodding	\$25/ running m	Schueler, 1987
Performance	<u>Design Manuals / Field Studies</u>	
- TSS	0-40% / 80%	BC Environment, 1992
- COD	0-40% / 25%	BC Environment, 1992
- Pb	0-20% / 50-80%	BC Environment, 1992
- Zn	0-20% / 50-60%	BC Environment, 1992
- Nutrients (P and N)	0-40% / 0%	BC Environment, 1992
* "Relative cost" based on Schueler (1987) for Washington, D.C. area. Costs derived from these factors can be used for comparison with those of other BMP's estimated from similar "relative costs" presented in this report. The costs will not be accurate Canadian estimates.		

5.5.3 Dry Ponds

Extended detention or "dry" ponds are basins that retain a portion of runoff from a few hours to a day or more. Basic dry ponds are grassed depressions that have a restricted discharge. In Calgary, they are designed to bypass low flows in the piped storm sewer system and the ponds only fill when runoff exceeds capacity of storm sewer flow control devices (restriction orifices that prevent excess flow rates downstream of the dry pond). These depressions can be part of recreation fields or park areas. Basic dry ponds are flood control devices which reduce peak downstream flow rates, hence the size and cost of conveyance systems. Accumulated sediment and grass clippings are subject to scour and resuspension during storm events (Pope and Hess, 1988, BC Environment, 1992), thus conventional dry ponds have limited effect on runoff quality.

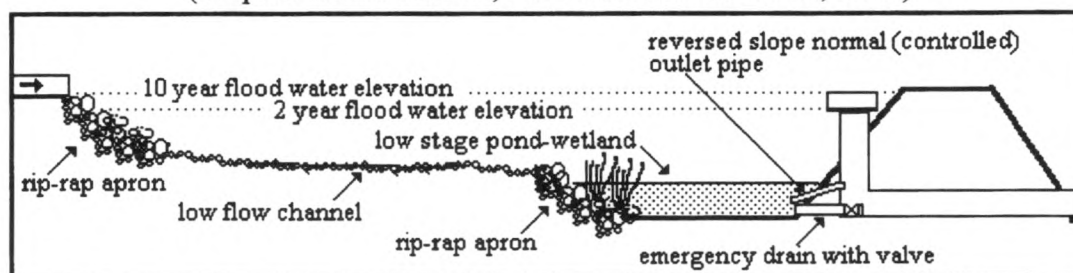
Schueler, et al. (1992) has proposed enhancements of dry ponds to improve the contaminant removal effectiveness of the systems as illustrated in Figure 5.8. Basic to the design is the routing of base flows through the dry pond and discontinuation of piped low-flow bypass systems.

Figure 5.8: Enhanced Dry Pond Illustration
(adapted from Schueler, 1987 and Schueler et al., 1992)



Dry ponds designed as flood control devices are not recommended as water quality improvement BMPs (BC Environment, 1992). Schueler et al. (1992) reported that dry ponds are prone to clogging, sediment accumulation and difficult mowing and notes that many fail to perform as designed. The reference suggests addition of a discharge micropool to protect the outlet orifice system along with two-stage design with a wetland in the lower stage to prevent contaminant resuspension. The additions are illustrated in Figures 5.8 and 5.9. Design and performance information for extended detention ponds is presented in Table 5.12.

Figure 5.9: Enhanced Extended Detention Pond Schematic
(adapted from Schueler, 1987 and Schueler et al., 1992)



Even relatively short detention can improve water quality. Schueler (1987) noted that 60% - 70% of sediments in urban runoff settle in six hours and that maximum settling after 48 hours is 80% - 90%. Detention time in the pond is a function of inflow rates, controlled outflow rates and the volume of any permanent pools. Pond discharge rates are typically set by downstream system capacity constraints, however, desired detention times within the pond should be taken into account when designing for water quality improvement.

While water quality is enhanced by long settling time, perceived safety of the ponds is not. Depending on the design, water in the normally dry play area or recreation field can be one or more metres deep, presenting a drowning hazard for children. This factor similarly works against permanent micropools in ED sites, although the danger can be mitigated by a shallow wetland buffer or bench along the margins of the pools. Shallow side slopes also enhance safety and Alberta guidelines recommend slopes of less than 7:1 for dry ponds (Hodgson et al., 1987).

Table 5.12: Dry Pond Design and Performance Factors

Factor	Value	Reference
Sizing Rules		
- #1	V= 1.3 cm runoff from watershed, release over 40 hr	Schueler, 1987
- #2	V= runoff from 1 in 1 yr - 24 hr storm, release over 24 hr	Schueler, 1987
- #3	V= runoff from 1 in 2 yr - 24 hr storm, release over 24 hr	Schueler, 1987
- #4	V= runoff from a 2.5 cm storm, release over 24 hr	Schueler, 1987
- #5	V= "first flush" runoff (1.3cm/ ha), release over 24 hr	Schueler, 1987
Small storm detention	> 6 hr	Schueler, 1987
Two-stage pond design		Schueler, 1987
- lower stage	V= (mean storm amount)x(runoff coef.)x(drainage area)	Schueler, 1987
- upper stage	size so that sum of both stages meets selected sizing rule	Schueler, 1987
Side slopes	7:1	Hodgson et al., 1987
Upper stage slope	2% - 5%	Schueler, 1987
Minimum drainage area	4 ha	Schueler, 1987
Enhancements	establish wetland in low stage to prevent resuspension use micropool to prevent outlet clogging avoid concrete lined pilot channels	Schueler et al., 1992 Schueler et al., 1992 Schueler et al., 1992
Costs		
- relative capital*	Cost = 10.71 (V**0.69), V in cu. ft.	Schueler, 1987
- maintenance	estimate at 4% - 5% of construction cost per year	Schueler et al., 1992
Performance	<u>Design Manuals / Field Studies</u>	
- TSS	50-100% / 3-74%	BC Environment, 1992
- COD	0-60% / 16-41%	BC Environment, 1992
- Pb	75-90% / 24-84%	BC Environment, 1992
- Zn	30-60% / 40-65%	BC Environment, 1992
- Total P	0-60% / 10-56%	BC Environment, 1992
- Total N	0-40% / 24-60%	BC Environment, 1992
* "Relative cost" based on Schueler (1987) for Washington, D.C. area. Costs derived from these factors can be used for comparison with those of other BMP's estimated from similar "relative costs" presented in this report. The costs will not be accurate Canadian estimates.		

5.5.4 Wet Ponds

Retention ponds or wet ponds are effective flood and water quality BMPs (Schueler, 1987). Wet ponds are permanent water bodies often two metres in depth where flood volumes are managed by allowing levels to temporarily increase, typically by up to two metres (Hodgson et al., 1987). The ponds have been successfully used in cold climates including major structures in Calgary. Adaptations to cold climate include inlets designed with pipe crown heights below expected ice level (Hodgson et al., 1987). Figures 5.10 and 5.11 illustrate wet pond concepts.

Figure 5.10: Enhanced Wet Pond Schematic
(adapted from Schueler, 1987 and Schueler et al., 1992)

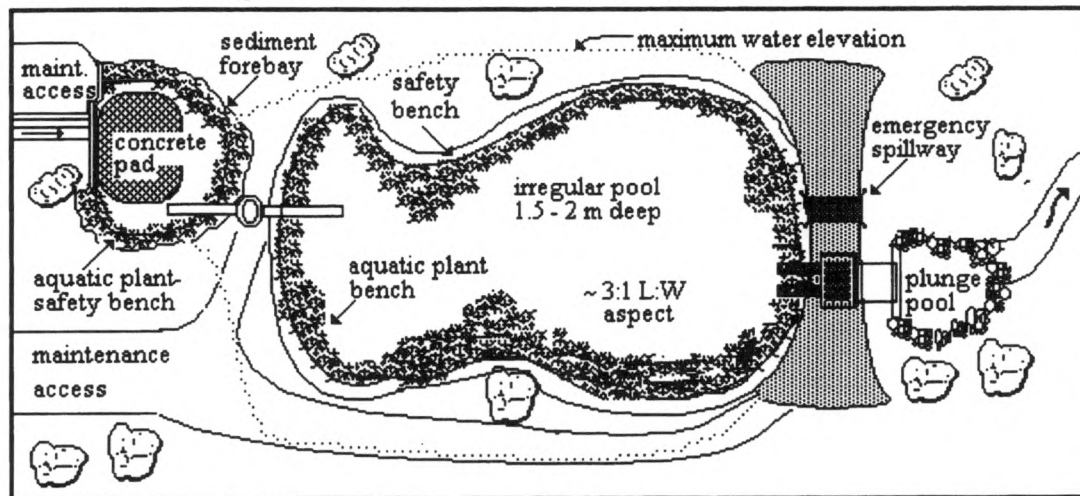
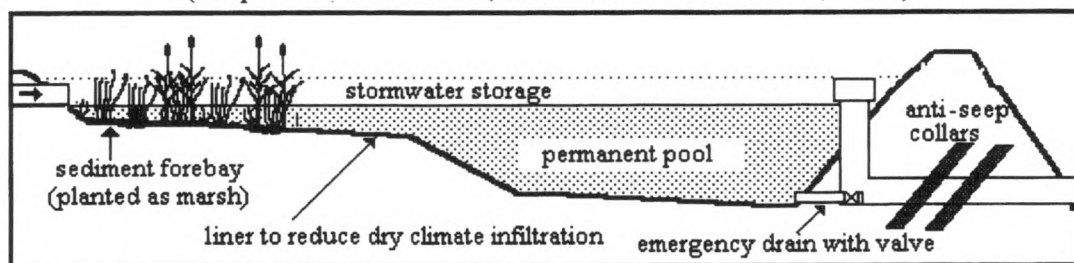


Figure 5.11: Wet Pond Cross-Section
(adapted from Schueler, 1987 and Schueler et al., 1992)



Enhancements to improve performance and maintainability have been suggested by Schueler et al. (1992). These include easily cleaned sediment forebays and fringe wetlands that provide incremental pollutant removal and passive safety barriers. The permanent pool of the wet pond helps mitigate scour and resuspension of sediments that can occur in dry ponds (BC Environment, 1992) and should maintain stable bottom biochemical conditions to prevent dissolution of precipitates. Figure 5.11 shows the significance of the permanent pool.

In climates where seasonal evaporation exceeds precipitation, hydrology of wet ponds must be closely evaluated. Excessive algae growth, high drawdown with subsequent odors (from decaying plant detritus) and problems similar to dry ponds can develop where a sustaining base flow is not available.

Pollutant removal mechanisms in wet ponds will be similar to those for stabilization lagoons (Section 4.2.1) and include settling, algal settling, wetland plant uptake (if part of wet pond) and bacterial decomposition (Schueler et al., 1992). Design and performance information are summarized in Table 5.13.

Table 5.13: Wet Pond Design and Performance Factors

Factor	Value	Reference
Permanent Pool Size		
- #1	Vp= 1.3 cm runoff over watershed	Schueler, 1987
- #2	Vp= 1.3 cm runoff from impervious areas of watershed	Schueler, 1987
- #3	Vp= 2.5 times runoff from mean storm	Schueler, 1987
- #4	Vp = 2 weeks HRT (about 4 times mean storm runoff)	Schueler, 1987
- optimum performance	Vp= 1.3-2.5 cm runoff from contributing watershed	Schueler et al., 1992
- Area	Ap > 1% of contributing watershed	BC Environment, 1992
Detention Volume	design to meet flood control requirements major event rise can be > 2 m	Hodgson et al., 1987
Contributing watershed	4.5 ha < watershed < 250 ha (baseflow needed if < 8 ha)	Schueler et al., 1992
Base flow	required in dry climates to maintain permanent pool	Schueler et al., 1992
Length to width ratio	L:W > 3:1	Schueler, 1987
Permanent pond depth	0.6 m < depth < 2.4 m (avoid thermal stratification) depth should be varied with 25% < 0.6 m	Schueler, 1987 Schueler, 1987
Aquatic plant shelf	3 m wide -- 15-30 cm deep	Schueler, 1987
Side slopes	3:1 < slope < 20:1 (> 10:1 for wildlife habitat)	Schueler, 1987
Enhancements	sediment forebays, wetland inlet/perimeter, reverse slope pipes	Schueler et al., 1992
Costs	(Vs= capacity in cu. ft. to emergency overflow)	
- Relative Capital	Cost = 6.1 (Vs**0.75) -- Vs < 100,000 cu. ft. Cost = 34 (Vs**0.64) -- Vs > 100,000 cu. ft.	Schueler, 1987 Schueler, 1987
- Maintenance	3% - 5% of construction cost per year	Schueler et al., 1992
Performance	Design Manuals / Field Studies	
- TSS	60-100% / 5-91%	BC Environment, 1992
- COD	20-60% / 2-69%	BC Environment, 1992
- Pb	20-80% / 9-95%	BC Environment, 1992
- Zn	20-80% / 0-79%	BC Environment, 1992
- Total P	40-80% / 3-79%	BC Environment, 1992
- Total N	20-80% / 0-60%	BC Environment, 1992
* "Relative cost" based on Schueler (1987) for Washington, D.C. area. Costs derived from these factors can be used for comparison with those of other BMP's estimated from similar "relative costs" presented in this report. The costs will not be accurate Canadian estimates.		

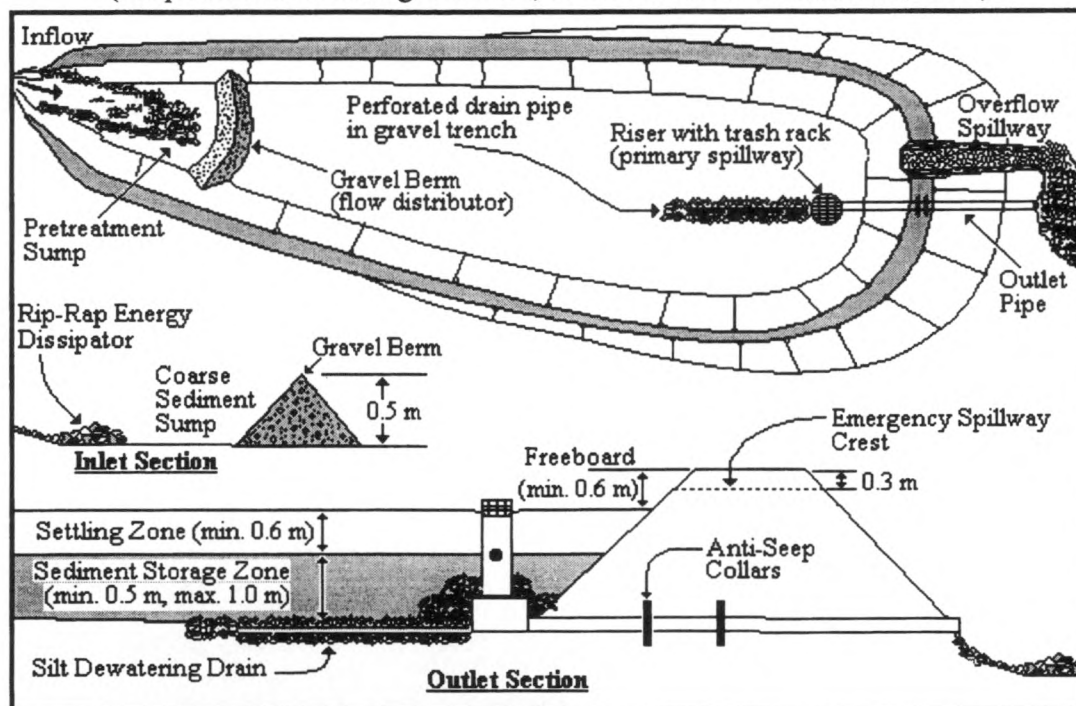
5.5.5 Sediment Control Ponds

Wet ponds and extended detention basins remove sediments from runoff, however, their primary function is to provide control of downstream discharge rates. Sediment control traps or basins are BMPs specifically designed for the purpose of

improving water quality and are not necessarily designed as detention facilities. Sediment control basins are commonly specified for construction phase protection of receiving water bodies (Washington DOE, 1992, BC Environment, 1993).

Despite relatively short HRTs (at design inlet rates), sediment ponds can be effective for water quality improvement. Schueler (1987) reports that 65% removal of suspended solids can be expected with residence times of 6 to 12 hours. Associated pollutant removals can also be significant with 84% of lead and 30% of chemical oxygen demand (COD) removed (Schueler, 1987). Design guidelines for sediment basin BMPs for the Puget Sound area are based on a theoretical trap efficiency of 80% (Washington DOE, 1992). Figure 5.12 illustrates typical sediment pond features.

Figure 5.12: Sediment Removal Basin Illustration
(adapted from Washington DOE, 1992 and BC Environment, 1993)



Sedimentation principles are discussed in Section 2.2. Typical guidelines are based on removal of medium silt or particles larger than 0.02 mm (Washington DOE, 1992, BC Environment, 1993). Table 5.14 provides design guidelines in addition to information displayed in Figure 5.13.

Table 5.14: Sedimentation Basin Design Guidelines
(based on Washington DOE, 1992 and BC Environment, 1993)

Design Parameter	Guidelines
Minimum Pond Surface Area	$A = 1.2 Q_{10} / V_s$ Where Q_{10} = peak inflow from 1:10 year, 24 hr. Storm V_s = settling velocity of design particles = 0.0003 m/s for 0.02 mm particles $A = 0.01(\text{drainage area})$
Design Particle Size	0.02 mm (20 micron) or “medium” silt
Design Retention Time	minimum 40 minutes
Design Drawdown Time	48 hours
Flow Path (L:W)	L:W at least 3:1 5:1 suggested for BC
Freeboard	minimum 0.6 m
Free Settling Depth (SD)	minimum 0.6 m $L/SD < 200$
Sediment Storage Depth	0.5 m < sediment depth < 1.0 m

Horizontal velocity should be checked to ensure scour conditions do not exist. The 5:1 length to width ratio suggested in the BC guidelines may result in major event scouring of sediments deposited during smaller storms. Horizontal velocity can be controlled by providing for a deeper settling zone or by reducing the L:W ratio.

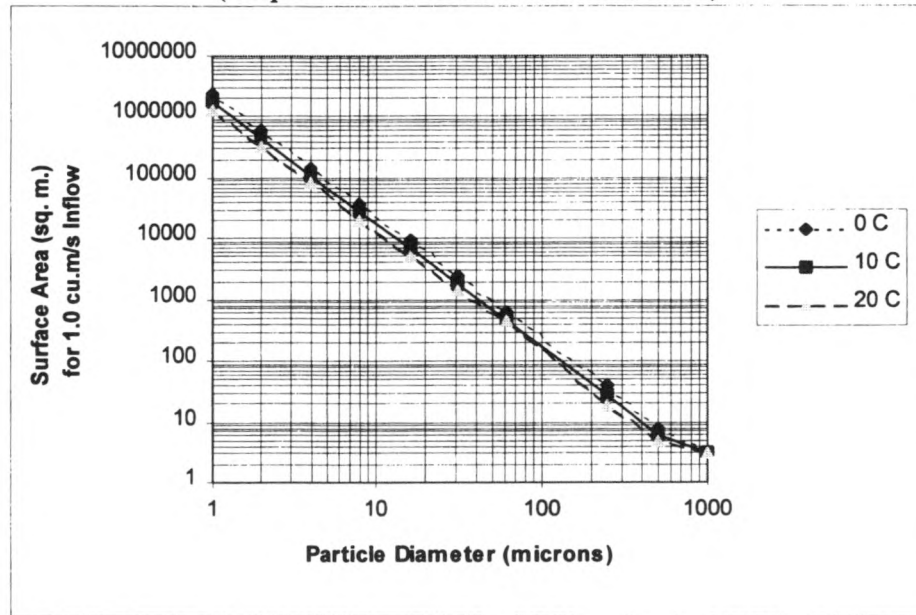
Design particle size should be assessed for the specific application. Particles 0.02 mm and larger represent 80% of the solids in typical Puget Sound runoff (Washington DOE, 1992). Where clay soils are prevalent and water quality is critical smaller design particles may be specified and larger areas will be needed. Designs based on 0.01 mm will require three times the surface area of the 0.02 mm guideline. Conversely designs based on fine (0.05 mm) or medium (0.25 mm) sand can result in smaller ponds, especially where the prime purpose is minimizing solids load on downstream BMPs. Required surface areas as a function of particle diameter are shown in Figure 5.12. The chart is based on $1.0 \text{ m}^3/\text{s}$ inflow rate.

Presetting basins can be used to reduce sediment loads on other primary detention or treatment BMPs. The basins do not necessarily have permanent pools and are designed based on the following (Washington DOE, 1992):

$$A_s = -(Q_o/V_s) * \ln(1-E)$$

where A_s = surface area of presettling basin (m^2)
 Q_o = average settling basin release rate (m^3/s)
 V_s = settling velocity of target particle (m/s)
 = 0.00012 m/s (recommended for Puget Sound)
 E = trap efficiency (0.8 recommended for Puget Sound)

Figure 5.13: Settling Pond Surface Area versus Particle Size
 (adapted from BC Environment, 1992)



The release rate will be a function of treatment detention time. The Washington DOE (1992) bases its requirements on the 6-month, 24-hour storm (storms of this size and smaller account for 91% of runoff in the Puget Sound Basin). Detention time (t_d) of six hours is suggested for up to 0.5 cm runoff, 12 hours for 1.3 cm and 18 hours for 2.5 cm. The average release rate is defined by:

$$Q_o = V / (3600 t_d)$$

where V = runoff from design 24-hour storm (m^3)
 t_d = design detention time (hours)
 ≤ 24 hours

Presettling basins for prairie climates will require evaluation of what constitutes the design storm for water quality control purposes and the Puget Sound factors may be inappropriate. Based on the recommendations for wetland sizing by Schueler (1992), initial evaluations could be based on 0.6 cm runoff for the watershed.

The guidelines will prove useful for the design of extended detention, wet pond and wetland BMP sediment forebays. Limited information was found on sizing practices for the forebays and simple ratios of volume or area relative to the main pond or wetland may not provide sufficient removal of sediments. While design treatment volumes for such water quality BMPs may be based on smaller, more frequent events, the forebays should be sized to handle coarse materials from major storms. Moreover the inlet basins should be designed so that peak inflow rates do not flush accumulated sediments into the downstream sections intended to be protected by the forebay. This can be managed by increasing the size of the forebay or by diverting extreme inflows around the system. Effective sediment bays minimize clean out of the main sections of wetlands or other BMPs, thus, reduce overall maintenance costs.

5.5.6 Constructed Stormwater Wetlands

The discussion of biofiltration, dry extended detention and wet ponds illustrates that concepts of WWT enhance pollutant removal from stormwater. These demonstrate that wetland retrofits to such existing facilities may enhance water quality, however, deliberately designed stormwater wetlands may offer even greater opportunity to optimize the water quality, landuse, habitat and aesthetic benefits of WWT. This subsection will describe design and performance of wetlands designed to treat urban stormwater.

Four types of constructed stormwater wetlands have been described that can achieve varying levels of flood and water quality improvement within differing cost and space constraints (Schueler, 1992):

- ***Design #1: Shallow Marsh System:*** The marsh provides a large surface area and ample shallow water for a diversity of plants. The design requires base flow and is applicable for drainage areas larger than 10 ha to support permanent water levels. A sediment forebay and outlet micropool are incorporated into the design. The shallow marsh BMP is shown in Figure 5.14.

- **Design #2: Pond-Wetland System:** The pond-wetland system shifts the bulk of stormwater treatment to a deeper pool. This permits a lower marsh surface area than Design #1. The two cell design firstly treats water in a wet pond which is followed by a shallow marsh system and is illustrated in Figure 5.15.
- **Design #3: Extended Detention Wetland:** The ED wetlands are designed with vegetated, shallow sloping sides that permit temporary level increases of one metre. The ED design conserves space relative to Design #1 because vertical storage reduces marsh lateral area requirements. The concepts are shown in Figure 5.16.
- **Design #4: Pocket Wetlands:** Pocket wetlands are designed for unreliable flow from small (less than 4 ha) watersheds. The systems are designed for widely fluctuating water levels. Ideally the pocket wetlands receive some support by having lower pools below groundwater level. Where base flow or groundwater is absent, species diversity, habitat value and pollutant removal capability is limited relative to other types of stormwater wetlands. The pocket wetland approach is presented in Figure 5.17.

Figure 5.14: Shallow Marsh Stormwater Wetland
(adapted from Schueler, 1992)

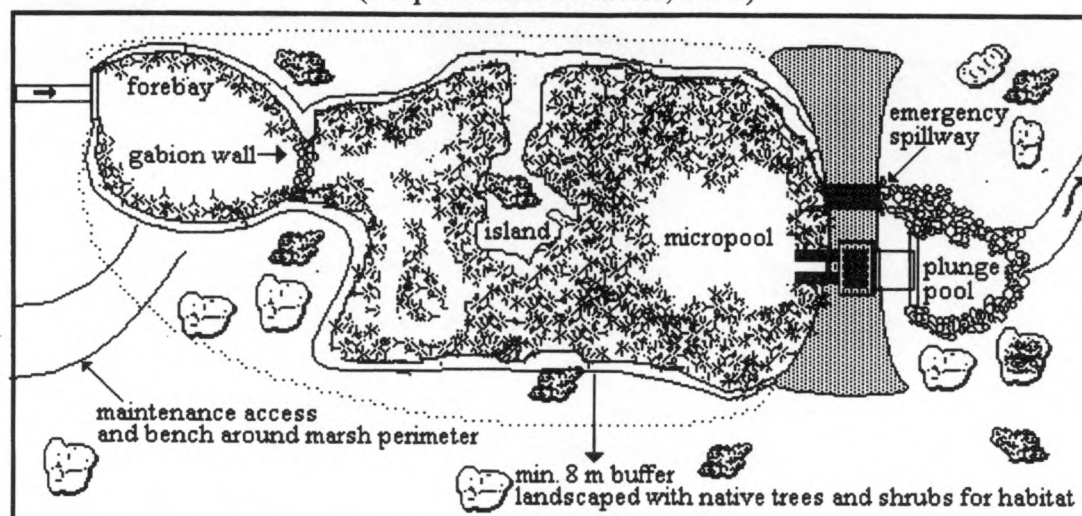


Figure 5.15: Pond-Wetland System
(adapted from Schueler, 1992)

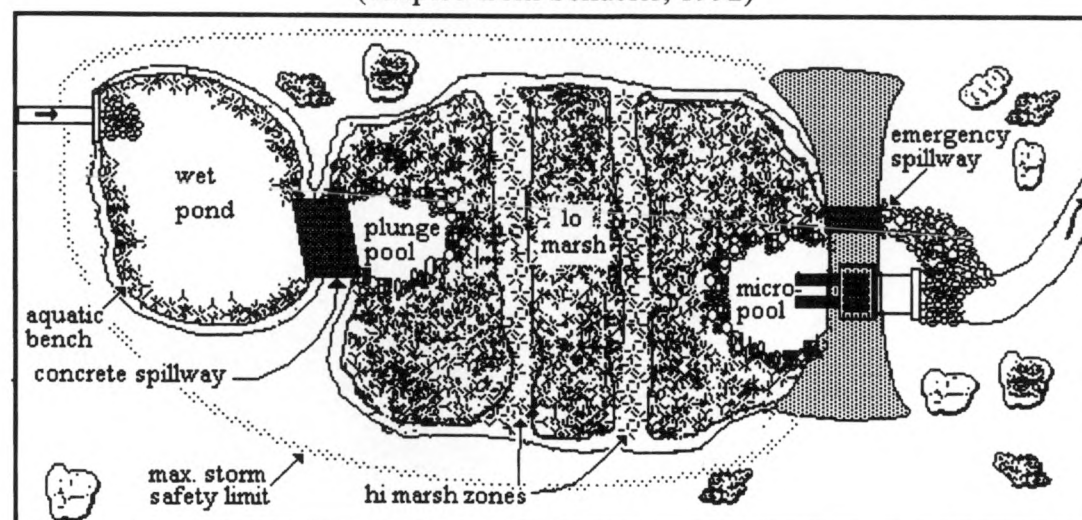


Figure 5.16: Extended Detention Wetland
(adapted from Schueler, 1992)

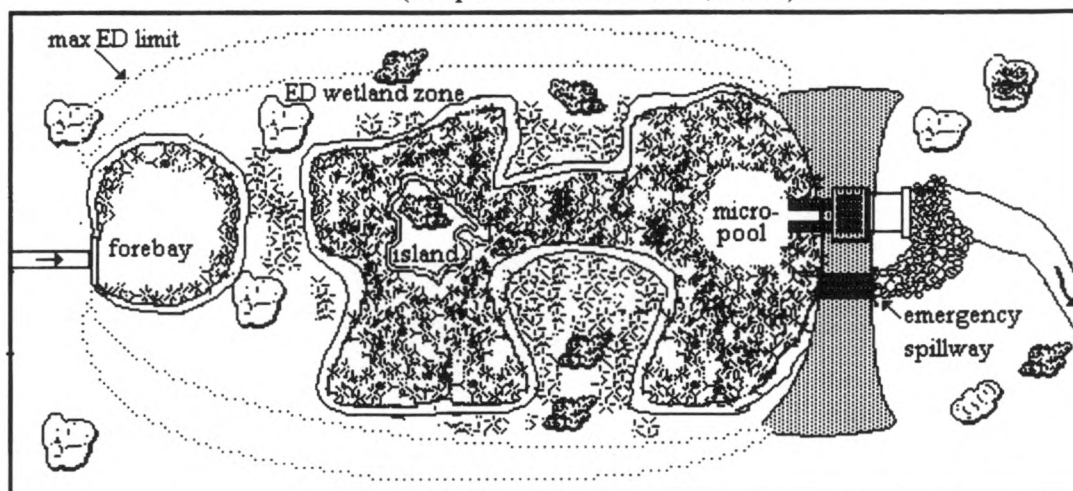
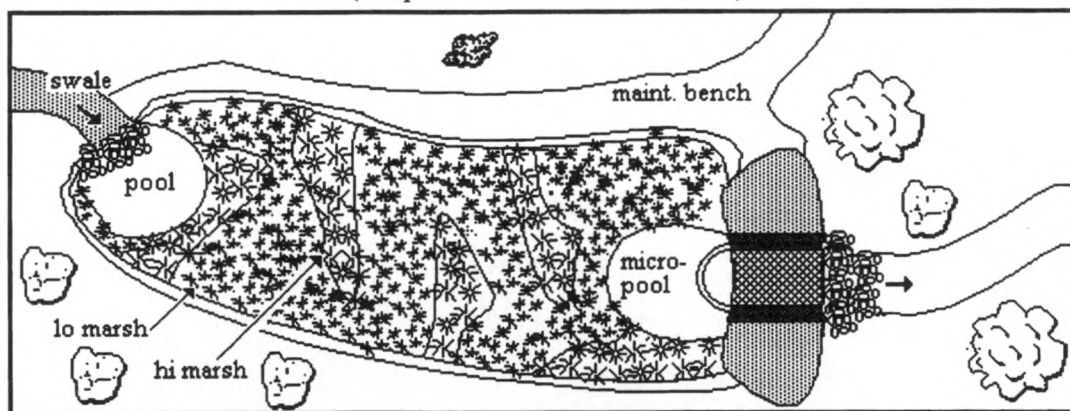


Figure 5.17: Pocket Wetland
(adapted from Schueler, 1992)



Design procedures for all four systems are a function of desired pollutant removal and stormwater retention. Procedures for agricultural and sewage treatment wetlands could be used for runoff WWT where reduction of known levels of specific contaminants is desired. Schueler (1992) was found to be the most complete reference on stormwater wetland design and he described nine key project considerations, as well as provided design information summarized in Table 5.15.

1. **Design Treatment Volume:** The wetland site should treat 90% of all runoff. Design is based on the local rainfall frequency spectrum and runoff volumes that can be expected at the proposed BMP location. The "design storm", then is the event for which 90% of annual runoff will be delivered by it and smaller storms. This volume is referred to as V_t (treatment volume) and defines the volume of the permanent pool, pond-wetland or marsh wetland system and excludes temporary extended detention above normal water levels. As a minimum, V_t should be equivalent to 0.6 watershed cm.
2. **Wetland Area:** Pollutant removal performance improves and becomes more consistent with increasing wetland:watershed ratio (Schueler, 1992, Strecker et al., 1992). As a rule of thumb, shallow marsh systems should be at least 2% of contributing watershed and the other designs should be at least 1%.
3. **Allocation of Depth Zones:** Allocation of depth- surface area is an important design feature. Designs should provide a variety of microtopography to encourage species diversity and to sustain sheet flow through emergent plant marsh zones. Each of the designs has different allocations of the following components:
 - *pools* (forebays, outlet micropools, wet pond pools) should be 0.45 to 2 m deep and typically support limited vegetation.

- *hi marsh* should be 0 - 0.15 m deep to encourage dense emergent wetland vegetation.
 - *lo marsh* should be 0.15 - 0.45 m deep and will support emergent vegetation that is typically less diverse than the *hi marsh*.
 - semi-wet zones of ED designs can permit temporary water rise by 1 m.
4. **Allocation of Zones by Volume:** Allocation of depth-treatment volume differs between the basic designs with volume apportionment designed to maximize removal efficiency.
 5. **Length to Width Relationships:** Flow path length improves treatment thus higher L:W designs will increase marsh contact. Designs must, however, consider scouring velocities in the limit. Dry weather flow paths can be enhanced by placing sections of *hi marsh* in the direct flow line. These act as submerged berms and distribute flow throughout the system. Minimum dry weather flow path L:W is suggested as 2:1.
 6. **Base Flow:** Designs 1 through 3 require adequate dry weather flow to sustain water elevations and wetland plants. The rule of thumb base flow requirement is $130 \text{ m}^3/\text{ha}\cdot\text{day}$ ($0.002 \text{ ft}^3/\text{acre}\cdot\text{s}$).
 7. **Extended Detention Volumes:** Extended detention volume should not be more than 50% of V_t . Detention time should be designed for 12 to 24 hours and maximum ED water elevation should not be more than one metre above normal water surface levels.

Table 5.15: Design Factors for Stormwater Wetlands
(after Schueler, 1992)

Sizing Criteria	Shallow Marsh Design #1	Pond-Wetland Design #2	ED Wetland Design #3	Pocket Wetland Design #4
Treatment Volume (Vt)	----- Capture 90% of runoff volume from contributing watershed -----			
Wetland:Watershed Ratio	0.02 : 1	0.01 : 1	0.01 : 1	0.01 : 1
Surface Area Allocation				
- % forebay	5	0	5	0
- % micropool	5	5	5	0
- % deep pool	5	40	0	5
- % lo marsh (15-45 cm)	40	25	40	50
- % hi marsh (0-15 cm)	40	25	40	40
- semi-wet	5	5	10	5
Treatment Volume				
- % forebay	10	0	10	0
- % micropool	10	10	10	0
- % deep pool	10	60	--	20
- % lo marsh (15-45 cm)	45	20	20	55
- % hi marsh (0-15 cm)	25	10	10	25
- semi-wet	0	0	50	0
Flow Path				
- min. L:W	1 : 1	1 : 1	1 : 1	N/A
- dry weather L:W	2 : 1	2 : 1	2 : 1	2 : 1
Water Balance	----- base flow > 0.002 cfs/acre -----			**
Extended Detention	N/A	N/A	EDv = 50% Vt ED rise < 1 m	N/A
Min. Drainage Area (ha)	10	10	4	0.4 < area < 4
Forebay	required	no	required	optional
Outlet Micropool	required	required	required	optional
Outlet Type	rev. slope pipe or broad crest weir	rev. slope pipe or broad crest weir	rev. slope pipe or broad crest weir	broad crest weir
Cleanout Frequency (yrs)	2-5	10	2-5	10
Buffer Zone (m)	8-15	8-15	8-15	0-8
Propagation Technique	mulch/transplant	mulch/transplant	mulch/transplant	volunteer
Pondscaping Emphasis	wildlife habitat marsh topography buffer	wildlife habitat hi marsh	ED zone stability pondscaping zones	pondscaping optional
** typically sustained by excavation to water table.				

Properly designed stormwater wetlands are among the most effective BMPs. Effectiveness of pocket wetlands can be expected to be lower than the other three designs due to resuspension of sediments exposed to drying between storm events (Schueler, 1992). Increasing retention volume or detention time, high surface area, constant pool elevations, sediment forebays and range of wetland microtopography all contribute to improved performance (Schueler et al., 1992). Lower contaminant removals are reported

during non-growing seasons, periods of ice cover and snowmelt runoff. This can limit the effectiveness of wetland BMPs in areas where much of the annual precipitation is received during the winter such as Puget Sound (Washington DOE, 1992). Effectiveness is also reduced by concentrated inflows (Schueler et al., 1992). Multiple pond systems (Designs #2 and #3) provide increased redundancy, flexibility and complexity, thus are more effective than single BMP type installations (Schueler et al., 1992).

Puget Sound guidelines offer alternative design guidelines as shown in Table 5.16 with calculations based on the following relationship (Washington DOE, 1992):

$$A_s = V_t / d$$

where A_s = wetland surface area (m^2)
 V_t = permanent pool volume = runoff from design storm (m^3)
 (suggested as the 6-month, 24-hour event)
 d = average permanent pool depth (m)
 = 0.6 m (maximum)

Table 5.16: Puget Sound Stormwater Wetland Guidelines
 (after Washington DOE, 1992)

Pool Depth m	Surface Area Allocation
0 - 0.15	50%
0.15 - 0.30	15%
0.60 - 0.90	15%
0.90 - 1.80	20%
Maximum Average BMP Depth	0.62 m
Length to Width Ratio	> 3:1 5:1 preferred
Liner recommended to sustain permanent pool and to prevent ground water contamination.	
Sediment forebay recommended with depth of 0.9 m and up to 25% of normal pool area.	

Results of runoff wetland performance evaluations show a wide range of effectiveness. Inconsistent design practices, limited duration of tests, variation in water sampling and analysis procedures and alternative methods of calculating removal

"efficiency" all serve to contribute to the wide range of results (Strecker et al., 1992).

Runoff wetland performance information is summarized in Table 5.17.

Table 5.17: Performance of Stormwater Wetlands

Performance	(all values expressed as percent removed)				
	BC Env., 1992 Design Manuals	BC Env., 1992 Field Studies	Schueler, 1992 Projected Perf.	Strecker et al., 1992 < 2% Watershed	Strecker et al., 1992 > 2% Watershed
- TSS	80-100	64-99	75	69.3	78
- COD	60-80	54-89	15	--	--
- Pb	60-80	88-97	75	--	--
- Zn	60-80	33-96	50	--	--
- Total P	40-60	0-97	45	46.5	64
- Total N	40-60	0-95	25	--	--
- Bacteria	--	--	2 log reduction	--	--
				Avg. of 4 sites	Avg. of 6 sites

Schueler (1987) does not include extensive discussion of stormwater wetlands in the "Washington Green Book" and consequently relative cost correlations consistent with those for other BMPs cannot be reported. Schueler et al. (1992) suggests that the shallow marsh system (Design #1) will be marginally more costly than wet ponds with annual maintenance expense estimated at 3% - 5% of construction costs. A greater maintenance burden can be expected over the first three years until the marsh is established (Schueler et al., 1992). Multiple pond systems can be expected to be "incrementally" more expensive than conventional systems due to more complex design (Schueler et al., 1992). Notably, wetlands and wetland enhanced BMPs retain moisture within the watershed and are planted with native vegetation. Competing stormwater management techniques such as dry ponds are often covered with turf that requires mowing and expensive irrigation systems. Elimination of the irrigation system and substitution of low maintenance naturalized vegetation in wetland type BMPs may actually reduce costs in the Calgary area relative to conventional stormwater management approaches.

5.6 Integration of Application Design Approaches

Design procedures for many of the applications appear to have evolved within a relatively narrow context. Much of the literature is organized in a manner that precludes reference to design procedures other than those for the application in question. This tends to hinder consideration of hybrid systems or adaptation of design procedures. The key purpose of this report is to place the basics of a variety of natural wastewater treatment processes into one resource.

It is probable that no one technology will be adequate for a given situation and that a combination of processes will be required. Even basic septic systems represent a combination of techniques, for example. The septic tank provides sedimentation and anaerobic digestion while the field applies principles of infiltration treatment processes. The hypothetical case study reviewed in Section 7 looks at integration of septic tanks, lagoons, FWS wetlands and VSB processes to provide an integrated sewage and stormwater treatment system.

The potential combinations are considerable. Table 5.18 summarizes the applications discussed in this section along with the more commonly used processes. The table provides suggestions on how approaches from other applications could be used to address specific issues.

Table 5.18 Summary of Natural Water Treatment Processes

Application	Primary Technologies	Supplementary Technologies
Sewage Treatment BOD, nutrient removal designs Hydraulic loading design Proven in cold climates	Lagoon, OF, FWS, VSB Septic fields, sand filters, SR, RI Lagoons, septic fields, septic tank-sand filter, lagoon-FWS, lagoon-OF	Acid mine drainage approaches may be useful for metals or pH control. Agricultural systems provide examples of combined processes.
Agricultural Systems Confined livestock wastes Nonpoint source agricultural water pollution	Lagoon + marsh-pond-marsh wetland Meadow-marsh-pond meadow wetland	Use sewage treatment correlations for lagoons and FWS wetlands as alternative to loading factors. Use stormwater wetland procedures to size runoff system water bodies. Use OF correlations to evaluate meadow components.
Industrial Wastewaters Pulp Mill effluent Petroleum production/processing	Lagoon or mechanical primary/secondary with FWS tertiary. Suitable primary/secondary with FWS tertiary. API separator pretreatment and lagoon cells in series. Aerated lagoons. FWS wetlands for polishing and site runoff control.	Use sewage treatment correlations for BOD & nutrients. Use sewage correlations for BOD & nutrients. Consider sand filters or VSB alternatives if space limited. AMD approaches could be considered for metals or pH problems. Use small watershed procedures for evaluating site runoff storage, sedimentation pond and treatment needs.
Acid Mine Drainage (AMD) Net alkaline water Net acid water - O ₂ , Fe & Al acceptable Net acid water -high O ₂ , Fe or Al	Settling pond + aerobic FWS wetland. Anoxic limestone drain + pond + -aerobic FWS (lower acidity) -compost wetland (high acidity) Anoxic limestone drain + pond + compost wetland	Use stormwater system procedures to estimate runoff treatment volumes and design sedimentation features. Peat or sand filters with replaceable media may provide for adsorption of contaminants.

Table 5-18 (continued)

Application	Primary Technologies	Supplementary Technologies
Landfill/Contaminated Site Leachates Landfill leachate Leachate or runoff from contaminated sites	Collection ponds + VSB or FWS wetlands Site specific but expected to be similar to landfill projects. Pretreatment with API separator where free-phase hydrocarbons are present. May require conventional recovery processes (e.g., stripping columns or activated carbon adsorbers) upstream of biological systems or open water for very toxic materials.	Use sewage correlations for BOD and nutrients. Use AMD procedures for metals and pH problems. Adsorption beds built like sand or peat filters may address certain toxic compounds. Stormwater wetland designs procedures may be useful for managing runoff.
Stormwater Management Detention Water Quality Improvement	Dry ponds, wet ponds, pond-marsh wetlands, ED wetlands with volumes based on flood prevention. Wet ponds, marsh wetlands, pond-marsh wetlands, ED wetlands, pocket wetlands, filter strips, grassed swales, sediment ponds. Treatment volumes based on capturing design fraction of runoff (90%).	Consider adding wetland features to dry or wet ponds to enhance pollutant removal. For specific BOD, nutrient or pathogen reduction use sewage treatment correlations for FWS wetlands to check treatment volumes determined by hydrologic analysis. For metals or pH problems consider AMD procedures.

6. WETLANDS VEGETATION DESIGN CONSIDERATIONS

Vegetation plays a major role in the treatment effectiveness of wetlands, their function as habitat and their value as aesthetic amenities. Vegetation must be selected to not only perform needed treatment functions but must also suit both macro and micro conditions of the development site. In general the first rule of selection is observation. That is, whatever grows locally in conditions similar to those to be imposed on a constructed wetland, will have a good chance of being suited to the project in question. This does not rule out introduced species and cultivars that will thrive in the system and that are appropriate to the landscape design and ecology of the wetland.

While direct observation of successful local wetland species is essential to constructed wetland vegetation design, it is recognized that discussion of basic concepts and identification of a short list of plants expected to be appropriate to northern prairie climates could be useful. This section will outline the general wetland vegetation zones, list a few representative plants expected to thrive under western Canadian conditions, outline basic propagation techniques used for constructed wetlands and overview basic construction and management factors.

6.1 Pondscaping Zones and Typical Vegetation Types

Previous sections of this document have suggested specific wetland plantings to optimize function, however it is desirable and even necessary (in the case of extended detention stormwater systems) to design a transition of vegetation from deep water zones to upland site uses. Monotypic stands of bulrush or cattails can be used in specific wastewater designs and, if aesthetics or wildlife habitat are not concerns, little more vegetation planning is essential. Monotypic stands are favored by and even require stable hydrologic and climatic environments. However, single species wetlands are potentially vulnerable to disease, pests and environmental variability. It is probable that aesthetics and wildlife habitat will be desirable features for many applications of the ecotechnology. Environmental variability should be expected in the Prairies and biotic diversity will

improve the resiliency of constructed wetlands. In the case of stormwater wetlands, especially those that provide some degree of extended detention, a diversity of vegetation is essential for long term stability of the system.

Vegetation selection is not only based on wetland function and local climate but also on expected microsite hydrology and the variability of the hydrological environment within the wetland. Figure 6.1 illustrates pondscaping zones that must be considered in constructed wetland design, as they are defined by water level.

Figure 6.1: Constructed Wetland Pondscaping Zones
(adapted from Marburger, 1992, Schueler, 1992 and Washington DOE, 1992)

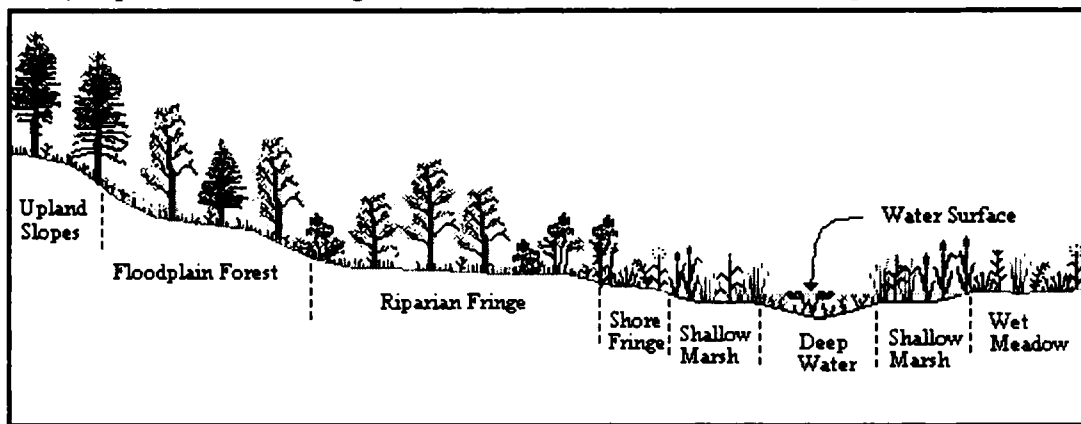


Figure 6.1 is a simplification of the effect of hydrology. It illustrates zones as largely defined by stable water level. Detailed design of a constructed wetland must define the variability of water level and the duration and timing of various inundation levels. This information is critical to the selection of plants that can tolerate and prosper in the proposed hydrological environment. For example, wooded bottomlands may survive winter inundation for several weeks but be damaged by more than 3 to 5 days flooding in the spring or early summer. Cattail or reed marshes can tolerate additional 0.2 - 0.5 m water depths for just a few weeks during the growing season before changes in species composition will occur (Hammer, 1991). Tolerance of a variety of wetland plants to flooding is presented in Table 6.1.

Water quality parameters affect the selection and viability of aquatic vegetation. Water clarity is critical to submerged water species and only rooted floating plants will survive in turbid water too deep for emergent plants. Runoff from saline soils and conditions in alkali wetlands require planting of tolerant species which include adapted forms of bulrush (*Scirpus acutus* and *S. fluviatilis*), cordgrass (*Spartina sp.*) and spikerush (*Eleocharis sp.*) provided the wetlands are moderate to high in pH. Low pH bog conditions result in low nutrient status substrate and dark colored waters. Adapted forms of cattail (*Typha latifolia*), grasses (*Calamagrostis sp.*, *Glyceria sp.* and *Deschampsia sp.*) and mosses can survive in the acid bog environment (Hammer, 1991). Table 6.2 provides examples of plant types and related environmental requirements. The listed plants were verified in Budd (1979) as resident in wetland areas of the Prairie Provinces.

According to Hammer (1991), most wetland plants tolerate a wide variety of soil types and nutrient ranges. Sandy loam soils foster rhizome and root penetration which is restricted by heavy clay soils. As plantings are typically widely spaced to minimize costs, development of plant cover depends on spreading of rhizomes and the development of new shoots. Thus, planting densities may have to be increased if site conditions and project plans do not provide for placement of suitable soils. Table 6.2 provides soil requirements of typical wetland vegetation.

The plants listed in Table 6.1 are not inclusive of all plants suitable for wetland conditions in western Canada. Common shrubs not listed but found in moist soils or along stream banks include buffaloberry (*Shepherdia canadensis*), chokecherry (*Prunus virginiana*) and red elderberry (*Sambucus pubens*). Such shrubs, to name a few, could potentially be useful and hardy for occasionally inundated sections or upper banks of constructed wetlands.

Table 6.1: Inundation Tolerance of Typical Wetland Plants
(adapted from tables in Hammer, 1991 with plants cross-referenced to Budd, 1979)

Scientific Name	Common Name	Scientific Name	Common Name
Herbaceous Plants		Herbaceous Plants	
<u>Seasonally Flooded</u>		<u>50 to 200 cm Water Depths</u>	
<i>Alopecurus spp.</i>	foxtail	<i>Callitriche spp.</i>	water starwort (not common)
<i>Bidens spp.</i>	beggarticks	<i>Ceratophyllum demersum</i>	coontail
<i>Distichis stricta</i>	alkali grass	<i>Nuphar spp.</i>	spatterdock
<i>Echinochloa crusgalli</i>	barnyard grass	<i>Potamogeton pectinatus</i>	sago pondweed
<i>Hordeum jubatum</i>	foxtail barley	<i>Ranunculus aquatilis</i>	white water buttercup
<i>Panicum virgatum</i>	switchgrass	<i>Ruppia maritima</i>	wigeongrass (not common)
<i>Polygonum lapathifolium</i>	pale smartweed	<i>Scirpus validus</i>	bulrush
<i>Scolochloa festucacea</i>	marshgrass	<u>Floating Plants</u>	
<i>Setaria spp.</i>	foxtail grass	<i>Lemna spp.</i>	duckweed
<i>Spartina pectinata</i>	prairie cordgrass	<i>Spirodela spp.</i>	giant duckweed
Herbaceous Plants		Woody Plants	
<u>Seasonally/Permanently Flooded to 15 cm</u>		<u>Flooding Tolerant for 1 Growing Season</u>	
<i>Beckmannia syzigachne</i>	sloughgrass	<i>Kalmia polifolia</i>	bog laurel
<i>Calamagrostis inexpansa</i>	reedgrass	<i>Larix laricina</i>	tamarack
<i>Caltha spp.</i>	marsh marigold	<i>Ledum groenlandicum</i>	Labrador tea
<i>Carex spp.</i>	sedge	<i>Picea mariana</i>	black spruce
<i>Cyperus spp.</i>	nut-grass (rare)	<i>Populus balsamifera</i>	balsam poplar
<i>Cypripedium spp.</i>	lady's slipper (rare)	<i>Populus deltoides</i>	cottonwood
<i>Eriophorum spp.</i>	cotton grass (Boreal Forest)	<i>Salix spp.</i>	willows
<i>Habenaria spp.</i>	swamp orchids	<i>Vaccinium uliginosium</i>	bog whortleberry
<i>Iris pseudacorus</i>	yellow iris	Woody Plants	
<i>Isoetes spp.</i>	quillwort (rare)	<u>Flooding Tolerant for less than 30 Days per Growing Season</u>	
<i>Juncus spp.</i>	rush	<i>Acer negundo</i>	Manitoba maple
<i>Leersia oryzoides</i>	rice cutgrass (eastern areas)	<i>Acer saccharinum</i>	silver maple (eastern areas)
<i>Phalaris arundinacea</i>	reed canarygrass	<i>Alnus incana</i>	speckled alder
<i>Polygonum coccineum</i>	swamp smartweed	<i>Betula pumila</i>	swamp birch
<i>Polygonum lapathifolium</i>	pale persicaria	<i>Cornus stolonifera</i>	red-osier dogwood
<i>Sarracenia spp.</i>	pitcherplant (not common)	<i>Crateagus spp.</i>	hawthorn
Herbaceous Plants		<i>Quercus macrocarpa</i>	bur oak (eastern Parkland)
<u>15 to 50 cm Water Depths</u>		<i>Rhus radicans</i>	poison ivy
<i>Acorus calamus</i>	sweet flag	<i>Smilax spp.</i>	carrion flower
<i>Alisma spp.</i>	water plantain	<i>Tilia americana</i>	basswood (SE prairies)
<i>Calla palustris</i>	water arum	<i>Ulmus americana</i>	American elm
<i>Eleocharis spp.</i>	spikerush	<i>Vitis riparia</i>	wild grape (eastern areas)
<i>Glyceria spp.</i>	mannagrass	Woody Plants	
<i>Menyanthes trifoliata</i>	buck bean	<u>Flooding Tolerant for less than 5 Days per Growing Season</u>	
<i>Polygonum amphibium</i>	water smartweed	<i>Betula papyrifera</i>	paper birch
<i>Sagittaria cuneata</i>	arrowhead	<i>Populus tremuloides</i>	aspen
<i>Scirpus americanus</i>	three-square		
<i>Scirpus fluviatilis</i>	river bulrush		
<i>Sparganium spp.</i>	burreed		
<i>Typha latifolia</i>	wide-leaved cattail		
<i>Zizania aquatica</i>	wild rice		

Table 6.2: Growing Environments for Typical Wetland Plants
(source: Hammer, 1991)

Plant Genus	Common Name	Preferred Environmental Conditions			
		Soil Type	pH Range	Salinity ppt	Growing Zone relative to water surface cm
<i>Calamagrostis</i>	reedgrass	silt	5 - 8	0 - 10	+20 to -20
<i>Carex</i>	sedges	organic, clay	5 - 7.5	0 - 0.4	+10 to -50
<i>Juncus</i>	rushes	organic	5 - 8	0 - 30	+10 to -10
<i>Nuphar</i>	pond-lily	organic, silt	3 - 8	0 - 5	-50 to -200
<i>Phalaris</i>	reed canary	silt loam	6 - 7.5	0 - 0.4	+10 to -10
<i>Phragmites communis</i>	reeds	silt, sand, clay, organic	3 - 8	0 - 30	+30 to -150
<i>Potamogeton</i>	pondweeds	organic, silt clay	4 - 10	0 - 20	-5 to -300
<i>Ruppia</i>	ditchgrass	silt clay, organic	5 - 10	0 - 25	-5 to -100
<i>Scirpus</i>	bulrushes	organic, clay	4 - 9	0 - 32	+10 to -100
<i>Spartina</i>	cordgrass	sandy loam	5 - 8	10 - 30	+20 to -50
<i>Typha</i>	cattails	organic, silt clay	5 -	0 - 25	+10 to -70

6.2 Deep Water Zone Pondscaping

Constructed wetland deep water zones include sediment forebays, outlet micropools, channels and other features deeper than 45 cm. These sections support only submerged and floating vegetation (Schueler, 1992).

Deep water sections are not only important for water treatment but also for wildlife habitat. Submerged aquatic vegetation are preferred food for water fowl (Scheuler, 1992). Large sections of open deep water should be provided if the project is intended to attract ducks and geese. Conversely, careful control of the size of deep sections can discourage excessive waterfowl populations which can exacerbate water quality problems.

Submerged plants can be allowed to establish naturally or be artificially introduced by seed or vegetative means. In new wetlands constructed of terrestrial soils, the influx of seeds and consequent establishment of plants will be slow. Viability of wetland plant seeds is typically low, germination properties can be unpredictable and new plants are vulnerable to herbivorous birds and animals (Hammer, 1991). Seeding of

submerged plants, as well as placing of weighted rhizomes has not been effective (Schueler, 1992). Hand planting of tubers, root stocks or whole plants is the most successful approach with 0.75 to 1.5 m spacing typically used for herbaceous wetland plants (Hammer, 1991). Root stocks can be planted in early spring or following autumn dormancy with growing plants placed in May to June. One growing season of careful management is typically required before the constructed wetland can be expected to assume its full treatment role. It is recommended that at least 5 to 10 species be planted in deep water zones (Hammer, 1991).

Water level control during the plant establishment phase is critical. Most submergent plants require wet soil conditions but not inundation for germination and initial growth followed by gradually increasing water levels to support vertical development. Flooding should be sufficient to avoid exposure and drying but shallow enough to permit oxygen diffusion and adequate light. Turbid or waste waters should be avoided during initial plant establishment to minimize stress on the plants during this critical period (Hammer, 1991).

6.3 Shallow Marsh Area Pondscaping

Shallow marsh zones are dominated by emergent herbaceous vegetation and include the 0 to 15 cm deep “hi marsh” and 15 to 45 cm deep “lo marsh” sections of constructed stormwater wetlands (Schueler, 1992). According to Schueler (1992) the shallow marsh zones are critical for:

- nutrient uptake and removal processes
- sedimentation
- stabilization of settled solids to prevent resuspension
- food resource and cover for wildlife
- habitat for mosquito controlling predators
- control of shore erosion

- wetland aesthetics including concealment of level fluctuations and trash

Microtopography of the shallow marsh zones is important for flow management and for sustaining biotic diversity. The placement of hi and lo marsh sections can provide for flow redistribution in the wetland to avoid channeling and bypassed zones. During periods of low flow or levels, a sinuous lo marsh path between inlet and outlet sections can provide a high L:W channel to ensure thorough treatment of water. Based on plants listed in Table 6.2, the range of water depths between 0 and 45 cm will provide conditions for a variety of herbaceous vegetation. This helps to avoid monotypic stands which offer lower habitat value, limited aesthetics and which can be susceptible to diseases, insects or changes in hydrologic conditions of the wetland.

Although most substrates are acceptable for establishing wetland plants, fertile loam to sandy loam soils are preferred. Nutrients may be added if necessary and it is noted in Hammer (1991) that dewatering apparently deficient wetlands soils may make nutrients available through oxidation of chemical species immobilized by the normally reducing conditions of the substrates. It is recommended, however, that conserved wetland soils should be stored underwater to maintain reduced conditions. Exposure to air could oxidize metals and other constituents which may adversely affect the new wetland when the oxidized forms dissolve on flooding.

Planting of the shallow marsh sections should follow guidelines discussed for submerged plants. Planting of root stocks or whole plants typically gives the most consistent results (Hammer, 1991), albeit at relatively high cost. Where seedlings or rootsalks are used, Schueler (1992) recommended that about half the wetland area be planned with “clumps” of single species plantings where individual stocks are placed on 45 cm centers. He suggested 5 to 7 species with at least three aggressive colonizers such as *Sagittaria spp.*, *Scirpus validus* and *Polygonum spp.* (*Typha latifolia* is noted as a very invasive wetland plant that may tend to monotypic stands). Hammer (1991) recommended shallow zones be planted with 10 to 15 species on 75 to 100 cm spacing in rows transversing the narrow axis of the wetland (e.g. rows perpendicular to water flow).

Schueler (1992) noted that 7 to 15 cm of wetland soil mulch will provide a diversity of propagules that will develop into vigorous cover in one season. The mulch should be obtained from sites to be affected by construction or maintenance (e.g., materials removed during highway ditch maintenance) and not from natural wetlands. The mulching technique, while not necessarily producing as consistent results as planting root stocks, is expected to be less costly.

Vegetation selection, propagule type and post planting management must consider environmental and wildlife stresses on the system. Initial vegetation selection should consider exposure of the constructed wetland to wind, direct solar radiation and winter conditions. In early years sheltering effects of shoreline shrubs and forest may be limited, leaving the wetland quite exposed (Schueler, 1992). It may be necessary to protect young shoots from herbivore damage with wire netting as nursery seedlings and newly germinated sprouts are especially attractive and vulnerable to certain types of waterfowl.

Ensuring that design water depths are maintained is critical for establishment of shallow marsh sections. These elements typically comprise the majority of the surface area of constructed wetlands. Both Hammer (1991) and Scheuler (1992) recommend newly graded constructed wetlands be flooded prior to planting. This permits settling of the soil and verification of grades, water depths and appropriateness of level control structures. Settling of 15 to 30 cm is possible, yet such variations from planned elevations will not only affect survival of selected plants but will also alter flow distribution in the wetland. Final vegetation plans may not be developed until after one year's flooding of stormwater systems to enable the designer to define depths and inundation frequencies (Schueler, 1992).

Water level control is also important following planting. The marsh section should be initially flooded, allowed to stand and then dewatered for planting of emergent marsh vegetation in saturated (not flooded) substrate. When stems and leaves are 5 to 10 cm high, flooding to 2 to 3 cm can be permitted and is often desirable to control invasion of terrestrial weeds. Although water levels can be gradually increased to operating levels

as vegetation grows (new shoots must not be over-topped), it is recommended that levels be maintained at 2 to 3 cm for the first season (Hammer, 1991).

6.4 Shoreline Fringe and Wet Meadow Pondscaping

6.4.1 Shoreline Fringe

The shoreline fringe spans the zone between normal water level and the upper limit of extended detention levels in stormwater wetlands or about 30 cm above stable water level in other systems. In stormwater applications, this zone is a harsh environment for plants which can be flooded for 24 hours many times per year and can also be subjected to long periods of drought (Schueler, 1992). The zone fulfills several key functions:

- stabilizes wetland shoreline
- improves wetland aesthetics
- provides passive barrier to water access (safety feature)
- provides food, cover and nesting habitat for wildlife
- shades pools to reduce warming and helps shelter wetland from wind

This zone can accommodate both herbaceous and woody plants. Herbaceous species listed under “seasonally flooded” and “seasonally/permanently flooded” in Table 6.2 should be suitable for this zone. It is expected that species of *Carex* (sedge), *Scirpus* (bulrush) and *Juncus* (rush), as well as *Phalaris arundinacea* (although reed canarygrass is potentially invasive) will be well suited to this zone. Early ground cover can be obtained from commercial or native grass seed mixes. Additionally woody plants such as *Cornus stolonifera*, *Salix spp.* and *Betula pumila* will provide taller cover, shade and visual value to the wetland zone.

Initial cover for the shoreline fringe, as well as the wet meadow, can be established by hydroseeding or scarifying soil and broadcast seeding a suitable mix of “wet” grass mixes (Schueler, 1992). Other vegetation can be established by transplanting nursery stock or materials from donor sites. Especially during the first year, sufficient

moisture must be supplied preferably by intermittent flooding to less than 1 cm followed by drying periods (Hammer, 1991)

6.4.2 Wet Meadow

Wet meadows can be part of the shoreline fringe where slopes are very shallow but may also form stand alone elements of runoff and wastewater treatment systems. Wet meadows can act as buffer strips to filter runoff, provide surface area for decomposition of wastewater contaminants and can be used to filter algae from pond or wetland effluent streams. It is also expected that a wet meadow of diverse and interesting vegetation could provide a low maintenance and aesthetically pleasing alternative to conventional dry pond covers of manicured turf. Wet meadow designs can be used to surround stormwater wetlands where substantial extended detention volume is needed.

In water treatment applications, the meadows are exposed to intermittent shallow sheet flow with natural or controlled drying periods. Plants must not only tolerate shallow flooding and saturated soils but also extended dry periods. Where extended dry periods are expected, Hammer (1991) suggested grasses (*Agrostis*, *Alopecurus*, *Festuca*, *Poa*, and *Spartina*) and spikerushes (*Eleocharis*). It is expected that grasses such as *Calamagrostis canadensis*, *Phalaris arundinacea* and *Agropyron trachycaulum* could also be used. Meadow vegetation including *Carex*, *Glyceria* and *Leersia* should not be allowed to be completely dry during the first season. Others such as *Cyperus*, *Juncus*, *Polygonum* and *Scirpus* can tolerate inundation (2 to 3 cm) but must have adequate soil moisture maintained (Hammer, 1991).

Vegetation can be established in similar manner as described for marsh and shoreline fringe zones. This includes depending on the resident seed bank in soil mulch from a donor site, hydroseeding of wet grass mixes and hand planting of seedlings, root stocks or whole plants. Adequate moisture must be maintained during the first growing season as per the procedure suggested for shoreline fringe zones.

6.5 Riparian Fringe Pondscaping

The riparian fringe is infrequently inundated and typically extends from 30 to 90 cm above normal wetland water levels (Schueler, 1992). Objectives listed by Schueler (1992) for pondscaping this zone include:

- provide lower portion of stormwater wetland forest buffer
- provide wildlife structural habitat and food resources
- bind sediments from large events
- stabilize wetland slopes and control erosion by dense ground cover

As riparian fringe vegetation includes shrubs and large trees, this zone is an important component of integrating the wetland visually into the surrounding landscape. Use of closely spaced, thick shrubs in this zone can provide passive access control to free water surface areas which reduces public liability of such systems installed in area frequented by children. The passive safety feature is an important aesthetic alternative to the barb-wire topped chain-link fences that typically surround stormwater wet ponds.

Vegetation chosen for the riparian fringe must tolerate infrequent inundation, as well as extended dry periods. It is expected that vegetation typical of the cottonwood plant communities along Prairie streams and rivers will be successful in this application. Trees such as *Populus deltoides* and *P. balsamifera* along with shrubs like *Betula pumila*, *Cornus stolonifera*, *Prunus virginiana* and *Salix* can be planted from nursery stocks. Ground cover consisting of grasses suggested for drier sections of the wet meadow can be hydroseeded to protect the zone until the woody vegetation develops. Attention should be paid to the selection of ground cover, however, as the zone will likely be quite exposed for several years until the trees and shrubs develop significant size.

6.6 Floodplain Terrace Pondscaping

The floodplain terrace is only rarely inundated and extends 1 to 2 m above the normal pool level. The zone provides the majority of the buffer between surrounding land uses and the wetland (Schueler, 1992). The zone also provides opportunity to

enhance the aesthetics of the wetland system and to provide recreation paths and viewpoints for observing wildlife (especially birds) in the marsh.

Similar to the initial cover for the riparian fringe, grass mixes chosen for occasional inundation should be established to control erosion. Trees and shrubs need less flooding tolerance and examples are listed in Table 6.2. As in the riparian fringe zone, it may take 10 or more years for woody vegetation to develop significant size, thus system design must provide for early ground cover that can survive on relatively exposed sites during dry periods yet tolerate some degree of flooding.

6.7 Upland Zone

The upland zone provides a visual and land use buffer to the wetland complex. This zone is not expected to be inundated, thus vegetation appropriate to the soils and climate of the surrounding area can be used (Schueler, 1992).

Consideration should be given to potential uses of the upland in design of the wetland. Many stormwater systems will likely be within or border urban areas and present potential value as parkland. It is possible to design the upland as manicured park (e.g., mowed turf with selected trees) or as a naturalized forest or grassland. The former can accommodate athletic fields whereas the latter favors paths and passive nature-centered recreation.

6.8 Other Design Considerations

6.8.1 Multiple Use Design

Aesthetic design of wetland systems will greatly enhance their acceptance and ultimate value to the community. Design teams must not only include wastewater, hydrological and civil engineering experts but also biologists and landscape architects. While the “technical” objective of flood control or water quality improvement may be the motivating reason for a constructed wetland, its value for wildlife habitat, nature-centered recreation and for the inherent beauty of diverse life should not be overlooked.

Attractive naturalized areas can be perceived as amenities and may improve land values. This creates developer “pull” to apply stormwater treatment technology for good business reasons.

The ancillary benefits of constructed wetland systems will depend on the application. Domestic wastewater systems will require access controls and significant setback from occupied buildings in many jurisdictions. In such applications, the wetlands may be monotypic plantings in relatively “engineered” basin forms. The surrounding vegetation zones can be designed to disguise the operation and to attract or control wildlife. In the western prairies, surface waters are often rare commodities, thus stormwater systems present an opportunity to create and sustain plant and wildlife communities atypical of the urban or grassland surrounding landscapes. Such surface stormwater facilities can be designed into a linear park systems that provide pedestrian pathways and wildlife corridors within city environs. These ancillary benefits can only be achieved by holistic design processes involving interdisciplinary teams of biologists, engineers, urban planners and landscape architects.

6.8.2 Site Preparation

Control of infiltration, design of grading and placement of soils and amendments are critical aspects of constructed wetland establishment. Uncontrolled infiltration may prevent maintenance of stable water levels and introduce contaminants into groundwater resources. Slight deviations in bed slope can result in sections that are too deep for desired vegetation, short circuiting and bypassing of treatment zones. Lack of suitable topsoil and substrate may preclude establishment of healthy plant communities.

Most constructed wetlands will require low permeability bases or impermeable liners to control infiltration. Liners prevent wastewater contamination of potable aquifers and minimize losses of standing water in stormwater wetlands. In cases where ample base flow through stormwater systems is expected, the need for liners may be questioned.

Compaction of soils with greater than 10 % clay content can be the simplest and most economical method of sealing the wetland basin (Hammer, 1991). Native subsoils

with hydraulic conductivity less than 10^{-6} or 10^{-7} cm/s may not need liners according to Hammer (1991). Alberta guidelines (Alberta Environment, 1988), however, require that sewage treatment pond compacted clay liners have a minimum thickness, defined by hydraulic conductivity as follows:

$$K_T = (5.2 \times 10^{-9}) T / (2 + T)$$

where T = liner thickness (m)
 K_T = maximum field hydraulic conductivity (m/s) of the liner which must be one order of magnitude greater than laboratory values

Alberta guidelines have been published summarizing design procedures for clay, soil admixture and synthetic liners (Alberta Environment, undated).

Synthetic liners can be an alternative to soil systems. Many synthetics must be protected by at least 10 to 15 cm of soil to prevent UV radiation damage. Rooting substrate placed on the synthetic liners must be at least 40 cm to over 60 cm thick to prevent root penetration and resulting loss of liner integrity (Hammer, 1991).

Dams, berms and dikes require proper engineering design especially if dam heights are greater than 2 m or if catastrophic failure will endanger the safety of those downstream. In general inside berm slopes should be shallower than 3:1, outside slopes not steeper than 4:1 and top width not less than 3 m for vehicle access (Alberta Environment, 1988 and Hammer, 1991).

Water level control structures should include adequate emergency spillways and stop log weirs, reversed sloped drain pipes or swivel elbow arrangements to enable control of water levels. The selection of level control structures must account for susceptibility to plugging and freezing weather conditions. In all but simple small systems, such structures should be designed by a qualified engineer.

Constructed wetland plans should clearly specify elevations of beds and system elements. Competent field supervision must ensure grading meets requirements for overall slope of wetland basins and berms, as well as for the creation of microtopography for hi and lo marsh sections (if applicable) of the system. It is suggested the system be

flooded to verify depths and elevations, as well as to provide for settling prior to accepting final grades. After the initial flooding, regrading or alteration of vegetation plans (or both) may be required (Hammer, 1991, Schueler, 1992).

Proper rooting media for terrestrial and wetland plants will be required. At the start of construction topsoil should be stripped from the site and conserved (in piles not greater than 1 m thick planted with an annual cover such as fall rye). Wetland mulch is the preferred substrate for marsh sections of the system and 8 to 15 cm of topsoil or mulch should be spread over exposed subsoils, especially the area from 30 cm below to 15 cm above normal water level (Schueler, 1992). General planting procedures are discussed in the preceding sections for the specific wetland zones.

6.8.3 Wetland Establishment and Maintenance

Planning of constructed wetland applications must provide adequate time for the system to become established, management of water levels during the first growing season and adequate resources for reinforcement planting.

In general, wetland plants should be planted on moist soils and water levels in the first year managed to avoid drowning the new plants. Planting should occur following autumn senescence or in the early spring. Stresses on new wetlands should be avoided or carefully managed, thus stormwater flooding should be controlled and wastewater streams gradually introduced (Hammer, 1991). It is not reasonable to expect wastewater treatment wetlands to be functional in the first growing season. Herbaceous vegetation can be expected to become established over three years with good cover after the first season. Not all plantings should be expected to be successful and provision should be made to assess zones of poor survival and provide for reinforcement plantings. Provision for inspection of the wetland at least twice per year for the first three years is recommended by Schueler (1992). Establishment of woody vegetation can be expected to take 10 to 20 years depending on the species mix and growing conditions.

Over the longer term, maintenance may involve replacement of winter kill (if needed), sediment removal, control of invasive species and management of excess biomass.

Sediment removal is a primary concern of stormwater wetlands. Sedimentation in the main wetland sections should firstly be managed by adequately sized forebays. The forebays trap coarse sediments and provide for cost effective removal of accumulated sediments. While the forebays may require clean-out every 3 to 5 years (Scheuler, 1992), accumulation of sediments in the main wetland should be slow.

Ongoing management of vegetation may be required to prevent development of monocultures. Manual removal of invasive species (e.g., *Typha*) is reported to provide only temporary relief. Proper design, with a wide range of depth zones and complex wetland structure, is a more reliable means to encourage plant diversity (Scheuler, 1992).

The need to harvest biomass is not clear from the author's review of the literature. Harvesting may result in marginal nutrient removal improvements potentially at the cost of wildlife habitat loss and increase operating expenses (Scheuler, 1992). As discussed in earlier sections, plant detritus provides additional surface area for biofilms and adsorption sites. Conversely, excessive buildup of organic detritus could impair water flow or result in channelization of the wetland. It is expected that vegetation management needs will have to be assessed through early trials of the technology under northern prairie conditions.

It is expected that annual management of wetland water levels will be required in cold climates. Hammer (1991) recommended that water levels be raised in winter to reduce frost penetration into the substrate and that levels be lowered in the spring to avoid over topping new shoots.

6.9 Summary

Selection of wetland vegetation is primarily a function of inundation depth and frequency, however, soil properties and water chemistry must be taken into account. A

variety of plants are suitable for each of the wetland zones (a small number of which have been listed in Tables 6.1 and 6.2) which can be established to create the diversity that improves habitat value, long term ability of the wetland to adapt to changes and enhances the aesthetics of the system.. Grading designs, substrate placement and construction quality are essential to successful projects, as is follow-up management of the wetland, especially in the first three years. This section was intended to provide a simple overview of vegetation issues and Hammer (1991) is suggested as an excellent overall reference on creation of wetlands once they have been sized by design procedures for the specific application.

7. WASTEWATER AND RUNOFF TREATMENT: A DESIGN EXAMPLE

7.1 Overview

This section describes the design of a stormwater and domestic wastewater system for a hypothetical residential development in the Calgary area in order to demonstrate how wetlands wastewater treatment might be applied. The model evaluation also demonstrates how several of the processes discussed in Section 4 can be linked to form an integrated water treating system. By examining stormwater management and sewage treatment systems, the hypothetical case study focuses on the two applications that are expected to be the primary uses of wetlands wastewater treatment in western Canada.

Small rural developments such as private homes, residential schools, retirement villages, institutions and recreation sites require cost effective wastewater treatment systems. Such sites typically cannot afford capital intensive technology and do not contemplate employment of skilled staff to operate complex processes. High initial capital costs and expensive ongoing user fees can be barriers to tying into nearby urban or regional sewage systems. These wastewater handling issues can become barriers to appropriate treatment of wastes or even barriers to new development.

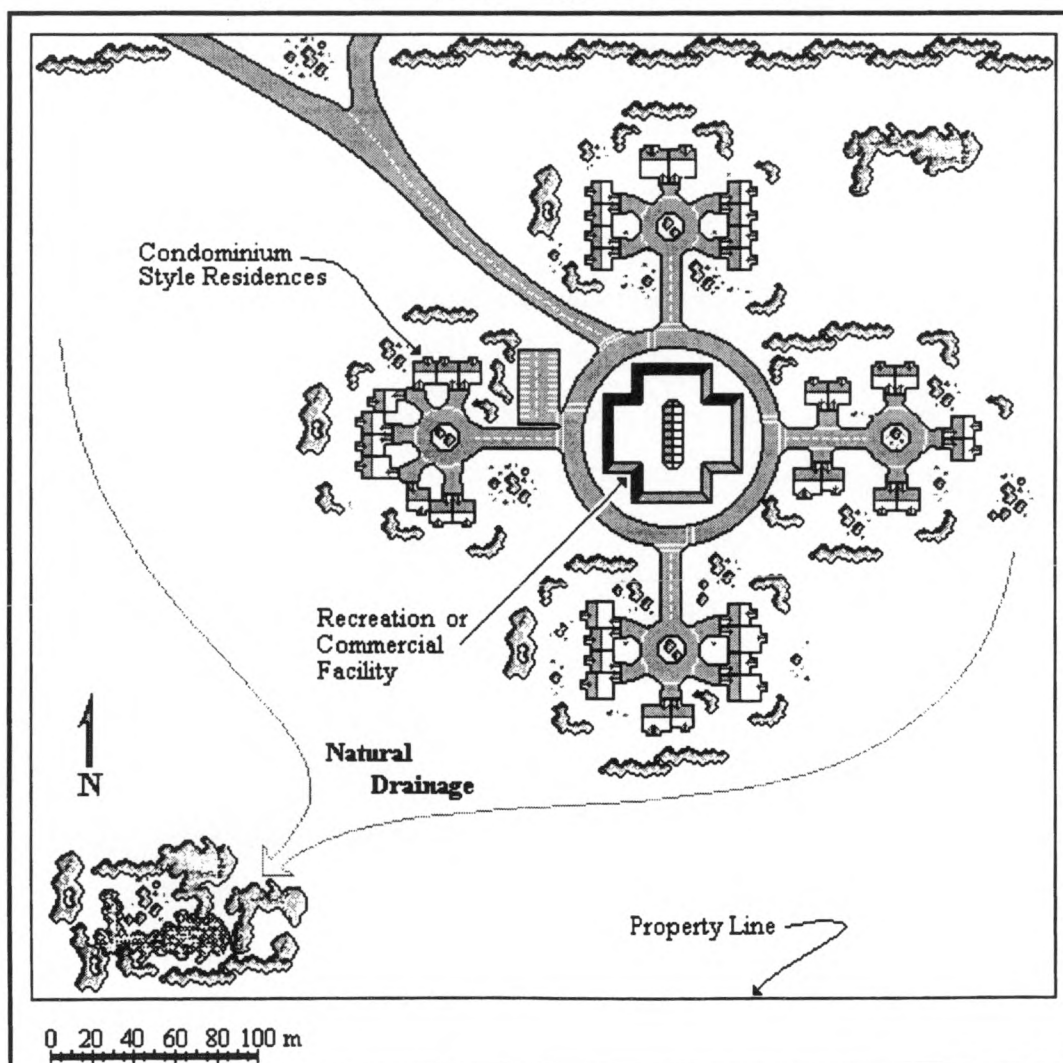
Four independent calculation models (one each for stormwater wetlands, lagoons, FWS system and VSB wetlands) were programmed in Microsoft EXCEL (Ver. 5) for the purpose of this case study. The EXCEL models were generalized to enable testing of the impact of various input parameters and evaluation of future studies. Printouts relative to this study are provided in Appendices A through D.

7.2 Design Basis

7.2.1 Hypothetical Development Description

The hypothetical development is assumed to be home for about 120 residents who, in keeping with less consumptive building styles, will live in two, three or four unit condominium type buildings. It is assumed that occupancy will average three persons per unit for a total of 40 residences. A central complex is included for community events or possibly a small store - laundromat facility. This design basis could broadly fit a variety of seniors facilities or resort complexes. Figure 7.1 illustrates the site size and layout used in the analysis.

Figure 7.1: Layout of Hypothetical Study Site



As the project is rural in nature, it is assumed sufficient lands have been included with the primary intent of providing ample, natural open space for residents. Notwithstanding acquisition of land for green space, it has been the author's experience that unlimited land for wastewater treatment is far from the case. For this reason an overall density of five persons per ha has been assumed making the study site 480 m by 500 m. In keeping with good stormwater quality source controls, roads and other impervious surface will be minimized. Open areas are assumed to be native grassland with occasional tree groves. Recreational developments likely would involve intrusion of a golf course or camping sites on portions of the green spaces and input to the calculation models used in this conceptual design can readily be changed to reflect these alternate land uses.

Such developments often depend on wells for potable water supply. This factor imposes two limitations on infrastructure system design. Firstly, water supply is often limited making it desirable to conserve treated wastewater and storm runoff for irrigation and utility water (e.g., for toilet flushing). Where treated effluent quality is not an issue for off-site disposal, reuse options may drive water quality standards. Secondly, the need for potable well water from the site makes it essential that wastewaters do not contaminate source aquifers. This limits the desirability of on-site disposal by septic field (which in any event does not permit water reuse).

7.2.2 Wastewater Rates and Composition

Alberta guidelines for small wastewater systems were selected as the design basis although a range of flow rate parameters exists. Provincial guidelines for small sewage lagoons set the minimum sizing as $0.25 \text{ m}^3/\text{person} \cdot \text{day}$ (Alberta Environment, 1991). Based on discussions with consultants, this value reflects a moderate level of water conservation including reduced flush toilets and restricted shower heads. As a hypothetical grass roots facility, it is reasonable to expect such features could be designed in at minimal incremental cost.

It is probable such a site will host some form of group gatherings which will impose surges on the wastewater system. Even a small hall could accommodate 100 or more and family parties could involve several dozen people. For the purposes of the hypothetical study it is assumed that the central hall will host one 10-hour event of 100 people per week.

The foregoing factors result in a base domestic wastewater flow of 30 m³/d with one 10-hour surge of 10.4 m³ each week. Annual sewage production totals 11,500 m³ or an average flow of 31.4 m³/d. This flow is less than the 40 m³/d maximum rate suggested for US EPA on-site wetlands wastewater treatment systems design procedures (Reed, 1993) discussed in Section 5.1.2, thus, the technology is appropriate for the design study. Table 7.1 summarizes wastewater quality for various types of effluents used in the hypothetical evaluation. For detailed design, measurements of actual wastewater quality (or that of closely related systems in the area) should be used.

Table 7.1: Wastewater Composition

all values mg/l unless otherwise noted					
Parameter	Domestic Wastewater		Septic Tank Effluent		Lagoon Effluent
	range	assumed	range	assumed	expected
reference	US EPA, 1980		US EPA, 1980		US EPA, 1983
BOD	200 - 290	245	7 - 480	140	30
COD	680 - 730	705	25 - 780	300	--
Suspended Solids	200 - 290	245	10 - 695	80	30
Total Nitrogen	35 - 100	68	9 - 125	40	15
Ammonia	6 - 18	12	--	15	--
Fecal Coliform*	1E+07 - 1E+9	1E+08	--	1E+08	1.5E+05**
* number per 100 ml					
** assumed value (conservative) based on ranges reported in the noted reference					

7.2.3 Effluent Objectives

The treated water quality should be sufficient to minimize constraints on use. Alberta guidelines for surface water quality (Alberta Environment, 1977) and Canadian guidelines (CCME) were reviewed. Table 7.2 summarizes the treatment effluent quality parameters used for design calculations. The parameters meet contact recreation

requirements. Notably, Provincial guidelines for lagoon effluent from projects serving less than 20,000 specify maximum BOD of 25 mg/l (Alberta Environment, 1988).

Table 7.2: Treatment Objectives

Parameter		Treated Water Objectives
BOD	mg/l	10
Suspended Solids	mg/l	10
Total Nitrogen	mg/l	1
Ammonia	mg/l	1
Fecal Coliform	number/ 100 ml	200

7.2.4 Assumed Site and Environmental Conditions

The site, located near Calgary, is assumed to be relatively level with an average slope of 2.5 %. For the purposes of simple runoff evaluations, surface soils are assumed to be hydrologic Group C (sandy clay loam) as classified by the US Soils Conservation Service (US SCS, 1986). It is further assumed the site, other than impervious surfaces (roofs, roads and parking) is covered with healthy grassland. Trees were assumed to have a negligible affect on surface hydrology

Provincial setback guidelines as summarized in Table 7.3 were assumed to apply to surface waters containing primary or secondary domestic wastewater.

Selection of wastewater treatment temperatures and evaluation of stormwater flows was based on monthly mean data for the Calgary International Airport as summarized in Table 7.4.

It should be noted that this evaluation is of a conceptual nature and the environmental data was used to select conditions for approximate sizing calculations. Detailed design of the site and any wetland or lagoon features must include analysis with a more complete hydrological computer model. Such modeling should be based on continuous simulation with historical temperature, wind and precipitation information and include provision for estimation of evapotranspiration and infiltration. Detailed

hydrological analysis should be carried out by a qualified engineer fully experienced with the appropriate design software.

Table 7.3: Lagoon and Probable Sewage Treatment Wetland Setbacks
(Alberta Environment, 1991)

	Setback	
	feet	metres
	<u>from outside toe of berm</u>	
Rural road (right of way)	100	30
Numbered highway	300	90
Property line or right of way	100	30
	<u>from water surface</u>	
On-Site residence	330	100
Off-Site residence	1000	300
Drainage course	75	23

Table 7.4: Environmental Data for the Calgary International Airport
(Environment Canada, 1990)

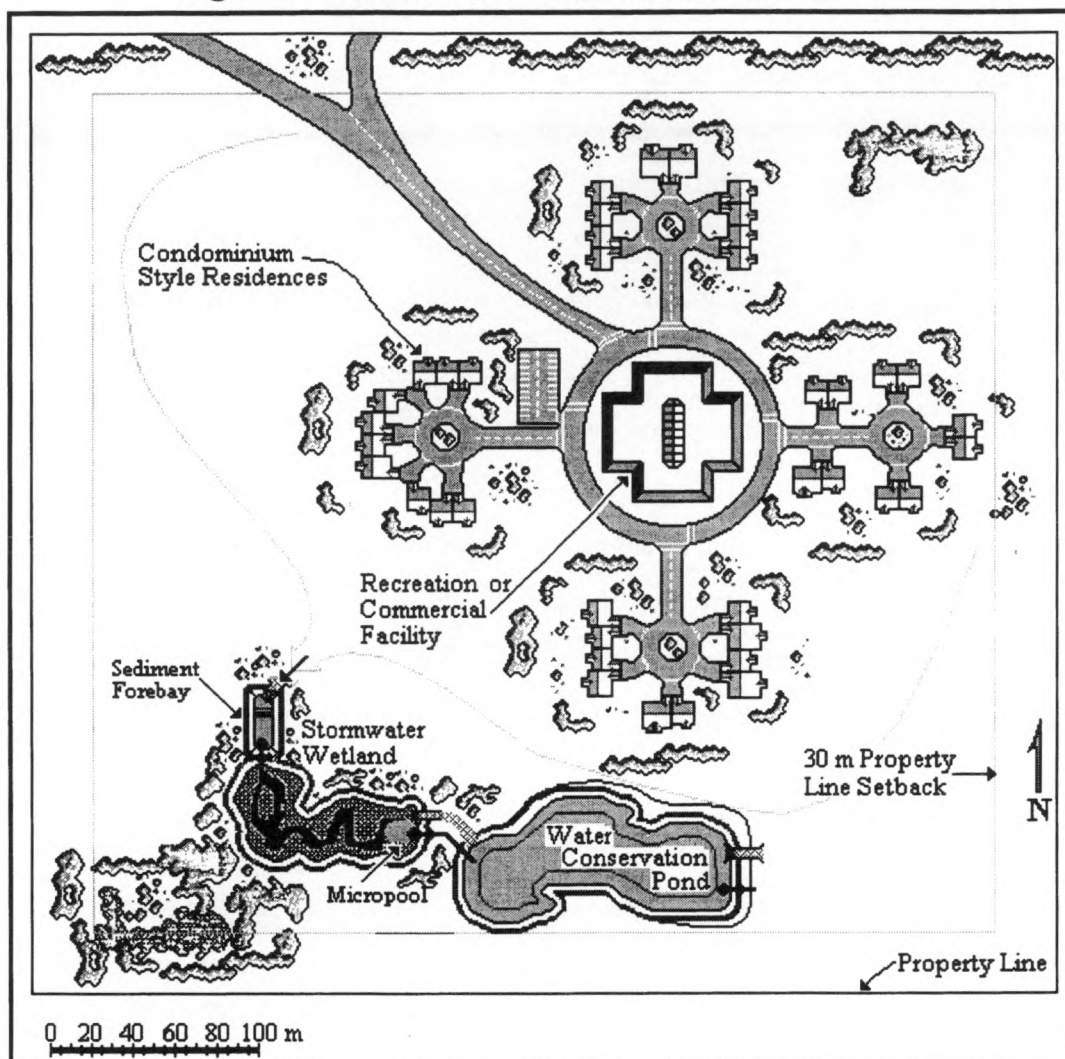
Month	<u>Monthly Mean Values (1961 - 1990)</u>		
	Temperature	Precipitation	Lake Evaporation
	deg. C	mm	mm
January	-9.6	12.2	--
February	-6.3	9.9	--
March	-2.5	14.7	--
April	4.1	25.1	--
May	9.7	52.9	--
June	14.0	76.9	143.9
July	16.4	69.9	154.0
August	15.7	48.7	129.6
September	10.6	48.1	95.4
October	5.7	15.5	--
November	-3.0	11.6	--
December	-8.3	13.2	--
		398.7	522.9

7.3 Stormwater Management

Conservation of site snow melt runoff and stormwater can provide a source of irrigation and utility (e.g., toilet flushing, outside taps) water. Although most runoff flow paths in rural developments like the hypothetical case are well vegetated (and thus filter the stormwater), it is assumed the presence of pets, higher sediment loads due to gravel access roads and use of horticultural chemicals will make it desirable to treat runoff.

A stormwater wetland with moderate extended detention is proposed to protect the quality of water used for utility purposes as is illustrated in Figure 7.2. The wetland is based on the ED wetland (Design #3) and Puget Sound guidelines discussed in Section 5.5.6. The water conservation pond could provide treatment of stormwater similar to wet ponds. The wetland, however, provides removal of sediments and a considerable portion of nutrients from the runoff, thus, protecting the quality of water in the pond for use for aquaculture or recreation, as well as for irrigation and utility water use.

The treatment volume of the wetland was based on the volume of runoff expected from a 1:2 year 24 hour storm in the Calgary area. The 1:2 year event was chosen as a proxy for an event selected so that the sum of runoff from it and all smaller storms accounts for 90 % of runoff. Detailed selection of the 90 % event requires evaluation of historical meteorological data with a continuous hydrological model and it is likely such an evaluation would result in a smaller system than the one described below. The site has about 11 % impervious area, which with the assumed clayey soil results in about 800 m³ of runoff from the design event. The assumptions about site conditions result in the wetland area requirements shown in Table 7.5. The predicted wetland size is a little less than one percent of the site drainage basin which agrees with preliminary sizing guidelines (Schueler, 1992). Calculations in Appendix A indicated that some runoff from larger events will likely have to be diverted off-site. The 1:100 year event, for example could produce sufficient inflow rates to wash-out parts of the wetland.

Figure 7.2: Relative Size of the Stormwater Wetland**Table 7.5: Stormwater Wetland Sizing**

Zone Type	Depth (m)	Relative Area	Area (sq m)	Volume (cu m)
Forebay	1.1	10%	225	248
Hi Marsh	0.1	50%	1127	113
Lo Marsh	0.3	30%	676	203
Deep Holes	0.9	5%	113	101
Micropool	1.1	5%	113	124
			2254	789

Figure 7.2 demonstrates a few features of stormwater wetland planning. Calculations (Appendix A) for the wetland, based on a 3:1 aspect ratio, suggested a rectangular water surface of 25 by 80 m. The wetland and conservation pond are

illustrated, however, with irregular shorelines that provide better wildlife habitat. The irregular shapes also augment the aesthetic value of the system. The light and dark patterns of the stormwater wetland shown in Figure 7.2 illustrate creation of more complex flow paths by altering hi and lo marsh sections. Surrounding landscaping should be designed to integrate the wetland into the site as a naturalistic amenity.

The size of the water conservation pond was chosen somewhat arbitrarily, however, the size illustrated has relevance to the following discussions of domestic wastewater treatment options. While several natural sewage treatment processes will be discussed, this stormwater management conceptual design will apply to all cases.

7.4 Domestic Wastewater Treatment

Several natural, passive wastewater treatment options could be considered for the site, as well as many different conventional mechanical processes. The author acknowledges that one or more mechanical systems may have cost, safety and environmental advantages for primary-secondary treatment in this situation. Sand filter technology, discussed in Section 4.2.1, could also be considered for the site. Sand filters could readily be designed for the hypothetical example and related experienced cold climate engineering services are available in Calgary. These systems will not be further discussed, however, as the following section is intended to illustrate wetlands wastewater treatment design procedures and related implications.

A primary design objective imposed by the hypothetical evaluation is the requirement for wastewater reuse. This feature eliminates the desirability of infiltration treatment-disposal systems (e.g., septic fields) even though the technology might otherwise be appropriate for the project. It also favors the use of a wetland to treat sewage lagoon effluent to a higher quality prior to reuse.

Several alternatives are possible to combine wetlands wastewater treatment with other passive technologies. In all cases it was assumed that treated effluent could be “polished” in the stormwater wetland. This provides base flow to sustain the runoff marsh during dry periods. The water conservation pond volume was assumed to be equal to one

years wastewater production. Options for wetlands domestic wastewater treatment include:

- lagoon-FWS wetland combination with winter sewage storage and growing season wetland treatment
- septic tank - year-round FWS wetland processing
- septic tank - year-round VSB wetland treatment.

7.4.1 Lagoon - Free Water Surface Wetland System

Sewage pretreatment and winter storage in a lagoon with growing season wetland treatment is a common approach for cold climate wetlands wastewater treatment systems (Lakshman, 1983, Hammer and Knight, 1992). Design procedures are well documented and major projects such as Minot, North Dakota afford evidence of system successes.

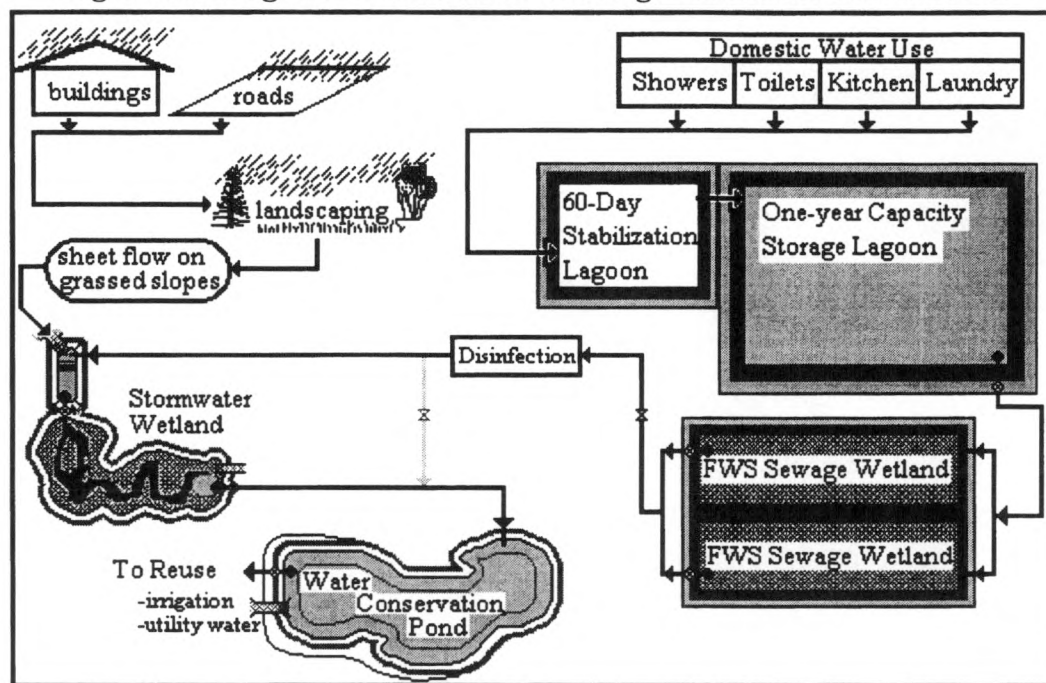
Lagoon sizes are expected to be governed by Alberta guidelines for small municipal developments (Alberta Environment, 1991). These require a 60-day residence time treatment cell and a one year capacity storage cell. Based on Alberta guidelines, the treatment cell should not be deeper than 1.5 m and the storage cell not deeper than 2.5 m.

Design procedures for sewage treatment FWS constructed wetlands recommend that surface drainage be directed away from the system. Thus it is proposed that a system separate from the stormwater wetlands be constructed. Depending on effluent quality and setback requirements, tertiary quality effluent from the sewage treatment wetland could be directed to the stormwater system to provide needed dry weather base flow. The process flow for the proposed system is illustrated in Figure 7.3.

The lagoons specified by Alberta guidelines should meet treatment objectives for BOD but may not meet desired water quality conditions for other contaminants. Lagoon sizing and performance calculations are provided in Appendix B and indicate that when the storage lagoon is at minimum drawdown both nitrogen and fecal coliform levels can be expected to be well above desired conditions. The calculations indicate that coliform levels will be very high. The literature suggests lower values can be expected, although

reported lagoon systems with good pathogen removal consisted of several cell in series (US EPA, 1983).

Figure 7.3: Lagoon - FWS Wetland Sewage Treatment Process Flow



The lagoons will require substantial space on the site. Square ponds were selected for the case study due to ease of calculation and for minimum construction cost considerations (square shapes minimize the length of berms). Table 7.6 summarizes sizing of the lagoons.

The constructed FWS wetland needed to reduce BOD from 30 mg/l to 10 mg/l requires 1.8 ha of surface area. The evaluation assumed the full storage pond would be drawn down in 100 days between mid May and September. The average ambient temperature during this period is only about 13°C which results in slow reaction rates in comparison to designs for more southerly climates. “Design Condition #1” calculations in Appendix C indicate the wetland should be able to reduce nitrogen to 2 mg/l from typical lagoon effluent concentrations (15 mg/l). The design should not be expected to reduce fecal coliform levels to contact recreation standards and disinfection of the effluent will be required to meet effluent quality objectives. Chlorine, ozone and

ultraviolet processes could be used to deal with residual bacteria levels and thus ensure irrigation and contact recreation water quality of downstream systems.

Table 7.6: Lagoon Sizes

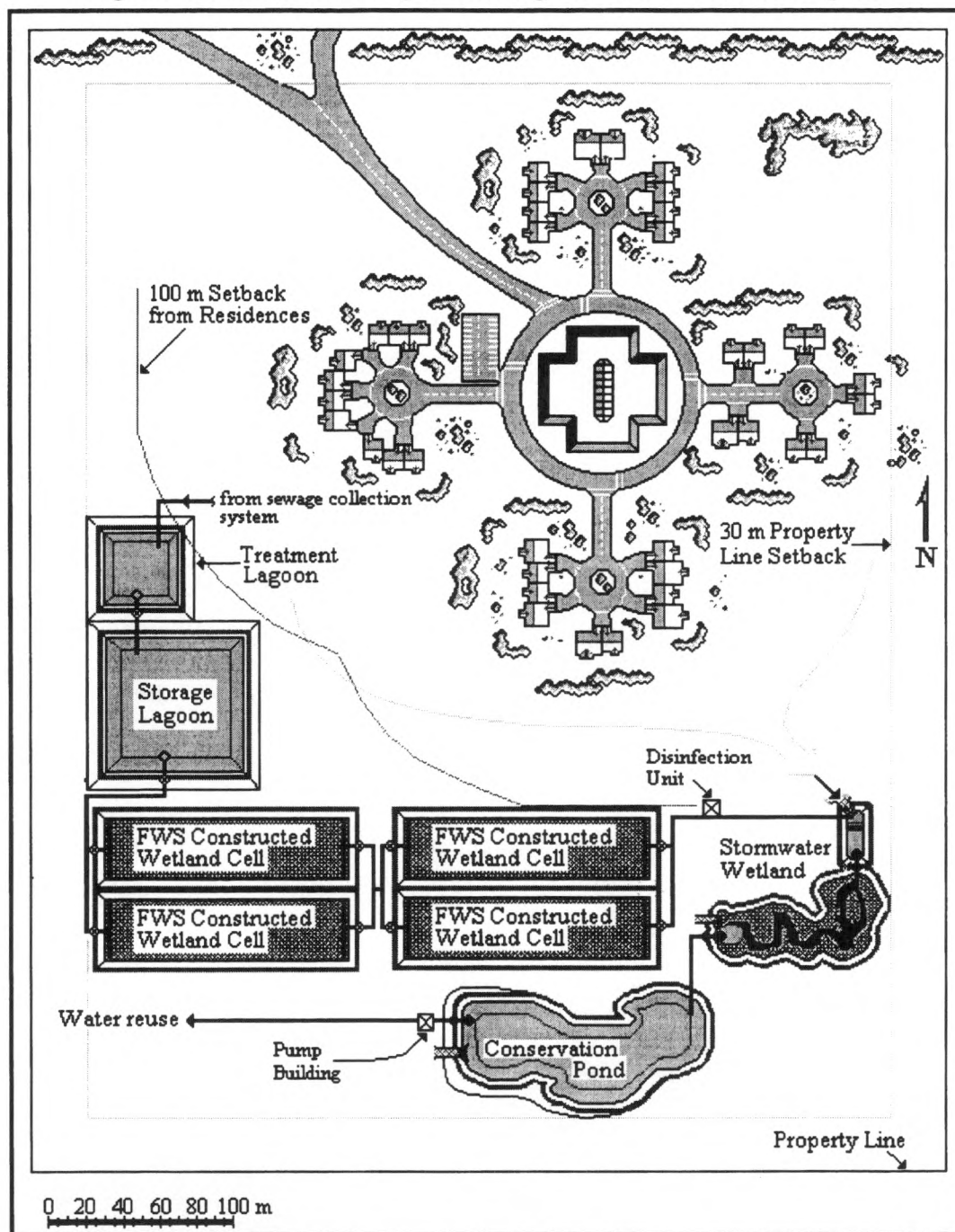
Parameter	Treatment Lagoon	Storage Lagoon
Hydraulic Residence Time	60 days	365 days
Maximum Water Depth	1.5 m	2.5 m
Width at Water Surface	39.9 m	75.2 m
Width at Top of i/s Berm	43.5 m	78.8 m
Width at base of o/s Berm	58.3 m *	93.6 m *
* Assumed excavation depth below grade: 1.0 m for treatment pond and 2.0 m for storage pond.		

The proposed FWS system design is based on four cells 46 m wide and 146 m long (assuming 0.6 m freeboard and 3 m wide access path around the cell). The four cells are assumed to be configured in two parallel trains. The multiple cell arrangement permits maintenance of the wetland while processing at least 50% of the design flow. The layout of the system is illustrated in Figure 7.4.

The proposed system clearly illustrates the primary trade-off of passive water treatment systems. While energy intensive mechanical or chemical processes are avoided, substantial land area must be employed. The complete wastewater - stormwater system was placed outside the 100 m setback specified for lagoons by Alberta guidelines. To facilitate appropriate spacing, the area in Figure 7.4 had to be increased by 7.8 ha. The incremental land costs and the ability of developers to acquire more land are impediments to use of the lagoon - FWS wetland sewage treatment system.

In addition to land requirements, other drawbacks to this approach exist. The lagoons must be fenced and it is likely the same requirement would be expected for the secondary treatment wetlands. Both the lagoons and the wetland are potential odor sources and the wetland will provide insect breeding habitat. Construction cost and maintenance considerations will favor regular, preferably square systems, limiting opportunities to make the lagoons and wetlands more aesthetically pleasing.

Figure 7.4: Relative Sizing of the Lagoon - FWS Wetland Option



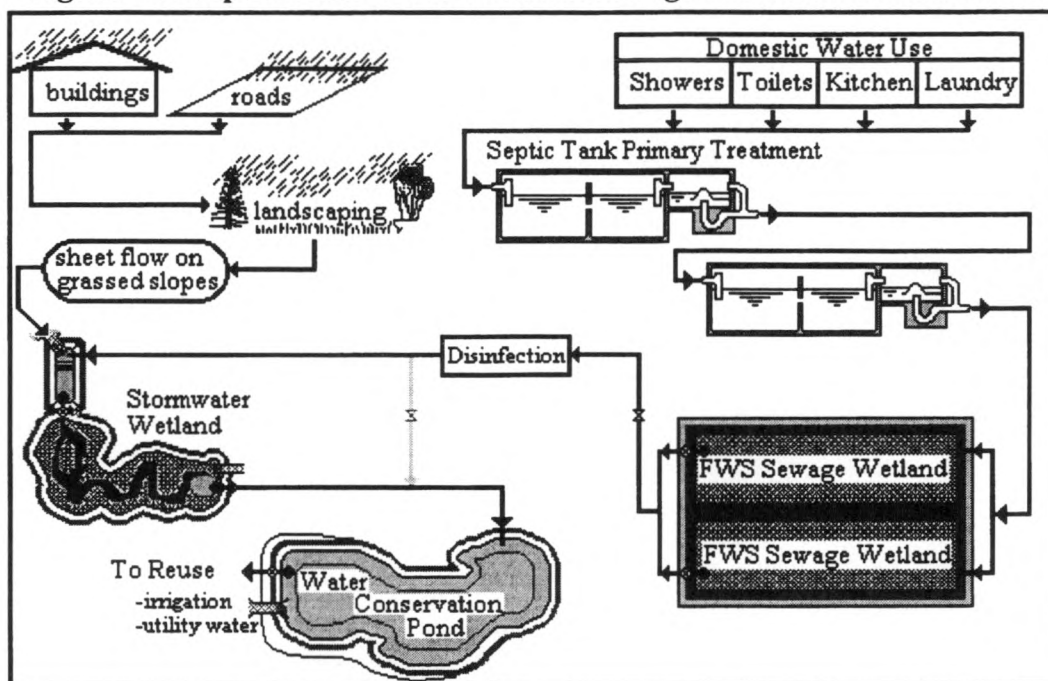
7.4.2 Septic Tank - Free Water Surface Wetland System

Elimination of the sewage lagoons is desirable not only to reduce land requirements for domestic wastewater treatment but also to eliminate the cost, safety and nuisance concerns associated with the ponds. Canadian pilots have demonstrated that FWS wetlands can effectively remove BOD from primary treated wastewater on a year-round basis (Herskowitz, 1986, Miller, 1989). It is possible that the lagoons could be eliminated and the wetlands modified (deeper flooding during the winter) to treat on a year-round basis. The process flow for such a system is illustrated in Figure 7.5.

A detailed evaluation of septic tank design was not undertaken. It is probable that many septic tanks may be required for the illustrated project including septic tank systems (two or more tanks in series) for each cluster of residences, for the central complex and possibly back-up - flow stabilization tank where the subsystems interconnect. For the purpose of this discussion, however, it was assumed that one septic tank installation would serve the whole site. Reed (1993) recommends two 3-compartment septic tanks in series for wastewater flows over $38 \text{ m}^3/\text{d}$ and that the combined volume provide two days hydraulic residence time. Applied to the hypothetical project, the guidelines require two tanks, each 1.8 m deep by 2.6 m wide and 7.7 m long.

Required wetland area for this option is approximately the same as required for the preceding case. Because wastewater is treated as it is generated, the design flow rates are lower than in the first case (which must drawdown annual storage during summer months). This factor is offset by the much cooler winter processing temperatures which depress reaction rate constants. "Design Condition #2" calculations in Appendix C were based on 2°C water temperature under an insulating blanket of wetland vegetation, ice and snow as recommended by Reed (1993). Winter water temperatures recorded for the Listowel, Ontario pilot project, however, indicate conditions as low as 0.5°C can be expected (Herskowitz, 1986).

Figure 7.5: Septic Tank - FWS Wetland Sewage Treatment Process Flow

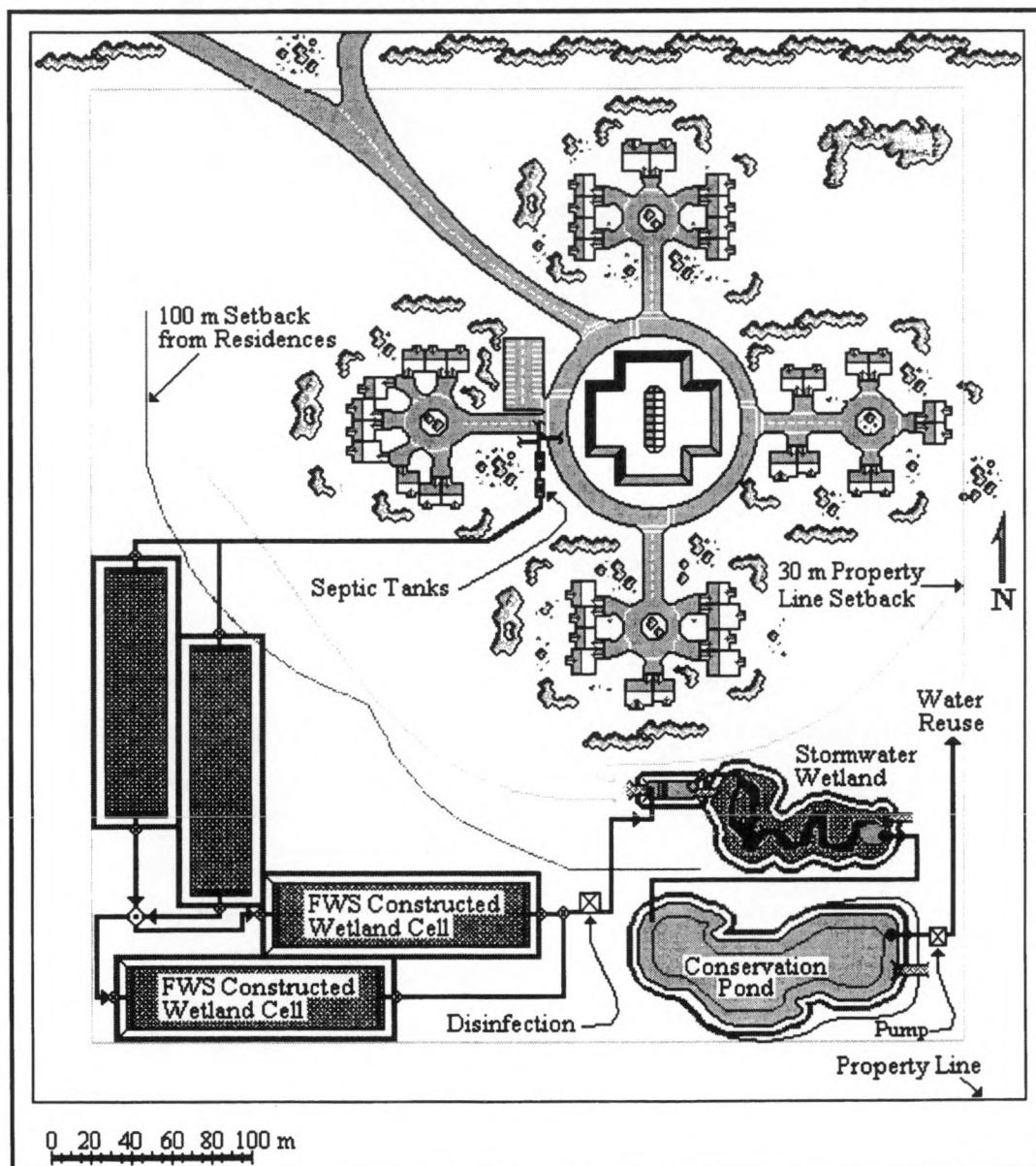


The wetland sizing calculations are based on BOD removal from a typical septic tank effluent value of 140 mg/l to 10 mg/l. Under winter conditions, a wetland so sized can be expected to reduce fecal coliform levels by at least one order of magnitude and to have limited effect on ammonia nitrogen (no oxygen transfer for nitrification). The resulting effluent would require disinfection before release into surface waters such as the proposed conservation pond. Under May to September conditions, the wetland would likely reduce fecal coliform levels by two orders of magnitude. Effluent disinfection would still be required as counts would still be several orders of magnitude greater than the selected treatment objectives.

Overall landuse for this option, however, is considerably less than for the case involving lagoons. The need for septic tanks will partially offset the elimination of the lagoons. Septic tanks require minimal setback (1 m) from buildings (Alberta Labor, 1990), however, it is assumed the sewage treatment FWS wetlands must meet the 100 m setback and fencing provisions required for lagoons. In this case a wetland surface area of 1.6 ha is predicted resulting in four cells in two parallel trains. The external berm dimensions of each cell was determined to be 44 m by 140 m. The relative sizes of the components of this option are shown in Figure 7.6.

In terms of land use issues this option fares little better than the case using lagoons. Assuming that the building layout will not change (wastewater questions are often asked after the site plan is more or less fixed), an additional 3.5 ha of land is required to accommodate the sewage treatment wetlands and setbacks. Issues related to odors and hazards associated with contaminated surface water waterbodies also remain.

Figure 7.6: Relative Sizing of the Septic Tank - FWS Wetland Option

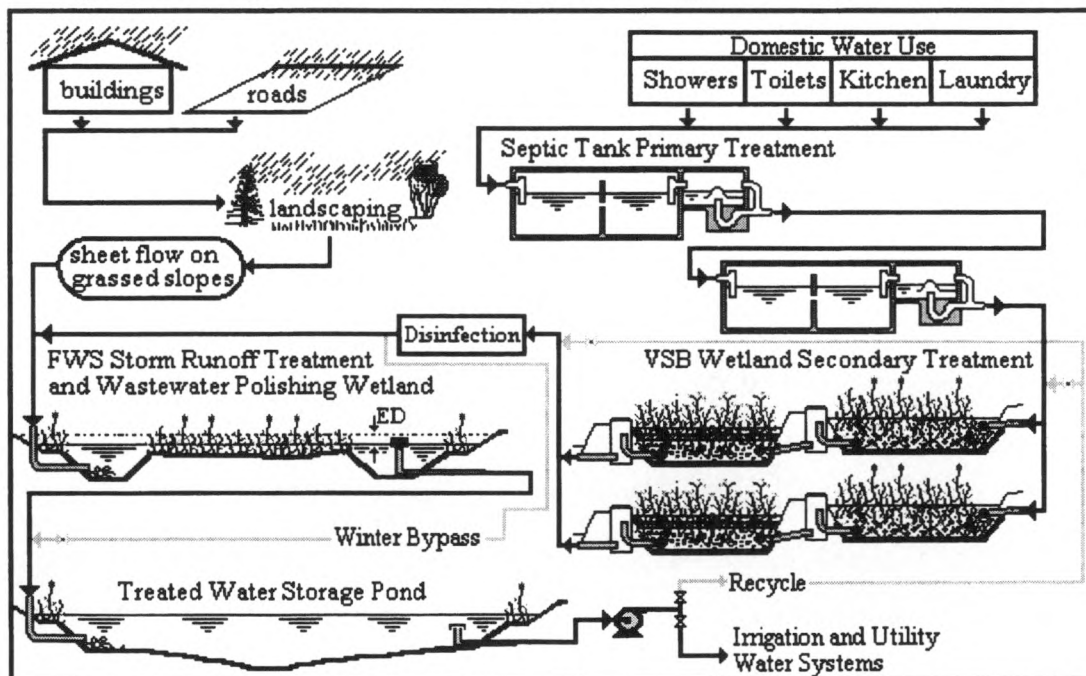


7.4.3 Septic Tank - Subsurface Flow Wetland System

Subsurface flow or vegetated submerged bed (VSB) wetlands may have greater ability to operate year-round in cold climates (WPCF, 1990). The technology requires less land than a FWS system meeting the same effluent BOD objectives and is less likely to be a source of odors or insects. It is anticipated that VSB systems could be treated much like septic fields in terms of setback requirements which will permit more compact sites. Offsetting the noted advantages are higher unit area construction and operating costs.

The proposed VSB wetlands wastewater treatment system consists of septic tank primary treatment followed by secondary sewage treatment in four wetland cells arranged in two parallel trains. As in preceding options, it is proposed to use the stormwater wetland for polishing which enables the treated domestic wastewater to provide base flow to the runoff system during dry periods. The concept is shown in Figure 7.7.

Figure 7.7: Septic Tank - VSB Wetland Sewage Treatment Process Flow



This alternative can fit within the originally proposed site unlike the other options. The two large septic tanks need to be 1.8 m deep by 2.6 m wide and 7.7 m long to provide two day residence time for expected wastewater flows. These tanks can be as

close as one metre to buildings on the site. Effluent from the septic tanks can be treated in four 12 m by 26 m VSB cells. Because there is no surface water flow (which minimizes health risks), it is reasonable to expect that septic field setback of 9 m from building basements will apply (Alberta Labor, 1990). Calculations are presented in Appendix D and the relative sizing of the VSB based system is illustrated in Figure 7.8.

As an aside, the VSB beds are nearly the same size as is predicted for single-pass buried sand filters (four parallel 13 by 26 m beds based on 25% redundancy). Only rough calculations were performed for the filters, but it would appear the technology represents a key “conventional” alternative to wetlands wastewater treatment for the hypothetical case.

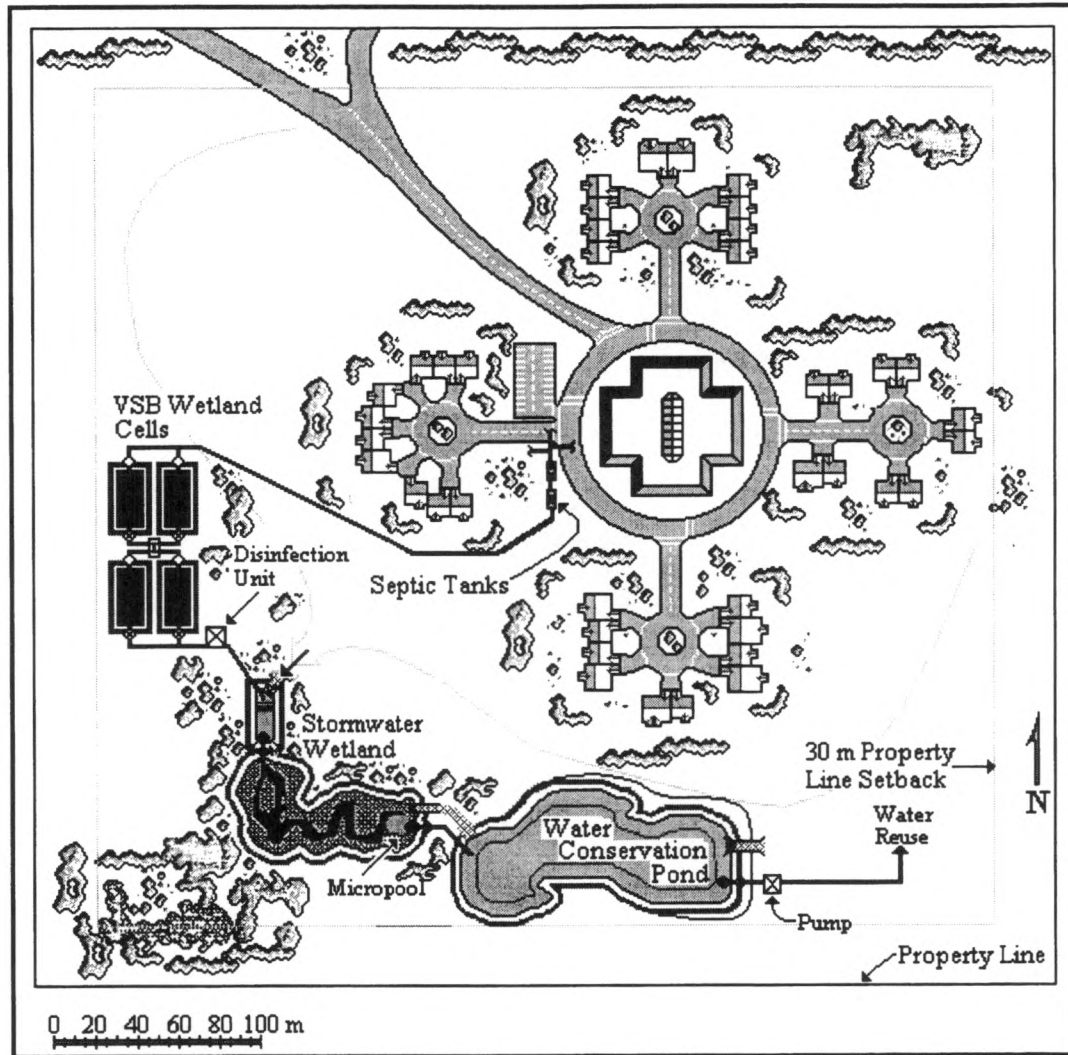
The proposed design will meet effluent objectives for BOD at 2°C and under summer conditions should meet nitrogen objectives. Due to the cold design temperature, the hydraulic capacity of the system is high which should minimize plugging problems. Under summer conditions, two of the four cells operating in series should be sufficient for BOD removal, thus the design provides for a high level of maintainability.

The literature reports that the system should result in a one to two log reduction of fecal coliforms (Reed, 1993) which will not be sufficient to meet contact recreation standards. For this reason a disinfection step is shown in the process flow diagram (Figure 7.7). Disinfection can be accomplished with chlorine, ozone or ultraviolet technologies and suppliers can offer packaged units suitable for this installation.

Detailed project designs will be required to address both media plugging and freezing problems associated with VSB technology. Plugging of VSB systems has been reported in the literature although Reed (1993) contends that many of the problems relate to inadequate hydraulic design, excessively short residence times and poor construction practices (the media must be clean, washed gravel). The combination of conservative hydraulic design, cold design water temperatures and use of high quality media would appear to mitigate problems. The author, however, has noted a great divergence of opinion on the applicability of VSB technology and it is strongly recommended detailed

design of specific projects provide funding for expert consultation on both sides of the issue.

Figure 7.8: Relative Sizing of the Septic Tank - VSB Wetland Option



It is anticipated that a combination of septic tank insulation, VSB cell mulching and possibly supplemental heating of the wastewater or VSB cells will overcome freezing problems. The biological degradation of organic matter will also provide energy to the VSB beds. In so much as septic drain fields are routinely used under Alberta winter conditions, it is expected that freeze protection of the VSB wetlands should be feasible.

7.5 Design Example Conclusions and Recommendations

7.5.1 Conclusions

1. Natural and probably conventional wastewater technologies are available for small wastewater flows that conserve water and enable reuse of treated streams for irrigation and utility water.
2. FWS or VSB treated domestic wastewater will require disinfection if it is to be used for contact recreation or unrestricted irrigation.
3. Passive sewage and stormwater ecotechnologies have the potential to meet requirements of the small developments.
4. The more common cold climate option of winter storage lagoons and warm season FWS wetland treatment requires an unacceptably large area relative to desired uses of typical sites.
5. A suitably freeze protected septic tank - VSB wetland system could treat domestic wastewater within the confines of typical sites.
6. VSB technology is noted for plugging problems and little design information is available for cold climate situations.
7. Stormwater wetland management is a reasonable means to conserve precipitation on the site.

7.5.2 Recommendations

1. Septic tank - VSB wetland treatment of domestic wastewater should be considered for stand alone developments where wastewater disposal options are limited or where utility or irrigation water resources are scarce.
2. Planning for detailed design of VSB wetland domestic wastewater treatment should include provision for investigation of reported plugging problems and techniques used in other locations to deal with freezing conditions (at least until such time as Alberta operating experience can provide standard design techniques for our conditions).

8. WETLANDS AND NATURAL WATER TREATMENT ISSUES, CONCLUSIONS, RESEARCH NEEDS AND RECOMMENDATIONS

The discussion of basic principles and design procedures for wetlands and other natural wastewater treatment technologies in Sections 2 through 6 reveal that much development has been done. The ability to create preliminary designs and estimate treatment effectiveness, as demonstrated in Section 7, exists as do many examples of operational systems. Wetlands wastewater treatment systems, however, are an emerging technology and consequently exhibit significant deficiencies in design information. Few of the pilots have been conducted in Canada and even fewer in the Canadian prairies.

Seasonal wetlands treatment of sewage lagoon effluent, even in Canada, appears to be a reasonably developed technology. Many of the other applications, and especially those with great potential to reduce nonpoint source water pollution still need to be proven in Western Canada. This section will discuss key issues identified from review of the literature and from discussion with professionals and regulators in Alberta, will summarize conclusions from the study and will recommend research and development needs for wetlands wastewater treatment in the Province. While Sections 4 and 5 addressed a range of natural treatment processes, this section will focus on wetlands treatment systems.

8.1 WWT Issues

A number of issues relating to WWT have been repeatedly identified in the literature and by key informants. The following issues will be discussed in detail in the following pages:

1. Performance limitations of WWT technology during winter, especially of wetlands used for runoff water quality management.
2. Ability to establish desired density and type of wetland vegetation in short frost-free seasons and in the dry prairie climate.

3. Public acceptance of the technology as safe and, in the case of stormwater wetlands, as desirable community amenities.
4. Potential for increased mosquito populations and associated health and public nuisance concerns.

8.1.1 The Cold Climate Issue

Successful sewage treatment pilots have been carried out in Canada. The much cited Listowel, Ontario project maintained operations during the winter with ice cover on the WWT cells (Herskowitz, 1986). Suspended solids and BOD results demonstrated seasonal variation depending on test cell design but high levels of removal were reported for winter operation. A system, operating year around in Cobalt, Ontario also reported treated sewage effluent quality consistent with secondary treatment despite a more northerly location than Listowel (Miller, 1989). The work of the Saskatchewan Research Council was based on seasonal treatment of municipal sewage lagoon effluent with winter storage (Lakshman, 1983). The Blue Quills School sewage system (near St. Paul, Alberta) has successfully used natural wetlands for over 40 years (Kent, 1987).

Several cold climate applications have been developed in the US, such as the process wastewater treatment wetland system for Amoco's Mandan, North Dakota refinery (Litchfield and Schatz, 1989). The Minot, North Dakota WWT system treats municipal effluent for a population of 35,000 on seasonal basis with winter storage (Hammer and Knight, 1992). Gover (1993) reports that 60 operational constructed wetlands have been installed in the Colorado, New Mexico and Arizona area where high elevations can result in severe temperatures (to -36 °C). These systems, mostly using VSB technology are designed for continuous cold weather operation. Both D.A. Hammer and S.C. Reed are quoted in the Gover (1993) article as recognizing the feasibility of winter operation with VSB and FWS wetlands. Longer residence times are required due to low reaction rates but BOD, TSS and fecal coliform removal was expected to be effective. Hammer noted that nitrogen removal could be limited due to sensitivity of removal processes to low temperature and low oxygen levels. Notably, design

procedures for sewage treatment wetlands advise winter storage (Reed et al., 1988, WPCF, 1990, Hammer and Knight, 1992).

Reuter, Djohan and Goldman (1992) reported effective turbidity, nitrate and TSS removal from Lake Tahoe area runoff even under alpine winter conditions. The 660 m² Lake Tahoe runoff VSB system was constructed to treat runoff from a 1 ha fertilized athletic field.

Both Suncor and Syncrude have established WWT pilots near Ft. McMurray, Alberta (Bishay et al., 1993, Boerger and MacKinnon, 1993). While the systems have been operated seasonally, information indicates the ability to develop the wetlands within one growing season.

The literature indicates that seasonal sewage treatment FWS wetlands for the northern prairies approaches a proven technology. This approach, however, requires substantial land and systems capable of year-round operation with VSB technology would appear to be more favorable. Less information was found for runoff applications, although those few suggest the technology could continue to provide some benefit for runoff (meltwater) even under winter or early spring conditions. It is concluded that wetlands water quality treatment applications such as agricultural and urban nonpoint source pollution control could prove to be economic and effective processes in Western Canada.

8.1.2 Establishing Vegetation in Constructed Wetlands

Dense growth of emergent vegetation is critical to the success of wetlands wastewater treatment. Development of wetlands in the western prairies of Canada will require assessment of suitable plant species and definition of procedures to establish vegetation in newly constructed marshes. Literature (Bishay et al., 1993, Boerger and MacKinnon, 1993) and discussions with key informants such as Ducks Unlimited (Usher, 1993, pers. comm.) indicates that successful establishment of emergent wetland plant communities in Alberta is possible within one growing season.

Selection of plants will require consideration of plant suitability to climate, hydroperiod, soils and design objectives for habitat, biological diversity and aesthetics. Soils, microtopography and hydroperiod are controllable to some extent by wetland design. Ducks Unlimited has developed information on germination, soil, water quality (salt tolerance) and water level requirements for a variety of wetland plants that could guide early WWT trials (Usher, 1993, pers. comm.). To date, wetlands wastewater treatment systems with limited diversity have been successfully created in Alberta. Suncor successfully established cattails (*Typha latifolia*) and bulrush (*Scirpus validus*) on 0.45 m of sand covered by 0.15 m of muskeg soil (Bishay et al., 1993).

8.1.3 Public Perception

Public perception of wetlands as unpleasant, disease infested and undesirable features of the landscape has been fostered over long human history, although recent environmental enlightenment has started to change that perception (Smardon, 1989, AWRC, 1990a). While sewage treatment wetlands may provide wildlife habitat, such systems will likely continue to be segregated from residential areas and be subject to restricted access. The issue of public perception becomes more critical for stormwater wetlands located within developments, however. Urban wetlands for nonpoint source pollutant control are potentially one of the greatest values of wetlands wastewater treatment technology and the remainder of this section will be based primarily on public reception issues associated with stormwater wetlands.

The author's discussion with municipal officials, Provincial regulators, developers and engineers identified concerns about wetlands including:

- perceived preference of home owners for manicured "green areas" and expectation that natural landscaping around and including stormwater wetlands will not be accepted.
- concern that stormwater wetlands will produce noxious odors.
- concern that wetlands present safety hazards, especially to young children and consequently liability risks for municipalities.

- perception that wetlands will be more maintenance intensive than conventional approaches.
- perception that wetlands will be significantly more expensive to establish and maintain than conventional urban open spaces.
- concern that the wetlands will increase mosquito populations.

Manicured versus Natural Open Space: Stormwater wetlands can be designed as multiple use urban wildlife habitat - park systems (Palmer and Hunt, 1989, Ellis, 1988). The addition of naturalistic green space and closer association with wildlife has been shown to be a valued community amenity (Ellis, 1988). A recent Calgary survey identified strong public preference for more natural open space and related nature focused recreation (City of Calgary, 1992). Design procedures suggest means of tailoring designs to encourage (or discourage) wildlife and stress the importance of aesthetic landscape appearance (Schueler, 1992). A growing public environmental awareness has no doubt conditioned many to at least be receptive to considering wetlands concepts. That awareness will aid the essential public information - input process that needs to accompany design of stormwater wetlands projects within or adjacent to residential neighborhoods.

Odors: In general, properly designed wetlands should not produce odors. Design must consider organic loading, which if excessive can result in odor producing anaerobic decomposition near inlet areas. Conditions that permit proliferation of algae and stranding of algae and dead water plants can also lead to odors. Designs should provide for dense emergent cover to control algae and ensure appropriate vegetation selection for areas exposed to drawdown.

Risk: Any standing water in areas accessible to children presents both an invitation to play and a drowning hazard. As a result potential amenities such as the City of Calgary's 17 Avenue and 68 Street SE wet pond are barricaded behind high security fences. While hazards of any waterbody cannot be completely eliminated, a number of passive design approaches can make the facilities safer. Schueler (1987 and 1992) provides for aquatic benches around wet ponds and wetland BMPs. These shallow,

vegetated strips discourage ready access to deeper water. Shallow side slopes can also reduce the chance of slipping into deep water. Buffers of dense shrubbery can be planted around wetlands to form not only passive fences but also cover for wildlife.

The functional alternative to wetlands are dry ponds. The risks posed by periodic deep and rapidly fluctuating water levels in dry ponds may be more significant than those of permanently flooded areas.

Maintenance: Maintenance costs for wetland systems can be higher than pond BMPs in the first years until a vigorous coverage of emergent plants is established, however, long term costs are assumed to be comparable to wet or dry pond systems (Schueler et al., 1992). Naturalistic landscaping in and around wetlands should be designed to minimize or eliminate mowing offering an advantage over grassed dry ponds.

Capital costs may be increased by additional grading to create a varied wetland microtopography and costs of obtaining and planting vegetation. These additional costs may be offset by funding from programs and agencies that support development of natural areas. Such funding would not apply to conventional stormwater management options. Wetland conservation of runoff and installation of passive irrigation techniques may eliminate the need for irrigation systems in modified extended detention ponds. In the Calgary area, it is possible dry ponds, modified with wetlands to receive low flows and landscaped with passively irrigated naturalized vegetation may have lower life-cycle costs than the technology currently in use.

Increased Value: Any discussion of costs must factor in the values gained, including improved condition of receiving waterbodies, aesthetic addition to the urban landscape and the opportunity to provide some wildlife habitat within developed areas. Appreciation of amenity worth will be reflected in land values for property bordering stormwater wetland parks. Property value appreciation notwithstanding, evaluations cannot be carried out solely on a fiscal plane as it is wrong to reduce aesthetic and environmental values to singular dollar figures.

Mosquito Control: Wetlands are mosquito habitats, however, design and natural control methods exist to inexpensively and effectively deal with the issue. Predator control has been successfully used in many systems. Designs supporting greater microhabitat diversity and developed with the assistance of mosquito control specialists have reduced mosquito problems (Dill, 1989). Methods for controlling mosquitoes suggested in Reed et al. (1988) include stocking with mosquitofish (*Gambusia*), encouraging duckweed (*Lemna sp.*) coverage of open water and using bacterial methods (*Bacillus thuringiensis var. israeliensis* or BTI). Mosquitofish controls are reported as inappropriate for cold climates (Reed et al., 1988) although the fish are known to survive in the Vermilion Lakes near Banff, Alberta. The Listowel, Ontario project reported increased mosquitoes but that the species collected did not feed on humans (Herskowitz, 1986). BTI was demonstrated to be an effective control in the Listowel project.

The City of Red Deer, Alberta has demonstrated a low cost, extremely effective biological mosquito control program (Wasylyshyn, 1993, pers. comm.). The program includes mapping and monitoring of breeding sites, understanding of which mosquito species present a nuisance, establishment of natural predators (stickleback fish, shrimp, flatworms, damselfly larvae and dragonfly larvae) and selective application of BTI to control mosquito larvae outbreaks. The control system does not depend on chemical means and thus protects natural mosquito predators.

Of the thirty or so mosquito species present, *Aedes flavescens* and *Aedes excrucians* are the principle species that attack humans. These species hatch when the eggs are flooded following a dry cycle, thus continuously flooded wetlands are not as significant a source as periodically flooded ditches, ruts and potholes (Wasylyshyn, 1993, pers. comm.). Information on mosquito control indicates that good wetland design and low cost biological treatment can mitigate serious problems with the pest.

8.2 MDP Conclusions

The primary purpose of this study was to review and consolidate background and particularly design procedure information on wetlands and natural wastewater treatment

processes. In the process of the literature review and the author's involvement in discussing wetlands wastewater treatment with developers, engineers and regulators, several conclusions have been reached.

1. Natural and wetlands wastewater treatment processes are based on scientifically sound principles of physical and biochemical removal and transformation of water contaminants. In general, the natural process rely on solar input, ample land and longer residence times to replace the capital and energy inputs required by conventional mechanical systems.
2. Hydrological evaluation is critical to developing healthy stormwater (and other) wetland systems, however, typical development hydrological studies do not provide the type of information needed for wetland design.
3. Natural and wetlands wastewater treatment rational design procedures along with recommended practices have been developed to enable reasonable design evaluations. The most complete procedures have been developed for sewage treatment applications. Designs for other applications are typically based on contaminant or hydraulic loading factors. Most of the design relationships are experiential or empirical and detailed understanding of natural processes, especially wetlands, is still being developed.
4. Wetland systems depend on natural physical processes and functioning ecology and consequently should be expected to exhibit variable performance as a function of local conditions and the season. As ecosystems, the processes probably will not be able to be designed with the same accuracy and predictability as closed mechanical or chemical processes.
5. Wetland and other natural wastewater treatment technologies have been successfully demonstrated in cold climates and some are commonly used in western Canada (e.g., SWIS, lagoons). Constructed wetlands have been successfully developed and vegetated in one growing season in Alberta, thus creation of the systems is possible.

6. Stormwater wetland technology offers the opportunity to improve runoff water quality and, with skilled naturalized landscape design, has the potential to add amenity and wildlife habitat value to urban landscapes.
7. There is a large body of literature on the subject, typically organized by application. The lack of simplified and consolidated design reference appears to be one barrier to wider use of the technologies in two ways. Firstly, it is time consuming for design professionals to acquire a broad understanding of the technology. Secondly, decision makers have limited means with which to acquaint themselves with the natural technologies and related development and policy implications.
8. The lack of local experience with the technology, particularly stormwater wetlands is a barrier to regulatory and engineering acceptance. While more scientific research has been called for, practical full-scale applications are needed to develop design, operation and maintenance experience and to demonstrate the technology. Demonstration projects are particularly needed for urban stormwater quality management, small volume domestic or municipal wastewater treatment and intensive livestock operation runoff and wastewater treatment.
9. The more common cold climate wetland sewage treatment process consisting of winter storage and seasonal wetland polishing requires substantial land area. The land requirements are likely excessive for many potential applications. It is concluded that passive technologies such as VSB wetlands which are capable of year-round treatment must be developed and demonstrated before wetlands wastewater treatment will be widely accepted.
10. Natural and wetlands wastewater treatment processes will be important not so much as replacements in applications currently served by mechanical technologies but rather as enablers of water quality improvement for pollution sources which are not now adequately treated. The technology represents potential BPT (best practicable technology) or BMP (best management

practice) for improving the quality of domestic wastewater from small communities and developments, for treating wastes from intensive livestock operations and for managing adverse impacts of urban and agricultural nonpoint source pollution.

The general conclusion is that wetlands and natural wastewater treatment technology will be important tools for environmental protection, however, greater understanding and local experience is required before the ecotechnologies will gain wide acceptance. The need for greater knowledge is reflected in the literature as is discussed in Section 8.3.

8.3 Research and Development Needs

Research and development of WWT technology requires effort at three levels:

1. Firstly, successful implementation of WWT in Western Canada requires development of local experience in WWT design, construction and operation. A basic understanding of wetlands functions and design procedures for related ecotechnology applications exists. Evidence supports the contention that the processes will be useful in the northern prairie climate, however, design procedures discussed in Sections 4 and 5 have been created for typically warmer and wetter climates than those prevailing on the Canadian prairies.
2. Secondly, even though the technology has been researched for 30 years or more, much is still not understood about wetland functions, processes and habitat values and the impacts of wastewater on them.
3. Thirdly, regulation and policy affecting natural wetlands and influencing created or constructed wetlands involves three levels of government in Canada. Work is needed to integrate, clarify and, where appropriate modify regulation and policies affecting constructed

wetlands. Notably, such effort will also require improved understanding of the function and value of wetlands.

A Western Canadian research and development program has the potential to generate new lower cost water quality improvement BMPs by creating local experience with the technology. Such a R & D program also has the potential to contribute to international understanding of this high potential ecotechnology.

Through this review it has become apparent that many of the WWT evaluations have been of relatively short duration frequently with limited sampling, flow measurement and material balance data. Many reported projects have involved natural wetlands (or modifications of them) or have evaluated constructed wetlands not necessarily designed according to guidelines and procedures ultimately presented in the literature. Thus, reported treatment effectiveness may not be consistent with proposed design procedures. This is believed to be especially true for stormwater wetlands.

The following paragraphs will review research and development needs identified in major recent reports.

8.3.1 Wetland Management in Alberta

Regulation and policy clarification and modification may at first seem unrelated to research and development, however, an understandable and technically sound regulatory and planning environment is essential to the wise application of wetland ecotechnologies. In Alberta alone, over 40 statutes are administered by at least 14 agencies (AWRC, 1993). Although the 1993 Interim Policy assigned primary responsibility for wetlands management activities including research to Alberta Environmental Protection, reports leading up to and including the 1993 Policy identify a number of non-technical and scientific issues including the following:

- Despite the Alberta Water Resources Commission's 1987 study, the inventory of Alberta's wetlands is not certain (AWRC, 1991 and 1993).

- Practical means of valuing wetlands, especially privately held wetlands need to be developed (AWRC, 1990 and 1993). For example, the farmer gains by draining wetlands but society loses, conversely society as whole may gain from mandated preservation but land owners may lose the freedom to use their lands for their profit. An exchange mechanism is required to compensate land owners for the imposition of land use controls that benefit the majority.
- The values of wetland and consequently the impacts of wetland loss are not understood (AWRC, 1990b).
- People are unaware of wetland benefits and consequently indifferent to loss or degradation of wetlands (AWRC, 1990b). The development of means to inform the public of wetlands value is required to overcome resistance to constructed wetlands as discussed in Section 8.1.3.
- Development of regional planning mechanisms for management of wetland objectives, preferably on a watershed basis is required (AWRC, 1993).
- Information on wetlands benefits, relationships and management techniques is needed requiring research in wetland ecology, hydrology, dynamics and processes , as well as development, landuse and climate change impacts on wetlands. Suggested research areas also included wetlands cost and benefits, wetland restoration, rehabilitation and management (AWRC, 1993).

8.3.2 Wetlands for Urban Stormwater R&D

In a 1992 update of a report for the US Environmental Protection Agency on wetland controls for stormwater pollution, Strecker et al. (1992) identified the following research needs:

- There is a need to obtain information on stormwater impacts on wetlands, particularly on biota.

- Additional studies are needed to establish maintenance needs such as sediment removal and plant harvesting in terms of requirement (if any), frequency and appropriateness (the author also adds costs).
- Additional studies to define wetlands effectiveness, long-term performance, seasonal characteristics and design guidelines for a range of wetland types are needed.
- More complete study reports are required containing information on wetland design and system characteristics (watershed landuse, climatological information, runoff volumes, wetland size).

8.3.3 Natural Wetlands and Urban Stormwater R&D

A February 1993 report on natural wetlands and urban stormwater (US EPA, 1993) identified unresolved issues arising from use of natural wetlands to receive urban runoff such as:

- Detailed understanding of wetland functions incorporating nutrient flows, hydrology, trophic dynamics, community structure, population distribution and abundance is not available for most types of wetlands. Greater understanding is needed of habitat processes and functions and how changes affect living organisms. Such understanding requires development of methods to assess wetland habitat functions.
- Better understanding of stormwater impacts on wetlands is required including:
 - impacts of level changes on wetland and habitat functions
 - stormwater quality and volume thresholds for impact on wetland functions
 - long-term impacts of water and sediment quality changes on biota

- potential enhancements to wetlands arising from stormwater inputs
- regional differences and concerns regarding natural wetlands particularly differences in hydrology and wetland type
- risks to public health from stormwater retention in wetlands
- methods to measure and assess changes in wetlands due to stormwater inputs and resulting affects on biological communities
- Resolution of stormwater management issues is required including:
 - stormwater management and wetland preservation programs may not cover entire systems and can be contradictory
 - most regulatory guidance focuses on quality not quantity management and hydrologic impacts must also be considered (author's note: this applies in the US, in Canada design focus by regulating authorities has been on quantity management with little quality control emphasis)
 - watershed level programs need to be implemented to minimize stormwater impacts
 - conditions where stormwater discharge to natural wetlands can be permitted need to be defined

8.3.4 WWT and Rural NPS Pollution R&D

Ethridge and Olsen (1991) of the US Environmental Protection Agency identified the following issues with respect to rural nonpoint source pollution:

- There is a need to develop a better understanding of the relationship between water quality and wetland functions. This work includes:
 - pollutant loading thresholds above which wetland structure and function are degraded

- the variation of the thresholds with wetland type and location
- the long-term sustainability of wetland treatment functions
- the recovery of wetland function following major event such as floods
- the effects of nonpoint source pollution on wildlife
- Better understanding of wetlands relationship with the landscape is required:
 - methods to evaluate wetland loading from watershed size and land use as well as models to evaluate impacts of wetlands wastewater treatment alternatives
 - evaluation of the need for restored or created projects to duplicate historical wetland patterns
 - evaluation of wetland positioning strategies in watersheds (near sources versus larger downstream systems) and integration of other BMPs to aid NPS pollutant control
 - further definition of the relationship between wetland function and hydrology and biogeochemistry
- Improved means to monitor and evaluate wetlands are needed:
 - development of indicators for wetland condition and function in response to stresses imposed by changing pollutant and physical conditions
 - definition of methods to monitor fate and impact of pollutants within watershed and evaluating BMP effectiveness
 - development of post-project evaluation methods aimed at continuous improvement of wetland restoration-creation procedures.

- Improved design and operational criteria are required for agricultural (animal waste and field runoff) treatment systems including means to predict longevity and maintenance needs.
- Improved technology transfer is needed including:
 - synthesis of literature and research in the form of clear and complete guidelines on integrating wetlands and NPS control programs
 - demonstration of integrated watershed level NPS-wetland programs
 - information on cost-benefit of BMPs
 - improved information exchange between levels of government and private entities.

8.3.5 Canadian Assessment of Runoff BMP R&D

Lastly, a Canadian workshop on runoff BMPs (OMOE, 1991) identified a substantial list of research and development needs including:

- Improved understanding of BMP operation and performance under winter snowmelt and spring runoff conditions and the effects of ice cover and freeze-thaw on BMPs.
- Development of designs to mitigate effects of ice on BMP level and flow control structures
- Better understanding of fate and impacts of dissolved and suspended solids including deicing chemicals, the effects of solid resuspension and relationships between particle size distribution and chemical transport and settling.
- Increased understanding of the relationship of runoff to urban landuse.

- Increased knowledge of the effects of runoff pollutants in the water column and sediments on biota.
- Development of standards for local BMP design.
- Evaluation of receiving waterbody improvements due to BMP implementation, effect of BMPs on bioaccumulation of stormwater pollutants, BMP impact on downstream watersheds and effects of BMPs on surrounding ecosystems.
- Definition of BMP operating and maintenance costs and assessment of long-term effectiveness of BMP types.
- Additional case studies of BMP systems from planning and design to operation and maintenance with evaluation of demonstrated effects on receiving waterbody quality are needed.

8.4 Recommendation: An Alberta WWT Development Program

It is the major conclusion of the author that ultimate acceptance of wetlands ecotechnology in Western Canada depends on the development of local expertise in system design and operation and on the full-scale demonstration of wetlands wastewater treatment effectiveness. It is recommended that WWT development projects be undertaken to serve real pollution control needs with sufficient monitoring and evaluation to produce research quality evidence and understanding of the processes.

The development program envisioned and described in this subsection will elevate local expertise to the current state of the art. It would further establish important international contributions, placing Western Canadian based research at the forefront of this emerging ecotechnology.

8.4.1 Sewage Treatment Applications

Sewage WWT systems have been successfully developed under northern prairie conditions. Needs currently exist in small municipalities for upgraded sewage treatment

that can be met with WWT. One or more projects should be undertaken with joint participation of local and Provincial agencies with the intent of evaluating design alternatives, testing actual system effectiveness, upgrading design procedures and publishing the case study report and ensuing WWT design - operation guidelines.

Objectives of such a project should be:

- Definition of effluent water quality objectives and expected beneficial impacts on receiving waterbodies.
- Identification, evaluation and application of known constructed wetland design procedures to the project including:
 - Evaluation of WWT technology (VSB, FWS) to be applied. Emphasis should be placed on technology which will permit year-round treating, thus eliminate the need for winter storage.
 - Evaluation of aesthetic and habitat design components to be integrated with water quality improvement design correlations.
 - Development of assessment and selection procedures for suitable wetlands vegetation.
 - Development of standardized construction and operating cost estimating procedures for the technology.
 - Development of funding and cost sharing mechanisms to transfer societal value to landowners and municipalities.
- Assessment of actual construction procedures and costs to seek improvements and to provide guidelines for future projects.
- Evaluation of project performance including:
 - Procedures and timing required to establish wetlands vegetation.

- Evaluation of effluent quality versus influent relative to objectives set for the project.
 - Evaluation of contaminant fate, particularly resolution of concerns about bioaccumulation of persistent toxic substances (PTS).
 - Evaluation of the seasonal and climatological impact on system performance. Assessment of expected high summer evapotranspiration rates on vegetation condition and effluent quality, over wintering characteristics of the wetland and of onset of autumn senescence on discharge contaminant levels.
 - Determination of the duration of effective annual treating periods.
 - Assessment of the potential for year-round treating.
 - Evaluation of how the WWT system could mitigate costs of upstream treatment facilities (e.g., by how much could sizes of storage-facultative lagoons be reduced).
 - Evaluation of aesthetic and/or wildlife value of the project.
 - Assessment of the long-term viability of the process.
- Publication of results in a form that transfers experience and appropriate design guidelines to other municipalities and wastewater treatment professionals.

In order to complete the assessment, a complete flow monitoring and water quality sampling system will be required. Data acquisition and evaluation will require higher levels of frequency, quality and statistical validity than is the norm for small sewage treatment systems. To that end Provincial and/or Federal assistance would be required for the municipality to justify the additional monitoring and evaluation. As long as appropriate design guidelines are followed, selection of a test system can be driven by

opportunity (e.g., carry out the pilot where and when an appropriate requirement for sewage treatment upgrading arises).

8.4.2 Runoff Quality Improvement Applications

Integration of wetlands wastewater treatment with runoff BMPs may be the most significant application of the technology in terms of environmental improvement. A number of tests are desirable including:

1. evaluation of WWT for livestock waste treatment
2. evaluation of constructed wetlands for control of agricultural NPS pollution
3. evaluation of complete urban stormwater wetland systems
4. evaluation of retrofit of wetland enhancements to existing urban stormwater BMPs

Projects should be developed in systems where the hydrology is well understood or can be established within the resources available to the project. In addition, test sites to evaluate items 1, 2 and 3 should be selected where land and financial resources are sufficient to permit installations consistent with published design guidelines. For runoff systems, the expectation should be that the project will require two percent of the drainage basin area. This latter feature is essential to producing quality research results and design procedure knowledge that can be transferred to others. Compromised sizing will likely yield less than optimal treatment results, information of questionable validity and could prematurely saddle the technology with a poor reputation.

Objectives of the studies will be broader and more varied than those for sewage WWT evaluation. Runoff water quality improvement pilot objectives will include:

- Definition of effluent water quality objectives and expected beneficial impacts on receiving waterbodies. The objectives should not only look at effluent concentrations but also at receiving waterbody assimilative capacity and seasonal tolerances (e.g., control of nutrients may be

more critical during the growing season than in the winter when wetland plants are dormant).

- Evaluation of public concerns and input regarding the project. The evaluation of public concerns should also include development of processes and materials to educate the public on the significance of the wetland approach to stormwater management.
- Evaluation of Provincial and municipal regulation and planning procedures as they apply to runoff wetlands. Part of the project assessment will need to define policy and regulation changes necessary to encourage wider use of the technology (assuming the pilots are successful).
- Identification, evaluation and application of known constructed wetland design procedures for the project including:
 - Evaluation of WWT technology (VSB, FWS) to be applied, along with the BMP design (e.g., shallow marsh, pond-wetland or ED wetland) most suitable to the application.
 - Evaluation of aesthetic, safety and habitat design components to be integrated with water quality improvement design correlations. Evaluation of landuse values and public input that should be integrated into the design.
 - Development of assessment and selection procedures for suitable wetlands vegetation.
 - Development of standardized construction and operating cost estimating procedures for the technology.
 - Development of funding and cost sharing mechanisms to transfer societal value to landowners and municipalities.

- Comparison of design water quality improvement expectations with project objectives and capabilities of conventional technologies.
- Assessment of actual construction procedures and costs to seek improvements and to provide guidelines for future projects. Cost evaluation should include comparison of capital and annual operating and maintenance expenses with conventional systems and assessment of cost-benefit in terms of pollutants removed and improvements in receiving waterbody.
- Evaluation of project performance including:
 - Procedures and timing required to establish wetlands vegetation. Evaluation should include assessment of appropriateness of selected plants, the extent of reinforcement plantings and expectations for ongoing vegetation management.
 - Evaluation of effluent quality versus influent and objectives set for the project. The evaluation should consider the fate of pollutants in the system, particularly the accumulation of PTS in sediments and assess the potential for detrimental bioconcentration in the wetland ecosystem.
 - Evaluation of the seasonal and climatological impacts on system performance. Assessment of expected high summer evapotranspiration rates on vegetation condition and effluent quality, need for summer base flows, Chinook runoff and spring meltwater treatment capability and of onset of autumn senescence on discharge contaminant levels.
 - Evaluation of seasonal treating effectiveness in terms of contaminant removal and total fraction of runoff treated

(wetland BMPs are not usually designed to treat major infrequent storm events and typically will include provision for bypass or storage of excess volumes). The evaluation should include assessment of the pollutant load reduction on downstream systems and the significance of the reduction.

- Evaluation of the impact of the wetland system on property values, its worth as an amenity and its impact for urban wildlife habitat. The evaluation should include assessment of safety features used to limit risk presented by deep water pools and water contaminants.
- Assessment of the long-term viability of the process.
- Publication of results in a form that transfers experience and upgraded design guidelines to other municipalities, landuse planners, environmental scientists and stormwater system design professionals.

To be of value as a research project, considerable monitoring of the pilot will be required. Data to be obtained will include watershed temperature, precipitation, evaporation and other climatological statistics. System influent and effluent should be monitored with continuous flow metres and flow-proportional samplers. Water level information should also be recorded. Sampling and analytical procedures should be scientifically and statistically valid.

It is expected that following construction of the system, one full growing season will be required to establish emergent vegetation. During the initial season it will be necessary to maintain saturated soil conditions while avoiding flooding of new shoots. Cost monitoring will be required for several seasons until the runoff wetland is fully established and a base maintenance cost is achieved. Monitoring of system performance should be carried out over at least two years following the initial growing season.

A sound runoff wetland BMP project will require an interdisciplinary approach involving planners, landscape architects, hydrologists, biologists and wastewater system designers. Because the technology offers the potential for cost effective control of runoff pollutants, it is expected that a joint developer-municipality-Provincial arrangement to cover project costs, analysis and data evaluation will be needed to complete the research. It is possible that some level of Federal environmental research funding can be attracted to such project.

8.4.3 Research Strategy, Evaluation and Integration

It is recommended that a strategy for developing WWT and related natural water quality treatment processes be established for the Province and that full scale pilots be carried out. The information and understanding to be gained from such a development program will not only define the ecotechnology but also will provide the basis for improved regulation of a variety of water pollution problems.

A means to support ongoing research at the pilots, to evaluate and consolidate learnings from the tests and to disseminate the information to technical and regulatory professionals is required. The piloting, development and evaluation program will span several years (at least three years and more likely five to seven years for each pilot), thus a system or means to ensure research continuity is required. It is, thus, recommended that one Alberta university establish a center of excellence to support long term development of water treatment ecotechnologies. Such a center would coordinate and integrate graduate student work in the field, produce consolidated study reports and act as an information clearinghouse for practicing professionals.

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10. PERSONAL COMMUNICATIONS

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- Stecko, J.R. Pierre. 1993. Project Biologist, EVS Environmental Consultants, North Vancouver, British Columbia. Personal communications and research progress reports submitted on CANMET Project No. 23440-1-9151.
- Usher, P.K. 1993. Construction Supervisor, Ducks Unlimited, Brooks, Alberta. Personal communication on wetlands creation practices of Ducks Unlimited.
- Wasylyshyn, P. 1993. Parks Planner, Parks Department, City of Red Deer. Personal communication and City of Red Deer mosquito control program documents.

11. APPENDIX A: STORMWATER CALCULATIONS

Small Basin Hydrology

Purpose:

Calculations below approximate runoff data for small urban land areas based on US Soils Conservation Service procedures (US SCS, 1986). Calculations are done for watersheds with connected impervious areas for the purpose of establishing design volumes for stormwater wetlands and peak runoff rates. THESE RESULTS SHOULD ONLY BE USED FOR CONCEPTUAL DESIGNS. Detailed design of stormwater systems and related retention/detention basins must be evaluated by simulation with a continuous hydrology model that incorporates snow melt, infiltration and evapotranspiration. Shaded sections highlight user input data.

		Symbol
Site Data	Length	500 m
	Width	480 m
	Area	240,000 sq m
	Average Slope	2.5%
	Area - buildings	11,150 sq m
	Area - roads/paths	14,200 sq m
	Impervious Area	11%
	Soil Group	C Sandy clay loam
	Cover Type	Good condition meadow with continuous grass cover
	Curve Number	71
		L
		W
		A
		s
		Pimp
		CN
Design Storm for Treatment System Sizing	Return Period - 1 in	2 year - 24 hr event
	Intensity	36.9 mm in 24 hours
	Source:	Environment Canada, 1990 - Calgary Int'l. Airport
		Pd
Design Storm for Flood Management	Return Period - 1 in	100 year - 24 hr event
	Intensity	90.7 mm in 24 hours
	Source:	Environment Canada, 1990 - Calgary Int'l. Airport
		Pmax

Runoff Calculations - Stormwater Treatment Design

Assumptions:

All impervious areas are connected.
Average antecedent runoff conditions.

Composite CN	Curve No.	74	CNc
	where $CN_c = CN + (P_{imp} * (98 - CN))$		
Runoff Retention	Potential Ret.	3.5 inches	Sr
	where $S_r = (1000/CN_c) - 10$		
Runoff Volume	Runoff	3.29 mm	Rd
	where $R_d = (P - 0.2S_r)^2 / (P + 0.8S_r)$		
	P [=] inches		
	Runoff - volume	789 cu m	Qd

Runoff Calculations - Flood Design

(assumptions, CNc and Sr same as previous calculation)

			Symbol
Runoff Volume	Runoff	32.51 mm	Rmax
	where $R_d = (P - 0.2S_r)^2 / (P + 0.8S_r)$		
	P [=] inches		
	Runoff - volume	7802 cu m	Qmax

Time of Concentration & Travel Time

Flowpath	Length	650 m	Lfp
	Type	sheet flow into shallow flow in grass swales	

Travel Time

Sheet Flow	Manning n	0.24 dense grass*	nsf
	(first 90 m) Travel time	0.78 hours	Tsf
where $T_{sf} = (0.007 (nL)^{0.8}) / (Pd^{0.5})(s^{0.4})$			
		Pd [=] inches	L [=] feet
Shallow Flow:	Velocity	0.78 m/s	Vshf
	where $V_{shf} = 4.9178 (s^{0.5})$ for unpaved surfaces*		
	Travel Time	0.20 hours	Tshf
* It is recommended stormwater in small developments be managed by sheet flow over vegetation and shallow flow in grassed swales as this improves water quality. Alternate calculations are required for paved and piped flow systems.			
	Total Travel Time	0.98 hours	Tt

Peak Discharge Calculation (Rainfall Type II)

Initial Abstraction	Pre-runoff loss	0.71 inches	la
	where $la = 0.2S_r$		
Pond/Swamp Factor	Percent Affected	0%	
	Adj. Factor	1	Fp
Abstraction/Precip	Design Storm	0.49	IPd
	Max. Storm	0.20	IPmax

Peak DischargeStormwater Treatment Design Case

Unit Peak Disch.	170 csm/in	qu
source: Exhibit 4-II, US SCS, 1986		
Peak Discharge	0.06 cu m/sec	qpd
<u>Flood Event Design Case</u>		
Unit Peak Disch.	325 csm/in	qumax
source: Exhibit 4-II, US SCS, 1986		
Peak Discharge	1.09 cu m/sec	qpmx
where $q_p = q_u \cdot A \cdot R \cdot F_p$ and A [=] sq miles, R [=] in.		
q _p [=] cu ft/sec		
Final product converted to cu m in spreadsheet		

Runoff Wetland Design

Calculation Application

Four types of stormwater wetlands are discussed by Schueler (1992):

1) Shallow Marsh 2) Pond/Wetland 3) ED Wetland 4) Pocket Wetland

This procedure is designed to primarily evaluate Type 1 designs, although it is readily usable for Types 3 and 4. Type 2 should include evaluation of the wet pond component of the system (not provided for in the following).

Design Conditions

		<u>Symbol</u>
Base Flow	31.4 cu m/d	Q_b
Treatment Vol.	789 cu m	V _t

where V_t: -Default is Q_d from preceding hydrology calculations
 -Suggested minimum is 6.3 watershed mm
 -Is ideally obtained from continuous simulation of historical events and set to volume equivalent to the largest event in the range of storms that accounts for 90% of runoff.

Base Flow HRT	25 days	HRT _b
where $HRT_b = V_t / Q_b$		

-HRT ideally 5-10 days for BOD and nutrient removal.

Detention Vol.	789 cu m	ED
ED Period	24 hours	ted

Comment: -ED selected on basis of flood control needs. System without ED works like a plug flow (PF) reactor (e.g., inflow from current event displaces treated volume from previous event).
 -Default sets ED = V_t so that current event inflow is detained to allow for non-ideal conditions and adequate sedimentation.
 -ED period is the time allowed for gradual release of event volume (suggested as 24-48 hours).

Flood Event Bypass Unless provision is made, major events (1:100 year storm) can result in flows that will washout the wetland. Diversion of high flows around the system is required if V_t + ED is much less than flood event volume (Q_{max}).

V _t + ED	1578 cu m
Q _{max}	7802 cu m

Bypass is required

Peak Inflow	0.06 cu m/s	Q _p
-------------	-------------	----------------

where $Q_p = Q_b + q_{pd}$

-Default is base flow plus peak discharge from stormwater treatment design storm.

Sediment Forebay Design

Design Particle	Coarse silt	0.05 mm diameter	<u>Symbol</u> dp
		0.002 m/s settling vel.	vs
If settling forebay is the only stormwater BMP planned, 0.02 mm particles with 0.0002 m/s settling velocity are recommended by several jurisdictions. Ideally design should be based on measured sediment loads for the watershed.			
Sediment Bay Area	Min. Surface	31 sq m	Afb
	where $A_{fb} = 1.2 \cdot Q_p / v_s$		
Sediment Bay Depth	Depth	1.1 m	
	Min. Depth*	0.6 m	dsbm
* Based on a 0.5 m max. sediment storage depth Min. flow depth should be 0.6 m at maximum sediment depth.			
Scouring Velocity	Scouring velocity at Q_p may control L:W of settling basin. Relationship is used to calculate minimum acceptable width of forebay.		
	where $v_{hs} = \sqrt{(8 \cdot k \cdot g \cdot d_p \cdot (spgr - 1))}$		
	k=	0.04 for sand	
	spgr=	2.65 for sand	
	g=	9.81 m/s*s (gravity)	
	Max. Horiz. Vel.	0.016 m/s	vhs
	Min. Width	6.0 m	
Forebay Layout	Aspect	2 :1 (L:W)	LWfb
	Width*	3.9 m	
	Length	7.9 m	
*Scouring velocity controls forebay (sediment basin) dimensions. Forebay from overall wetland layout must not be smaller than:			
	Width	7 m	Wfbmin
	Length	13 m	

Stormwater Wetland Design

Wetland Layout and Depth Allocation

<u>Suggested factors by the Washington DOE (1992)</u>		
	Zone Depth (m)	Relative Area
Hi marsh	0 - 0.15	50%
Lo Marsh	0.15 - 0.30	15%
Deep pools	0.60 - 0.90	15%
Forebay/Micropool	0.90 - 1.80	20%
Avg. System Depth	0.62 m (maximum)	
Overall L:W	3 :1 (3:1 is minimum)	
Forebay up to	25% of total system area	

dmax
LWw

Stormwater Wetland Design

(continued)

Symbol**System Size and
Area Allocation**

Zone Type	Depth (m)	Relative Area	Area (sq m)	Volume (cu m)
Forebay	1.1	10%	225	248
Hi Marsh	0.1	50%	1127	113
Lo Marsh	0.3	30%	676	203
Deep Holes	0.9	5%	113	101
Micropool	1.1	5%	113	124
			2254	789

Average Depth

0.35 m

davg

Surface Area

2254 sq m

Asw

Layout

Forebay L:W **3.5 :1**
 Width 8 m (min. default = Wfbmin)
 Length 28 m
 HRT at Qp 1.2 hours

Wetland L:W **3 :1**
 Total Area* 1916 sq m
 *Total area excluding forebay and micropool.
 Overall Width 25 m
 Overall Length 76 m

Micropool L:W **1 :1**
 Width 11 m
 Length 11 m

The wetland drain system is located in the micropool which minimizes plugging by surrounding vegetation. In typical designs the micropool is a nearly circular deep section at the outlet section of the wetland. Thus the overall wetland area is extended by the area of the micropool.

Wetland + Micropool Total Area 2028 sq m
 Width 25 m Ww
 Length 80 m Lw
Details Berm Slope 3 :1 Minimum Sw
Freeboard* 0.75 m fbdw

*Subject to review by civil engineers, it is suggested that freeboard above normal operating level be at least 0.6 m with emergency spillway overflow 0.3 m below berm elevations. Extended detention can be added by increasing freeboard.

ED Volume

ED Depth 0.45 m (fbdw - 0.3) ded
 Vol. to Spillway 812 cu m

Stormwater Wetland Design

(continued)

**Stormwater Wetland
Summary**

<u>Forebay Dimensions</u>		
	<u>Normal level (m)</u>	<u>Berm Top (m)</u>
Width	8	13
Length	28	33
<u>Wetland Dimensions</u>		
	<u>Normal level (m)</u>	<u>Berm Top (m)</u>
Width	25	30
Length	80	85

Stormwater - Treated Water Storage Pond Sizing**Purpose:**

In dry climates or in areas where water resources are limited it may be desirable to store a portion of the runoff and/or base flow. The following calculation permits estimation of rectangular ponds with rounded corners. If storage provision is being made for primary domestic sewage the pond should not be deeper than 2.5 m. Suggested design parameters include 0.6 m freeboard, 4:1 external berm slopes and 3:1 internal berm slopes.

Input parameters

Stormwater Vol.	-	cu m
Base Flow	31.4	cu m/d
Base Flow	365	d
Storage	11,461	cu m
Pond Vol.	11,461	cu m

SymbolV_{sw}Q_bt_{bf}V_{bf}V_{pond}

Pond Depth	2.5	m
Pond Aspect	3 :1	L:W
o/s Berm Slope	4 :1	(horiz.:vert.)
i/s Berm Slope	3 :1	(horiz.:vert.)
Freeboard	0.6	m
Excavation Depth	2	m
Berm top width	3	m

d_{pond}LW_pS_{po}S_{pi}f_{bdp}d_{exc}w_t**Water Surface
Dimensions**

General Relationship (per US EPA, 1983):

$$\text{Vol.} = (d/6)[(LW) + (L-2sd)(W-2sd) + 4(L-sd)(W-sd)]$$

where: d= depth, L= length, W= width, s= berm slope

Width 44.1 m

W_p

Length 132.3 m

L_p

volume check 11,461 cu m

Pond Dimensions

	<u>Width (m)</u>	<u>Length (m)</u>
Inside Base	29	117
Inside Berm Top	48	136
Outside Berm Top	54	142
Outside Berm Base	62	151

12. APPENDIX B: SEWAGE LAGOON CALCULATIONS

Alberta Sewage Lagoon Calculation System

(shaded sections indicate required user input)

Design Requirements: Alberta Environmental Protection specifies that small lagoon based sewage treatment systems must have a 60-day pretreatment cell and a storage cell with one year capacity (Alberta Environment, 1991). Rational correlations can be used to predict performance, although literature on effluent quality suggests that design of downstream treatment facilities should be based on actual measured lagoon effluent parameters or on those published in the literature.

Design Information and Assumptions

				Symbol
Residents Served		120		
Peak Usage	Additional users	100		
Events	Duration	10 hours		tev
	Frequency	52 per year		fev
Wastewater Rate		0.25 cu m/capita-day		
Assumed Wastewater Composition	BOD ₅	220 mg/l		Co
	Susp. Solids	220 mg/l		Cso
	Total N	40 mg/l		Cno
	Ammonia	15 mg/l		Cao
	Fecal Coliform	1.00E+08 number/100 ml		Cfc
Expected Lagoon Effluent Quality (per US EPA, 1983)	BOD ₅	30 mg/l		
	Susp. Solids	30 mg/l		
	Total N	15 mg/l		
	Ammonia	-- mg/l		
	Fecal Coliform	-- number/100 ml		
Treated Water Quality Objectives	BOD ₅	10 mg/l		Ce
	Susp. Solids	10 mg/l		Cse
	Total N	2 mg/l		Cne
	Ammonia	2 mg/l		Cae
	Fecal Coliform	200 number/100 ml		
Assumed Inlet Temperature	Winter "Design"	15 deg. C		Td
	Summer	20 deg. C		Tsd
Average Ambient Temperatures (deg.C) (1961 - 1990 Climate Normals for Calgary Int'l. Airport)	January	-9.6	July	16.4
	February	-6.3	August	15.7
	March	-2.5	September	10.6
	April	4.1	October	5.7
	May	9.7	November	-3.0
	June	14.0	December	-8.3

Determination of Treatment and Storage Lagoon Sizes

			<u>Symbol</u>
Base Inflow Rate	Flowrate	30 cu m/d	Q _b
Event Volume	Volume/event	10.4 cu m	Q _{ev}
Annual Volume	Volume/year	11,492 cu m/yr	
	Daily Average	31.5 cu m/d	Q _{av}
Note: Detailed design should account for precipitation and evaporation.			
	Precipitation*	398.8 mm/yr (mean)	
	Lake Evap.**	522.9 mm/yr (calculated)	

Conclusion: No allowance is needed for precipitation into lagoons.

* 1961-1990 Climate Normals for Calgary Int'l. Airport, Environment Canada.

** 1951-1980 Climate Normals for Calgary Int'l. Airport, Environment Canada.

Lagoon Depths and Berm Constraints	Treatment Cell	1.5 m max. depth	dtc
	Storage Cell	2.5 m max. depth	dsc
	Min. Level	0.15 m	dmin
Berm	-Outside Slope	4 :1 horiz.: vert.	So
	-Inside Slope	3 :1 horiz. : vert.	Si
	-Top Width	3 m	W_{top}
	-Freeboard	0.6 m	Fbd

Per Provincial Guidelines (Alberta Environment, 1991).

Lagoon Sizing

$$Vol = d/6[(L+W) + (L-2Sd)(W-2Sd) + 4(L-Sd)(W-Sd)]$$

Per US EPA (1983) for rectangular basins with sloped sides and rounded corners.

For square ponds L=W (W= width at water surface)

$$W = (Sd) + (0.5/d)SQRT[(-4/3)(S^2)(d^4) + 4dV]$$

60-Day Cell Volume	1889 cu m	V _t
Surface Dimen.	39.9 m square	W _t
Top of Berm	43.5 m square	
Base of Berm	30.9 m square	
O/S Berm Base*	58.3 m square	

*Assuming bottom of lagoon is 1 m below surrounding grade.

Vol Check 1889 cu m

Storage Cell Volume	11,492 cu m	V _s
Surface Dimen.	75.2 m square	W _s
Top of Berm	78.8 m square	
Base of Berm	60.2 m square	W _{sb}
O/S Berm Base*	93.6 m square	

*Assuming bottom of lagoon is 2 m below surrounding grade.

Vol Check 11,492 cu m

Storage Cell	Surface Width	61.1 m square	W _{sm}
at Min. Drawdown	Volume	551 cu m	V _{smin}
	Depth	0.15 m	

Calculation of Lagoon Performance

Loading Rates (Kg/d)			Symbol
	Average	Peak	
BOD ₅	6.9	8.9	
Susp. Solids	6.9	8.9	
Nitrogen	1.3	1.6	
Ammonia	0.5	0.6	

Treatment Lagoon Water Temperature	$T_w = (A_f T_a + Q T_i) / (A_f + Q)$		
	$f = 0.5$		
	Surface Area = 1591 sq m		
	At		

Month	T _a ** (deg C)	T _w (deg C)	
		T _i = T _d	T _i = T _{sd}
January*	-9.6	0.5	0.5
February*	-6.3	0.5	0.5
March	-2.5	0.5	0.5
April	4.1	4.5	4.7
May	9.7	9.9	10.1
June	14.0	14.0	14.2
July	16.4	16.3	16.5
August	15.7	15.7	15.9
September	10.6	10.8	11.0
October	5.7	6.1	6.2
November*	-3.0	0.5	0.5
December*	-8.3	0.5	0.5

*minimum temperature under ice cover assumed to be 0.5 deg. C.
 **mean ambient air temperature for the month.

Nov.-Mar. Temp.	0.5 deg C	T _{ww}
April-Oct. Temp.	11.1 deg C	T _{ws}

Treatment Lagoon Performance	Complete Mix Model for BOD per WPCF (1990)		
	$C_e = C_o[1/(1 + kct)]$		
	where $k_c = 1.2 (1.085)^{(T-20)}$		
	Winter constant	0.245 1/d	k _{cw}
	Summer constant	0.582 1/d	k _{cs}
	Residence time	60 days	HRT _t
	winter	summer	
Effluent BOD (mg/l)	14	6	

Nitrogen Removal Model per WPCF (1990)		
$C_{ne} = C_{no} \exp[-kT(t + 60.6(pH-6.6))]$		
where $kT = 0.0064 (1.039)^{(T-20)}$		
assume pH =	7	pH
Winter constant	0.003 1/d	kT _w
Summer constant	0.005 1/d	kT _s
	winter	summer
Effluent N (mg/l)	31	27

Calculation of Lagoon Performance

(continued)

Symbol**Treatment Lagoon Performance**

Bacteria Removal Model per Reed et al. (1988)

$$C_{fce} = C_{fc} / (1 + k_f t)$$

$$\text{where } k_f = 2.6 (1.19)^{(T-20)}$$

$$\text{assume } t = \text{HRT}/2$$

Winter constant	0.087 1/d	k _{fw}
Summer constant	0.556 1/d	k _{fs}
	<u>winter</u> <u>summer</u>	
Fecal coliforms*	27,595,428 5,651,674	

*lagoon effluent number / 100 ml

Storage Lagoon Performance

Above calculations repeated using treatment lagoon effluent concentrations as inlet values for the storage lagoon. The same rate constants are used.

Determinations done assuming full and minimum levels in the lagoon.

Residence Time

Max. level	365 days	HRT _{max}
Min. level	18 days	HRT _{min}

BOD Removal

values in mg/l

		<u>winter</u>	<u>summer</u>
Influent	-per above	14	6
Effluent	-at max. level	0.16	0.03
	-at min. level	2.66	0.55

N Removal

values in mg/l

		<u>winter</u>	<u>summer</u>
Influent	-per above	31	27
Effluent	-at max. level	9.5	4.6
	-at min. level	27.3	22.6

Bacteria Removal values in number/100 ml

		<u>winter</u>	<u>summer</u>
Influent	-per above	27,595,428	5,651,674
Effluent	-at max. level	1,626,956	55,109
	-at min. level	15,631,399	962,845

Note: Based on US EPA (1983) performance expectations and reports in the literature, the predicted BOD removal beyond 30 mg/l is likely optimistic. The nitrogen removal presumes the major pathway is ammonia volatilization, thus the results for winter condition (ice cover) may be optimistic and the summer results conservative (WPCF, 1990). The calculations do verify if selected lagoon sizes are able to meet the performance effluent quality predicted by the US EPA (1983). Predicted fecal coliform levels are higher than typically reported.

13. APPENDIX C: FWS WETLAND CALCULATIONS

FWS Wastewater Treatment Wetland Design System

(shaded sections indicate required user input)

Design Information and Assumptions

			Symbol
Residents Served		120	
Wastewater Rate		0.25 cu m/capita-day	
Peak Usage	Additional users	100	
Events	Duration	10 hours	tev
	Frequency	52 per year	fev

Assumed Wastewater Composition	BOD ₅	220 mg/l	Co
	Susp. Solids	220 mg/l	Cso
	Total N	40 mg/l	Cno
	Ammonia	15 mg/l	Cao
	Fecal Coliform	1.0E+08 number/100 ml	Cfc

Assumed Pretreatment and Wetland Influent Quality			
Expected Lagoon	BOD ₅	30 mg/l	Col
Effluent Quality	Susp. Solids	30 mg/l	Csol
(per US EPA, 1983)	Total N	15 mg/l	Cnol
Assumed Septic Tank	BOD ₅	140 mg/l	Cos
Effluent Quality	Susp. Solids	80 mg/l	Csos
	Total N	40 mg/l	Cnos
	Ammonia	15 mg/l	Caos
	Fecal Coliform	1.0E+08 number/100 ml	Cfcs

Treated Water Quality Objectives	BOD ₅	10 mg/l	Ce
	Susp. Solids	10 mg/l	Cse
	Total N	2 mg/l	Cne
	Ammonia	2 mg/l	Cae
	Fecal Coliform	200 number/100 ml	Cfce

Mean Daily Ambient Temperatures (deg.C) (1961 - 1990 Climate Normals for Calgary Int'l. Airport)	January	-9.6	July	16.4
	February	-6.3	August	15.7
	March	-2.5	September	10.6
	April	4.1	October	5.7
	May	9.7	November	-3.0
	June	14.0	December	-8.3
	Averages April-Oct.	10.9	deg. C	
	May-Sept.	13.3	deg. C	
	Nov.-March	-6.8	deg. C	

Design Water Temperatures Most cold climate FWS systems are designed for with winter storage and growing season wetland treatment. Assumed FWS water temperatures are as follows:

Winter Temp.	2 deg. C (under ice)	T _{dw}
Summer Temp.	13 deg. C	T _{ds}

Determination of FWS System Feed Rate

Base Condition	Flowrate	30 cu m/d	Q _b
Event Volume	Flow/event	10.42 cu m	Q _{ev}
Annual Volume	Flow/year	11,492 cu m/yr	
Average Inflow	Avg. Daily Rate	31.5 cu m/d	Q _{av}

Two possible means to handle wastewater will be evaluated. Firstly, conventional constructed FWS wetland approach for cold climates involves pretreatment and storage in upstream lagoons with wetland processing in warmer weather (May to late October or early November). A second and less proven (in cold prairie climates) method is to pretreat the wastewater (e.g., septic tanks for small systems) with year-round treatment in the FWS wetland. Winter operation is facilitated by deeper flooding of the wetland and possibly use of insulating mulch over ice cover.

Evaluation of Design Condition #1

Design FWS wetland to handle daily flows plus drawdown storage pond in td days assuming an average ambient and water temperature of T_d C. Such a system could probably process for an additional 80 days per year at lower temperatures and influent rates.

	Drawdown Period	100 days	td
	Storage Vol.	11,492 cu m	V _{st}
Design Conditions	Design Flow	146 cu m/d	Q _d
	where $Q_d = Q_{av} + (V_{st} / t_{dd})$		
	Design Temp.	13 deg C	T _{1d}
Assumed Wetland Factors (per Reed et al., 1988)	System Porosity	0.75	n
	Spec. Surf. Area	15.7 1/m	A _v
	BOD Factor	0.52	A
	Rate Constant	0.002925001 1/day	K _T
	where $K_T = 0.0057 [1.1^{(T-20)}]$		
Hydraulic Residence Time (per Reed et al., 1988)	HRT	9.2 days	t
	where $t = [(\ln(C_{oi}) - \ln(C_e) - \ln(A))] / (65K_T)$		
	Required Vol.	1349 cu m	V _{1w}
Wetland Design		<u>assume</u>	
	Water depth	0.1 m	ds
	Aspect	4 : 1 L:W	LW
		<u>calculated</u>	
	Required Area	17,993 sq m	A _{1w}
	System Width	67 m	
	System Length	268 m	

Evaluation of Design Condition #1

(continued)

Symbol**Wetland Layout**

It is recommended the system be laid out as *N1trn* sets of parallel marshes.

Headers should permit flow adjustment between parallel beds. Thus each marsh should have the following dimensions:

Parallel trains	2	N1trn
Number of Cells	4	N1cell
Cell Width	34 m	
Cell Length	134 m	

For cold weather operation water depth should be increased to 0.45m. Additional freeboard of 0.6 m should be provided. Inside slopes to be 3:1 and access paths along top of berms should be 3 m wide.

Max. Depth	0.45 m	dmax
Freeboard	0.6 m	fb
Berm Slopes	3 :1 (horiz.:vert.)	Si
Berm top	3 m	top

Thus external dimensions of the wetland cells are as follows:

Cell Width	46 m
Cell Length	146 m

Expected Nitrogen Removal
(per WPCF, 1990)

Hyd. Loading	0.8 cm/d	HLR1
Effluent N	2 mg/l	
where $C_e = (0.193C_{no}) + (1.55HLR) - 1.75$		

Expected Effluent Bacteria Level
(per Reed, 1988)

Calculations for sewage lagoon performance for medium strength domestic wastewater in cold climate conditions indicate that effluent fecal coliform levels could be between 55,000 and 16,000,000 per 100 ml.

Inf. Fecal Colif.	100,000 number/100 ml	Cfcol
Rate Const.	0.77 1/day	kf

where $k_f = 2.6 (1.19)^{(T-20)}$

Effl. Fecal Colif. 13005 number/100 ml

where $C_{fce} = C_{fcol} / [1 + (t/2)k_f]^n$

n = number of cells in series (2)

t = residence time per cell

This design does not meet contact recreation guidelines (200 fecal coliforms/100 ml). Disinfection will be required for irrigation or recreation uses of the treated water.

Evaluation of Design Condition #2

Design FWS system to operate throughout the year. Based on results of Ontario pilots (Herskowitz, 1986; Miller, 1989) the wetland should be flooded to 45 cm depth and a 15 cm free water zone maintained under winter ice cover. Reed et al. (1988) suggests minimum water temperature of 2 C, however Listowel, Ontario pilot results (Herskowitz, 1986) show minimum water temperatures near 0.5 C. Formulas and factors per Condition #1 procedures except as noted.

			<u>Symbol</u>
Design Conditions	Flooded Depth	0.45 m	
	Ice-Free Depth	0.15 m	dw
	Porosity	0.75	n
	Water Temp.	2 C	T _{dw}
Septic tanks shall be designed to provide surge detention of extra volumes from events. Initial assumption is that event volume will be released from the septic tanks in the 24 hours (tp) following the event. Thus:			
	Incremental flow =	7.4 cu m/d	Q _i
	where $Q_i = 24 \cdot Q_{ev} / (t_{ev} + t_p)$		
	Design Flow	37.4 cu m/d	Q _{2d}
Hydraulic Residence Time	Rate Const	0.00103 1/day	K _{2T}
	HRT	49 days	HRT ₂
Wetland Size	Wetland Vol.	1,846 cu m	V _{2w}
	Area (@15 cm)	16,407 sq m	A _{2w}
	Aspect Ratio	4 :1 L:W	LW _{2c}
	Width	64 m	
	Length	256 m	
Wetland Layout	Cell depth, freeboard and side slopes assumed to be the same as those for Condition #1. Two parallel sets of multiple cells in series are recommended.		
	Max. Depth	0.45 m	d _{max}
	Freeboard	0.6 m	fbd
	Side slope	3 :1 (horiz.:vert.)	S _i
	Berm top	3 m	top
	Number of trains	2	N _{2trn}
	Number of cells	4	N _{cell2}
	Cell Width	32 m (inside berm base)	
	Cell Length	128 m (inside berm base)	
	External dimensions of the wetland cell including top of berm access paths.		
	Cell Width	44 m	
	Cell Length	140 m	

Evaluation of Design Condition #2

(continued)

			<u>Symbol</u>
Expected Nitrogen Removal	Hyd. Loading	0.23 cm/day	HLR2
	Influent N	40 mg/l	Cnos
	Effluent N*	6 mg/l	
* Correlation from WPCF (1990) does not have a temperature coefficient and likely over predicts cold weather performance (much of the N will be ammonia and ice cover will limit oxygen available for nitrification).			
Expected Effluent Bacteria Level	Inf. Fecal Colif.	1.00E+08 number/100 ml	Cfcs
	Rate Const.	0.114 1/day	kf2
	Effl. Fecal Colif.	1.73E+07 number/100 ml	
In cold weather fecal coliform reduction may be limited.			
Disinfection of treated water is required for irrigation or recreation uses.			

14. APPENDIX D: VSB WETLAND CALCULATIONS

VSB Wastewater Treatment Wetland Design Calculator

(shaded sections indicate required user input)

Design Information and Assumptions

			Symbol	
Residents Served		120		
Peak Usage	Additional users	100		
Events	Duration	10 hours	tev	
	Frequency	52 per year	fev	
Wastewater Rate		0.25 cu m/capita-day		
Assumed Wastewater Composition	BOD5	220 mg/l		
	Susp. Solids	220 mg/l		
	Total N	40 mg/l		
	Ammonia	15 mg/l		
	Coliform	1.00E+08 number/100 ml		
Assumed Septic Tank Effluent Quality	BOD5	140 mg/l	Co	
	Susp. Solids	80 mg/l	Cso	
	Total N	40 mg/l	Cno	
	Ammonia	15 mg/l	Cao	
	Fecal Coliform	1.00E+08 number/100 ml		
Treated Water Quality Objectives	BOD5	10 mg/l	Ce	
	Susp. Solids	10 mg/l		
	Total N	1 mg/l	Cne	
	Ammonia	2 mg/l	Cae	
	Fecal Coliform	200 number/100 ml		
Assumed Treatment Temperature	Winter "Design"	2 deg. C	Td	
	Summer	13 deg. C	Tsd	
VSB Media Assumptions	Type	washed 1.25 - 2.5 cm gravel		
	Porosity	38%	ng	
	Conductivity	1500 m/day	ks	
BOD5 Removal Rate Constant	Std. Constant	1.104 1/d @ 20 C	Kstd	
	Temp. Coef.	1.06 dimensionless	ftc	
	Safety Factor	75%	fss	
Average Ambient Temperatures (deg.C) (1961 - 1990 Climate Normals for Calgary Int'l. Airport)	January	-9.6	July	16.4
	February	-6.3	August	15.7
	March	-2.5	September	10.6
	April	4.1	October	5.7
	May	9.7	November	-3.0
	June	14.0	December	-8.3

Determination of Wastewater Flows

			<u>Symbol</u>
Base Conditions	Flowrate	30 cu m/d	Q _b
Event Load	Event Volume	10.4 cu m/event	Q _{ev}
Annual Load	Annual Volume	11,492 cu m/year	
Average Flow	Avg. Daily Flow	31.5 cu m/d	Q _{av}
Event Processing	Septic tanks shall be designed to provide surge detention of extra volumes from events. Initial assumption is that event volume will be released from the septic tanks in the 24 hours (tp) following the event. Thus:		
	Incremental flow =	7.4 cu m/d	Q _i
	where $Q_i = 24 \cdot Q_{ev} / (t_{ev} + t_p)$		
Design VSB Feed	Flowrate	37.4 cu m/d	$Q_d = Q_b + Q_i$

Determination of VSB Loading

		<u>at Avg. Flow</u>	<u>At Design Flow</u>
VSB Loading	BOD ₅	4.4	5.2
in kg/d	Susp. Solids	2.5	3.0
(based on assumed septic tank effluent composition)	Total N	1.3	1.5
	Ammonia	0.5	0.6
Sizing based on WPCF (1990) factors at Q_d	Area for BOD ₅	0.065 ha @ 80 kg/ha-d	
	Area for N	0.050 ha @ 60 kg/ha-d	
	Area for flow	0.062 ha @ 6 cm/d	

VSB Design Calculations

Step 1: Define Bed	Media	washed 1.25 - 2.5 cm gravel	
	Depth	0.8 m	d
	Primary Plant	Bulrush	
	Bed Temperature	2 deg. C	T _d
	Design Flow	37.4 cu m/d	Q _d
Step 2: Calc. Area	Rate Constant	0.290 1/d	KT
	where $KT = f_{ss} \cdot K_{std} (f_{tc})^{(T_d - 20)}$		
	Surface Area	1118 sq m	As
	where $As = Q_d \cdot \ln(C_o/C_e) / (KT \cdot d \cdot n_g)$		
Step 3: Aspect Ratio and Dimensions	Aspect Ratio	2 :1 L:W	Ra
	Width	24 m	W
	Length	47 m	L

VSB Design Calculations

(continued)

Nitrogen Removal

Check (continued)

Method 2

The second method is reported to yield more reasonable results and is based on oxygen transfer capability of the plant roots. About 5 gm oxygen is required to convert 1 gm ammonia to nitrate (Reed, 1993).

Step 1: Calculate area needed to reduce BOD to 20 mg/l

BOD Area 326 sq m

where $KT = K_{std} \cdot (f_{tc})^{(T_{sd}-20)}$

HRT 2.7 d

Step 2: Calculate Oxygen Available

Avg. O₂ Avail. 7.5 gm/cu m-d per Reed (1993)

O₂ per sq m 6 gm/sq m-d

where O₂/sq m = 7.5 * d

Step 3 : Calculate Oxygen Required

O₂ Required 2427.941176 gm/d

where O₂ req = Qd*(C_{ao}-C_{ae})*5

Step 4: Calculate Area for Nitrification

NH₃ Area 405 sq m

Step 5 : Calculate Total Area Needed

Total Area 730 sq m

L:W 2 :1

Dimensions W 19 m

L 38 m

This is less than area needed for winter BOD removal, thus design is adequate for controlling summer ammonia levels. It is unlikely the VSB system can meet consistent ammonia objectives on a year-round basis.

Layout Design

Single beds are reportedly used for single residences and multiple cells in series and parallel are used for larger flowrates.

Parallel Trains 2 Ntrn

of Cells 4 Ncell

Weed control is typically accomplished by occasionally flooding the wetland which kills weeds but not the desired vegetation. Freeboard should account for flooding and suitable safety factor. A 3 m wide access path is suggested around the cells.

Flooding depth 0.3 m dfl

Freeboard 0.6 m fbd

Top path width 3 m top

Inside slope 3 :1 (horiz.:vert.) Si

Inlet and outlet sections should be provided and consist of a 1 m length of bed filled with 50-100 mm washed coarse stone to full depth. Inlet distributors and outlet collectors are typically 100 mm Sch. 40 PVC pipe with 25 mm drilled holes.

The distributor/collector pipes should span the cell width and have cleanout access.

Inlet section 1 m in length In

Outlet section 1 m in length Out

Cell Size	Width (m)	Length (m)
VSB dimension	12	26
Top of Berm	14	27
O/S top path	20	33

VSF Design Calculations at Summer Conditions

			<u>Symbol</u>
Step 1: Define Bed	Media	washed 1.25 - 2.5 cm gravel	
	Depth	0.8 m	d
	Primary Plant	Bulrush	
	Bed Temperature	13 deg. C	T _{sd}
	Design Flow	37.35294118 cu m/d	Q _d
Step 2: Calc. Area	Rate Constant	0.551 1/d	K _{Ts}
	where $K_{Ts} = f_{ss} \cdot K_{std} (f_{tc})^{(T_d - 20)}$		
	Surface Area	589 sq m	A _{sum}
	where $A_s = Q_d \cdot \ln(C_e/C_o) / (K_{Ts} \cdot d \cdot n_g)$		
Step 3: Aspect Ratio and Dimensions	Aspect Ratio	2 :1 L:W	R _a
	Width	17 m	W _s
	Length	34 m	L _s
Step 4: Calc. Max. Flow w/ Darcy's Law	Bed should be constructed with nearly flat base or sloped at 0.5% to outlet end.		
	Maximum hydraulic gradient will be equivalent to bed depth assuming the outlet structure is designed to permit complete draining. The method of Reed (1993) suggests that only 10% of available hydraulic gradient be used for design.		
	Hydr. Gradient	0.08 m	S = 0.1*d
	Max. Flow	549 cu m/d	Q _{maxd}
	where $Q_{maxd} = (k_s/3) \cdot (W_s \cdot d) \cdot S$		
	As a safety factor only 1/3 of hydraulic conductivity (k _s) is used (Reed, 1993).		
	Retention Time	4.8 days	HRT

Pretreatment System Specifications

Symbol

Assumed

Pretreatment:

Multi-compartment Septic Tank
Two tanks in series required for rates over 38 cu m/d.
Minimum volume = twice daily flow
Per US EPA procedures (Reed, 1993)

Required Septic

Tank Size:

Combined Vol.	70.4 cu m	Vst
	where $V_{st} = (2 \cdot Q_b) + Q_{ev}$	
Number of Tanks	2	Ntks
Tank Capacity	35 cu m (each of 2 tanks)	
Normal Vol.	60 cu m	
Surge Vol.	10.4 cu m	

Conceptual Septic

Tank Design:

Design factors obtained from Metcalf and Eddy (1991) to qualitatively describe septic tanks only. A qualified engineer should prepare final design.

Depth	1.8 m	dst
Head Space	25 cm	
L:W	3 :1	Rast
Compartments	3	
Vol. Allocation	33%, 33%, 33%	

Tank Width 2.6 m

Tank Length 7.7 m

Norm. Op. Level 1.5 m

Effluent control to restrict outflow to a maximum rate of:
37.4 cu m/d

Septic tank level control range must be from:
1.5 m to a max. of 1.8 m