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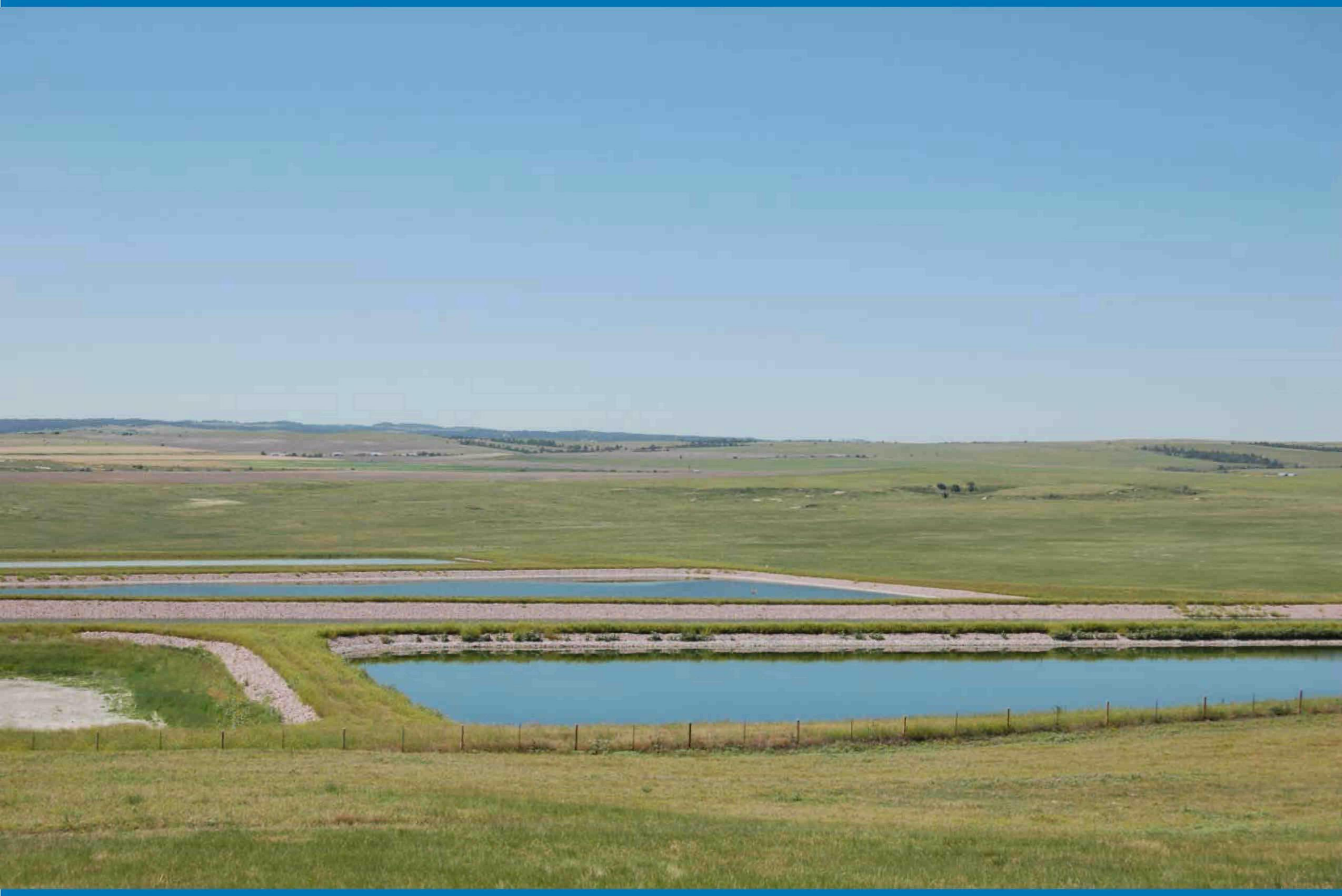
Principles of Design and Operations of Wastewater Treatment Pond Systems for Plant Operators, Engineers, and Managers



Office of Research and Development
National Risk Management Research Laboratory - Land Remediation and Pollution Control Division

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National Risk Management Research Laboratory
Office of Research and Development
U.S. Environmental Protection Agency
Cincinnati, Ohio

Notice

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Sally Gutierrez, Director
National Risk Management Research Laboratory

Abstract

Principles of Design and Operations of

Wastewater Treatment Pond Systems for Plant Operators, Engineers, and Managers

Wastewater pond systems provide reliable, low cost, and relatively low maintenance treatment for municipal and industrial discharges. However, they do have certain design, operations, and maintenance requirements. While the basic models have not changed in the 30-odd years since EPA published the last ponds manual, there have been some innovations and improved understanding of the complex biological processes at work in these systems. Additionally, new water quality requirements are either in place or about to be put in place throughout the United States, particularly relating to nutrient concentrations, that were not factored into the design specifications when many of the existing ponds were constructed. This updated version of the wastewater treatment ponds manual includes basic design recommendations, discusses the innovations in design that have been made in new, expanded or modified systems, as well as the additional processes that have been added to address nutrient requirements. An emphasis is placed on the importance of operations and maintenance, which is demonstrated in the troubleshooting section and appendices from several states, directed at providing training for operators.

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Table 1. Metric Conversion Factors (SI to U.S. Customary Units)

Multiply the SI unit		by	To obtain the U.S. unit	
Name	Symbol		Symbol	Name
Area				
hectare (10,000 m^2)	ha	2.4711	ac	acre
square centimeter	cm^2	0.1550	in^2	square inch
square kilometer	km^2	0.3861	mi^2	square mile
square kilometer	km^2	247.1054	ac	acre
square meter	m^2	10.7639	ft^2	square foot
square meter	m^2	1.1960	yd^2	square yard
Energy				
Kilojoule	kJ	0.9478	Btu	British thermal unit
joule	J	2.7778×10^{-7}	kWh	kilowatt-hour
megajoule	MJ	0.3725	hp · h	horsepower-hour
conductance, thermal	$W/m^2 \cdot {}^\circ C$	0.1761	$Btu/h \cdot ft^2 \cdot {}^\circ F$	conductance
conductivity, thermal	$W/m \cdot {}^\circ C$	0.5778	$Btu/h \cdot ft \cdot {}^\circ F$	conductivity
heat transfer coefficient	$W/m^2 \cdot {}^\circ C$	0.1761	$Btu/h \cdot ft^2 \cdot {}^\circ F$	heat transfer coefficient
latent heat of water	344,944 J/kg	—	144 Btu/lb	latent heat of water
specific heat, water	4215 J/kg · °C	—	1.007 Btu/lb · °F	specific heat of water
Flow rate				
cubic meters per day	m^3/d	264.1720	gal/d	gallons per day
cubic meters per day	m^3/d	2.6417×10^{-4}	MGD	million gallons per day
cubic meters per second	m^3/s	35.3157	ft^3/s	cubic feet per second
cubic meters per second	m^3/s	22.8245	MGD	million gallons per day
cubic meters per second	m^3/s	15.8503	gal/min	gallons per minute
liters per second	L/s	22.8245	gal/d	gallons per day
Length				
centimeter	cm	0.3937	in	inch
kilometer	km	0.6214	mi	mile
meter	m	39.3701	in	inch
meter	m	3.2808	ft	foot
meter	m	1.0936	yd	yard
millimeter	mm	0.03937	in	inch
Mass				
gram	g	0.0353	oz	ounce
gram	g	0.0022	lb	pound
kilogram	kg	2.2046	lb	pound
megagram (10^3 kg) (metric ton)	Mg (mt)	1.1023	ton (t)	ton (short: 2000 lb)
megagram	Mg	0.9842	ton	ton (long: 2240 lb)
Power				
kilowatt	kW	0.9478	Btu/s	British thermal units per second
kilowatt	kW	1.3410	hp	horsepower
Pressure				
pascal	Pa (N/m^2)	1.4505×10^{-4}	lb/in^2	pounds per square inch
Temperature				
degree Celsius	°C	1.8(°C) + 32	°F	degree Fahrenheit
kelvin	K	1.8(K) − 459.67	°F	degree Fahrenheit
Velocity				
kilometers per second	km/s	2.2369	mi/h	miles per hour
meters per second	m/s	3.2808	ft/s	feet per second
Volume				
cubic centimeter	cm^3	0.0610	in^3	cubic inch
cubic meter	m^3	35.3147	ft^3	cubic foot
cubic meter	m^3	1.3079	yd^3	cubic yard
cubic meter	m^3	264.1720	gal	gallon
cubic meter	m^3	8.1071×10^{-4}	ac-ft	acre foot
liter	L	0.2642	gal	gallon
liter	L	0.0353	ft^3	cubic foot
liter	L	33.8150	oz	ounce (U.S. fluid)
megaliter (L × 10^6)	ML	0.2642	MG	million gallons

Table 2. Conversion Factors for Commonly Used Design Parameters

Multiply the SI unit		by	To obtain the U.S. Customary unit	
Parameter	Symbol		Symbol	Parameter
cubic meters per second	m^3/s	22.727	mgd	million gallons per day
cubic meters per day	m^3/d	264.1720	gal/d	gallons per day
kilogram per hectare	kg/ha	0.8922	lb/ac	pounds per acre
metric ton per hectare	Mg/ha	0.4461	ton/ac	tons (short) per acre
cubic meter per hectare per day	$m^3/ha \cdot d$	106.9064	$gal/ac \cdot d$	gallons per acre per day
kilograms per square meter per day	$kg/m^2 \cdot d$	0.2048	$lb/ft^2 \cdot d$	pounds per square foot per day
cubic meter (solids) per 10^3 cubic meters (liquid)	$m^3/10^3\ m^3$	133.681	ft^3/MG	cubic feet per million gallons
cubic meters (liquid) per square meter (area)	m^3/m^2	24.5424	gal/ft^2	gallons per square foot
grams (solids) per cubic meter (liquid)	g/m^3	8.3454	lb/MG	pounds per million gallons
cubic meters (air) per cubic meter (liquid) per minute	$m^3/m^3 \cdot min$	1000.0	$ft^3/10^3 \cdot min$	cubic feet of air per minute per 1000 ft^3
kilowatts per 10^3 cubic meter (tank volume)	$kW/10^3\ m^3$	0.0380	$hp/10^3\ ft^3$	horsepower per 1000 ft^3
kilograms per cubic meter	kg/m^3	62.4280	$lb/10^3\ ft^3$	pounds per 1000 ft^3
cubic meter per capita	m^3/capita	35.3147	ft^3/capita	cubic feet per capita
bushels per hectare	bu/ha	0.4047	bu/ac	bushels per acre

Table 3. Physical Properties of Water

Temperature (°C)	Density (kg/m^3)	Dynamic viscosity $\times 10^3$ ($N \cdot s/m^2$)	Kinematic viscoity (γ) $\times 10^6$ (m^2/s)
0	999.8	1.781	1.785
5	1000.0	1.518	1.519
10	999.7	1.307	1.306
15	999.1	1.139	1.139
20	998.2	1.002	1.003
25	997.0	0.890	0.893
30	995.7	0.798	0.800
40	992.2	0.653	0.658
50	988.0	0.547	0.553
60	983.2	0.466	0.474
70	977.8	0.404	0.413
80	971.8	0.354	0.364
90	965.3	0.315	0.326
100	958.4	0.282	0.294

Table 4. Dissolved Oxygen Solubility in Fresh Water*

Temperature (°C)	Dissolved oxygen solubility (mg/L)
0	14.62
1	14.23
2	13.84
3	13.48
4	13.13
5	12.80
6	12.48
7	12.17
8	11.87
9	11.59
10	11.33
11	11.08
12	10.83
13	10.60
14	10.37
15	10.15
16	9.95
17	9.74
18	9.54
19	9.35
20	9.17
21	8.99
22	8.83
23	8.68
24	8.53
25	8.38
26	8.22
27	8.07
28	7.92
29	7.77
30	7.63

*Saturation values of dissolved oxygen when exposed to dry air containing 20.90% oxygen under a total pressure of 760 mmHg.

Glossary

Abate®	larvicide
AFP	advanced facultative pond
AIWPS	Advanced Integrated Wastewater Pond System®
AMTA	Activated Membrane Technology Associations
AMTS	Advanced Microbial Treatment System
ASP	algae settling pond
ATLAS-IS	Advanced Technology Lagoon Aeration System with Internal Separator
AWT	activated waste treatment
BIOLAC® Process	activated sludge in earthen ponds
BOD	biochemical oxygen demand
BOD_5	5 day biochemical oxygen demand
Bti	Bacillus *thuringiensis israelensis* (larvicide)
CAPM	Centre for Aquatic Plant Management
CBOD	carbonaceous biochemical oxygen demand
CFID	continuous feed, intermittent discharge
CLEAR™	Cyclical Lagoon Extended Aeration Reactor
COD	chemical oxygen demand
CWT	Centralized Waste Treatment
DAF	dissolved air flotation
DFR	design flow rate
DMR	discharge monitoring reports
DO	dissolved oxygen
DPMC	Dual-power, multi-cellular system

DPMC-IS	Dual-power, multi-cellular intermittent sand filter system
e.s.	effective size
EDI	Environmental Dynamics, Inc.
ENRCCI	Engineering News Record Construction Cost Index
FC	fecal coliform(s)
F/M	food/microorganism
FWS	free water surface
GIS	Geographic Information System
gpm	gallons per minute
HCR	hydrogen controlled release
HLT	high level transfer
HPAPS	high-performance aerated pond system
HRP	high rate pond
HRT	hydraulic residence time/hydraulic retention time
IPDs	in-pond digesters
I&I	Inflow & Infiltration
JTU	Jackson Turbidity Unit
Lagoon-ISF	Lagoon Intermittent Sand filter
M	metal ion
MBBR™	Moving Bed Biofilm Reactor
MCRT	mean cell resident time
mgd	million gallons per day
MLSS	mixed liquor suspended solids
MLVSS	mixed liquor volatile suspended solids
MPN	most probable number
MSL	mean sea level

NEIWPCC	New England Interstate Water Pollution Control Commission
NFB	nitrification filter bed
NH_4 - N	ammonia-N, ammonia nitrogen
NOAA	National Oceanic and Atmospheric Administration
NPDES	National Pollutant Discharge Elimination System
NRCS	National Resources Conservation Services
OD	oxygen demand
O&M	operation and maintenance
%	percent
POTWs	publicly owned treatment works
RAS	return activated sludge
RO	reverse osmosis
SAR	sodium adsorption ratio
SBR	sequencing batch reactor
$SCBOD_5$	soluble carbonaceous BOD_5
scfm	standard cubic feet per minute
sf	square foot
SF, SSF	subsurface flow
SFP	secondary facultative pond
SS	suspended solids
STEP	septic tank effluent pumping system
TFCC	total fecal coliform count
TKN	total Kjeldahl nitrogen
TP	total phosphorus
TSS	total suspended solids
TVSS	total volatile suspended solids

U.C., u.c.	uniformity coefficient
UF	ultrafiltration
USGS	U. S. Geological Society
VSS	volatile suspended solids
WAS	waste activated sludge
WHO	World Health Organization
WTCost	a CD-Rom for estimating plant membrane treatment costs

Acknowledgments

This document, an up-to-date revision of the Municipal Wastewater Stabilization Ponds design manual published by USEPA in 1983, is the result of the interest and commitment of many contributors, who are listed below in alphabetical order.

Editors*
Eugenia McNaughton, USEPA, Region 9, San Francisco, CA
James E. Smith, Jr., retired, formerly with USEPA, NRMRL, Cincinnati, OH
Sally Stoll, USEPA, NRMRL, Cincinnati, OH

Contracted Authors*
Richard H. Bowman, R.H. Bowman and Associates
E. Joe Middlebrooks, Consultant

Contributing Writer
Laurel J. Staley, USEPA, NRMRL, Cincinnati, OH

Special Contributions, Technical Assistance and Review
Robert K. Bastian, USEPA, OWM, Washington, DC
Robert B. Brobst, USEPA, Region 8, Denver, CO
David Chin, USEPA, Region 1, Boston, MA
Ronald W. Crites, consultant
Wayne Daugherty, City of St. Helena CA Public Works
Steve Duerre, MN Pollution Control Agency
Gene Erickson, MN Pollution Control Agency
F. Bailey Green, consultant
Geoffrey Holmes, Fisherman Bay Sewer District, Lopez Island, WA
Peter Husby, USEPA, Region 9 Laboratory, Richmond, CA
Craig Johnson, University of Utah
Thomas Konner, USEPA, Region 9
Russell Martin, USEPA, Region 5, Chicago, IL
Larry Parlin, City of Stockton CA Public Works
Charles Pycha, USEPA, Region 5
Wes Ripple, NH Department of Environmental Services
Robert Rubin, North Carolina State University
Michael Sample, City of St. Helena CA Public Works
Amy Wagner, USEPA, Region 9 Laboratory
Jianpeng Zhou, Southern Illinois University at Edwardsville

Technical Reviewers
Don Albert, retired, formerly with ME Department of Environmental Protection
Edwin Barth, USEPA, NRMRL
Charles Corley, IL Environmental Protection Agency
John Hamilton, Indian Health Service, USEPA Region 9

Paul Krauth, UT Department of Environmental Quality
Tryg Lundquist, California State University at San Luis Obispo
Paul Olander, retired, formerly with VT Department of Environmental Conservation
Syed Shahriyar, USEPA, Region 6, Dallas, TX

Editing and Production:
Jan Byers, contractor USEPA, Region 9
Jean Dye, USEPA, ORD, Cincinnati, OH
Katherine Loizos, contractor, USEPA
Patricia Louis, USEPA, ORD, Cincinnati, OH

The authors wish to dedicate this manual to the memory of Dr. William J. Oswald, whose vision of using design principles respecting and modeled after natural processes has come to be understood as basic to our survival as a species: reducing energy consumption, reimagining "waste products" as resources, and building sustainable projects, in order to solve some of civilization's most complicated and persistent problems. Dr. Oswald continues to be an inspiration to generations of his students, in and out of universities, and throughout the world.

*In 2000, USEPA Office of Wastewater Management (OWM) underwrote a needs assessment to determine whether a revised and updated edition of the 1983 Wastewater Stabilization Ponds Design Manual was needed. The answer was affirmative and OWM, working with ORD NRMRL, Cincinnati, hired a consultant, E. Joe Middlebrooks to conduct the work. Several of the Regions contributed funding to complete the project: Regions 5, 8 and 9 applied for Regional Applied Research Effort (RARE) funds; Region 6 contributed funds from its Tribal program; Region 1 funds were from RARE as well as general funding. Gajindar Singh, Office of Water has been a tireless supporter of pond technology. The final product represents the work of the consultant and his subcontractor and many USEPA staff, who share the belief that the benefits of wastewater pond technology should be more widely known and accepted among the community of design engineers, city and community managers, and that information about them should be more readily available, especially to the plant operators, who work with them every day.

Cover Picture is of a modified AIWPS® treating wastewater from Pine Ridge, South Dakota on the Pine Ridge Reservation, home the Oglala Sioux (Lakota) Tribe. Startup was in 2009 treating wastewater from the 5,500 residents of the village. The Treatment works consist of AIWPS® primary Pond with fermentation pits, followed by secondary cells, followed by a wetland for final treatment prior to discharge. The design was based on the Tribe's desire for something other than a conventional lagoon system and the need to keep operation and maintenance to a minimum. The Treatment area is surrounded by pasture lands and can be expected to stay remote for the foreseeable future. Each of the primary ponds will contain two Oswald type fermentation pits that are conservatively designed. The outer pond is oversized to compensate for lack of aerators. The secondary cells have been sized for holding during the winter with a total detention of 150 days. The wetland design is based on 25,000 gpd/ac which was recommended by the State of South Dakota. The facility will discharge 725,000 gallons per day at design capacity. For additional information contact Anthony Kathol, P.E. at Anthony.Kathol@ihs.gov.

Preface

Eugenia McNaughton (U.S. EPA Region 9), James E. Smith, Jr. (U.S. EPA ORD NRMRL, retired), Sally Stoll (ORD NRMRL)

January 21, 2011

Stabilization ponds have been used for treatment of wastewater for over 3,000 years. The first recorded construction of a pond system in the U.S. was at San Antonio, Texas, in 1901. Today, over 8,000 wastewater treatment ponds are in place, involving more than 50% of the wastewater treatment facilities in the U.S. (CWNS, 2000). Facultative ponds account for 62%, aerated ponds 25%, anaerobic 0.04% and total containment 12% of the pond treatment systems. They treat a variety of wastewaters from domestic wastewater to complex industrial wastes, and they function under a wide range of weather conditions, from tropical to arctic. Ponds can be used alone or in combination with other wastewater treatment processes. As our understanding of pond operating mechanisms has increased, different types of ponds have been developed for application in specific types of wastewater under local environmental conditions. This manual focuses on municipal wastewater treatment pond systems.

We should note here that we will use the word "treatment" in place of "stabilization," which has come to have a much more specific meaning since the first manual was published. We will also refer to "ponds" versus "lagoons," for consistency in the manual, though we recognize that in this case either term is acceptable.

The U.S. Environmental Protection Agency (EPA) last published a Wastewater Stabilization Ponds Design Manual in 1983 under the Technology Transfer Program, which was developed "to describe technological advances and present new information." EPA support for pond systems as options for municipal wastewater treatment was most welcome, particularly for small communities that could not afford to match even the generous construction grants that were offered at that time to bring communities of all sizes some level of wastewater treatment.

While the tendency in the U.S. has been for smaller communities to build ponds, in other parts of the world, including Australia, New Zealand, Mexico and Latin America, Asia and Africa, treatment ponds have been built for large cities. As a result, our understanding of the biological, biochemical, physical and climatic factors that interact to transform the organic compounds, nutrients and pathogenic organisms found in sewage into less harmful chemicals and unviable organisms (i.e., dead or sterile) has grown since 1983. A wealth of experience has been built up as civil, sanitary or environmental engineers, operators, public works managers and public health and environmental agencies have gained more experience with these systems. While some of this information makes its way into technical journals and text books, there is a need for a less formal presentation of the subject for those working in the field every day.

In gathering the information for this revision, we interviewed state regulators, local operators, engineers, consultants, and academics. We read as much of the literature as we could find, always searching for case histories illustrating new performance achievements and associated design details that might be employed in other systems. We found that there has been some evolution of design, such as in the AIWPS™, but many improvements have included, for example, the addition of more aerators, moving the systems closer to activated sludge with the attendant high energy and sludge removal costs. Much recent work has focused on pond hydraulics and we understand now that for consistent performance, the design and placement of inlet and outlet structures to avoid short circuiting and loss of solids is critical, and that redundancy must be built into the system to allow for flexibility in operation and maintenance. Some additions have been necessary to meet nutrient requirements that were not in place when the systems were built. Overall, however, pond systems still offer an alternative that is lower in capital outlay, operations, and maintenance costs. Appropriately designed ponds are capable of meeting strict environmental standards with minimal biosolids management requirements and reasonable energy costs.

Looking to the future, what has been the most problematic element for stabilization ponds, the growth and persistence of algae throughout the system, is lately coming to be seen as a potential asset. It may soon be time to talk about enhancing the growth of algae for use as biofuel or livestock food supplements to replace irrigated feed crops and conventional energy sources. Opportunities to install solar power collectors, either to supply the entire system's energy needs or to run aerators, may make an already low energy use system effectively carbon neutral or net-energy positive. The cost to add elements that treat wastewater to reduce nutrient discharge would be less challenging for a community that has a system that is already a low energy consumer.

In the spirit of the times, we acknowledge that another book isn't necessarily the way to get information out these days. This version of the manual is instead a compendium organized around the topics related to design, operation, and maintenance of wastewater stabilization ponds that must meet ever more stringent discharge requirements. It will be available on the web in its entirety chapter by chapter. It is our hope that this will be a resource to which you will return many times over the course of your involvement with wastewater ponds. And we look forward to hearing from you about improvements to existing text or other information that we might include to make the manual an evolving and dynamic document, attesting to the importance of this wastewater treatment process and to the continued enthusiasm for it that inspired us to make this effort to bring it to you.

CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

1.1.1 History

Treatment ponds have been employed for treatment of wastewater for over 3,000 years. The first recorded construction of a pond system in the U.S. was in San Antonio, Texas, in 1901 (Gloyna, 1971). Today, over 8,000 wastewater treatment ponds, comprising more than 50 percent of the wastewater treatment facilities in the United States, are in place (Bastian, pers. comm., 2010). Ponds are used to treat wastewater generated by small communities in Europe. Larger pond systems are in place in New Zealand, Australia and Africa (Mara, 2003). They are used to treat a variety of wastewaters, from domestic to complex industrial effluent, and they function under a wide range of climatic conditions, from tropical to arctic. Ponds can be used alone or in combination with other wastewater treatment processes. As understanding of pond operating mechanisms has improved, different types of ponds have been developed to meet specific conditions. Ponds generally require less energy than other treatment systems and have lower operation and maintenance costs.

1.1.2 Trends

The basic elements of pond system design have remained unchanged in the 25 years since the publication of the EPA manual (Design Manual: Municipal Wastewater Stabilization Ponds, EPA-625/1-83-015, 1983a). Aspects of the basic pond designs have evolved and several modifications have been developed. These have been in response to increasingly stringent water quality regulatory requirements for point source discharges.

The major procedures, processes and design methods relevant to wastewater treatment ponds that will be discussed in this manual are:

Basic Processes (flow through basins)

- Anaerobic
- Facultative
- Aerobic

In-Pond Design Evolution and Enhancements

- AIWPS® (Oswald)
- Deep Fermentation Pits
- High Performance Shallow Ponds

Oxygen Addition

- LAS International, Ltd.
- PRAXAIR, Inc.

Modifications that Require Energy

- Partial Mix
- Complete Mix

- High-Performance Aerated Ponds (Rich)
- BIOLAC™

Nutrient Removal
- Nitrogen
    - In pond
    - Modified high performance aerated systems for nitrification/denitrification
    - In pond with wetlands and gravel bed filters
- Phosphorus

Effluent TSS (Algae) Removal
- Lemna
- Algae settling basins
- Barley straw

1.1.3 Manual Objective and Scope
This manual provides an overview of wastewater treatment pond systems through the discussion of factors affecting treatment, process design principles and applications, aspects of physical design and construction, effluent total suspended solids (TSS), algae, nutrient removal alternatives, and cost and energy requirements. In this chapter, the biological, physical and chemical processes that occur in wastewater treatment ponds are discussed.

Chapter 2 describes a sequential approach to the development of a wastewater management project. This approach determines feasibility of the process itself and the land area required for treatment, and identifies possible sites. These sites are evaluated based on technical and cost-effective alternatives.

Chapter 3 includes design for the basic types of treatment ponds.

Chapter 4 discusses the physical design and construction criteria that define effective pond performance, regardless of the design equation employed, and must be considered in the facility design process.

Chapter 5 describes the evolution and enhancement of the basic designs within ponds over the last 30 years.

Chapter 6 presents a discussion of the capability of conventional facultative and aerated lagoons to reduce nutrient concentrations, including commercial products for nitrogen (N) and phosphorus (P) removal.

Chapter 7 presents alternatives for control and removal of algae-derived TSS.

Chapter 8 covers cost and energy requirements.

Chapter 9 includes information on the operation, maintenance and troubleshooting of treatment ponds.

Appendix A lists the state criteria for wastewater treatment ponds. A summary of pond design methods is presented in Appendix B. Design models and examples are presented in Appendix C. Case studies are found in Appendix D. Appendix E is a troubleshooting guide; Appendix F contains study guides for operators from the state of Wisconsin; discharge guidance from the state of Minnesota is in Appendix G. Appendix H presents guidance for the use of barley straw to reduce algal TSS from the state of Illinois. Appendix I contains the glossary, and Appendix J contains a conversion table and other general information.

1.2 POND NOMENCLATURE

Ponds are designed to enhance the growth of natural ecosystems that are either anaerobic (providing conditions for bacteria that grow in the absence of oxygen *[O_2]* environments), aerobic (promoting the growth of O_2 producing and/or requiring organisms, such as algae and bacteria), or facultative, which is a combination of the two. Ponds are managed to reduce concentrations of biochemical oxygen demand (BOD), TSS and coliform numbers (fecal or total) to meet water quality requirements. Table 1-1 summarizes information on pond application, loading, and size of wastewater treatment ponds.

Table 1.1. Basic Wastewater Pond Specifications (adapted from Curi and Eckenfelder 1980).

Pond	Application	Typical Loading (BOD_5)*	Typical Detention Time (d)	Typical Depth (m)	Comments
Anaerobic	Industrial wastewater	280-4500 kg / 1000 m^2/d	5-50	2.5-4.5	Subsequent treatment normally required.
Facultative	Raw municipal wastewater. Effluent from primary treatment, trickling filters, aerated ponds, or anaerobic ponds.	22-56 kg/ 1000m^2/d	7-50	0.9-2.4	Most commonly used wastewater treatment pond. May be aerobic through entire depth if lightly loaded.
Aerobic	Generally used to treat effluent from other processes. Produces effluent low in soluble BOD_5 and high in algal solids.	112-225 kg/ 1000 m^2 /d	2-6	0.18-0.3	Maximizes algae production and, if algae are harvested, nutrient removal.

*BOD_5 = Biochemical Oxygen Demand measured over 5 days

1.2.1 Anaerobic Ponds

Anaerobic ponds receive such a heavy organic loading that there is no aerobic zone. They are usually 2.5 – 4.5 m in depth and have detention times of 5 - 50 days. The predominant biological treatment reactions are bacterial acid formation and methane fermentation.

Anaerobic ponds are usually used for treatment of strong industrial and agricultural (food processing) wastes, as a pretreatment step in municipal systems, or where an industry is a significant contributor to a municipal system. The biochemical reactions in an anaerobic pond produce hydrogen sulfide (H_2S) and other odorous compounds. To reduce odors, the common practice is to recirculate water from a downstream facultative or aerated pond. This provides a thin aerobic layer at the surface of the anaerobic pond, which prevents odors from escaping into the air. A cover may also be used to contain odors. The effluent from anaerobic ponds usually requires further treatment prior to discharge.

1.2.2 Facultative Ponds

The most common type of pond is the facultative pond, which may also be called an oxidation or photosynthetic pond. Facultative ponds are usually 0.9 - 2.4 m deep or deeper, with an aerobic layer overlying an anaerobic layer. Recommended detention times vary from 5 - 50 days in warm climates and 90 - 180 days in colder climates (New England Interstate Water Pollution Control Commission [NEIWPCC], 1998, heretofore referred to as TR-16). Aerobic treatment processes in the upper layer provide odor control, nutrient and BOD removal. Anaerobic fermentation processes, such as sludge digestion, denitrification and some BOD removal, occur in the lower layer. The key to successful operation of this type of pond is O_2 production by photosynthetic algae and/or re-aeration at the surface.

Facultative ponds are used to treat raw municipal wastewater in small communities and for primary or secondary effluent treatment for small or large cities. They are also used in industrial applications, usually in the process line after aerated or anaerobic ponds, to provide additional treatment prior to discharge. Commonly achieved effluent BOD values, as measured in the BOD_5 test, range from 20 - 60 mg/L, and TSS levels may range from 30 - 150 mg/L. The size of the pond needed to treat BOD loadings depends on specific conditions and regulatory requirements.

Facultative ponds overloaded due to unplanned additional sewage volume or higher strength influent from a new industrial connection may be modified by the addition of mechanical aeration. Ponds originally designed for mechanical aeration are generally 2 - 6 m deep with detention times of 3 - 10 days. For colder climates, TR-16 suggests 20 - 40 days. Mechanically aerated ponds require less land area but have greater energy requirements.

1.2.3 Aerobic Ponds

Aerobic ponds, also known as oxidation ponds or high-rate aerobic ponds, maintain dissolved oxygen (DO) throughout their entire depth. They are usually 30 - 45 cm deep, which allows light to penetrate throughout the pond. Mixing is often provided, keeping algae at the surface to maintain maximum rates of photosynthesis and O_2 production and to prevent algae from settling and producing an anaerobic bottom layer. The rate of photosynthetic production of O_2 may be enhanced by surface re-aeration; O_2 and aerobic bacteria biochemically stabilize the waste. Detention time is typically two to six days.

These ponds are appropriate for treatment in warm, sunny climates. They are used where a high degree of BOD_5 removal is desired but land area is limited. The chief advantage of these ponds is that they produce a stable effluent during short detention times with low land and energy requirements. However, their operation is somewhat more complex than that of facultative ponds and, unless the algae are removed, the effluent will contain high TSS. While the shallow depths allow penetration of ultra-violet (UV) light that may reduce pathogens, shorter detention times may work against effective coliform and parasite die-off. Since they are shallow, bottom paving or covering is usually necessary to prevent aquatic plants from colonizing the ponds. The Advanced Integrated Wastewater Pond System® (AIWPS®) uses the high-rate pond to maximize the growth of microalgae using a low-energy paddle-wheel. This use of the high-rate pond will be discussed in Chapter 5.

1.3 ELEMENTS OF POND PROCESSES

1.3.1 The Organisms

Although our understanding of wastewater pond ecology is far from complete, general observations about the interactions of macro- and microorganisms in these biologically driven systems support our ability to design, operate and maintain them.

1.3.1.1 Bacteria

In this section, we discuss other types of bacteria found in the pond; these organisms help to decompose complex, organic constituents in the influent to simple, non-toxic compounds. Certain pathogenic bacteria and other microbial organisms (viruses, protozoa) associated with human waste enter into the system with the influent; the wastewater treatment process is designed so that their numbers will be reduced adequately to meet public health standards. Their fate in wastewater ponds will be discussed in Chapters 5 and 9.

1.3.1.1.1 Aerobic Bacteria

Bacteria found in the aerobic zone of a wastewater pond are primarily the same type as those found in an activated sludge process or in the zoogleal mass of a trickling filter. The most frequently isolated bacteria include *Beggiatoa alba, Sphaerotilus natans, Achromobacter, Alcaligenes, Flavobacterium, Pseudomonas and Zoogoea spp.* (Lynch and Poole, 1979; Pearson, 2005). These organisms decompose the organic materials present in the aerobic zone into oxidized end products.

1.3.1.1.2 Anaerobic Bacteria

Hydrolytic bacteria convert complex organic material into simple alcohols and acids, primarily amino acids, glucose, fatty acid and glycerols (Brockett, 1976; Pearson, 2005; Paterson and Curtis, 2005). Acidogenic bacteria convert the sugars and amino acids into propionic, acetic and butyric acids. Acetogenic bacteria convert these organic acids into acetate, ammonia (NH_3), hydrogen (H), and carbon dioxide (CO_2). Methanogenic bacteria break down these products further to methane (CH_4) and CO_2 (Gallert and Winter, 2005).

1.3.1.1.3 Cyanobacteria

Cyanobacteria, formerly classified as blue-green algae, are autotrophic organisms that are able to synthesize organic compounds using CO_2as the major carbon source. Cyanobacteria produce O_2 as a by-product of photosynthesis, providing an O_2 source for other organisms in the ponds. They are found in very large numbers as blooms when environmental conditions are suitable (Gaudy and Gaudy, 1980). Commonly encountered cyanobacteria include *Oscillatoria, Arthrospira, Spirulina, and Microcystis* (Vasconcelos and Pereira, 2001).

1.3.1.1.4 Purple Sulfur Bacteria

Purple sulfur bacteria (Chromatiaceae) may grow in any aquatic environment to which light of the required wavelength penetrates, provided that CO_2, nitrogen (N), and a reduced form of sulfur (S) or H are available. Purple sulfur bacteria occupy the anaerobic layer below the algae, cyanobacteria, and other aerobic bacteria in a pond. They are commonly found at a specific depth, in a thin layer where light and nutrient conditions are at an optimum (Gaudy and Gaudy, 1980; Pearson, 2005). Their biochemical conversion of odorous sulfide compounds to elemental S or sulfate (SO_4) helps to control odor in facultative and anaerobic ponds.

1.3.2 Algae

Algae constitute a group of aquatic organisms that may be unicellular or multicellular, motile or immotile, and, depending on the phylogenetic family, have different combinations of photo-synthetic pigments. As autotrophs, algae need only inorganic nutrients, such as N, phosphorus (P) and a suite of microelements, to fix $CO_2$and grow in the presence of sunlight. Algae do not fix atmospheric N; they require an external source of inorganic N in the form of nitrate (NO_3) or NH_3. Some algal species are able to use amino acids and other organic N compounds. Oxygen is a by-product of these reactions.

Algae are generally divided into three major groups, based on the color reflected from the cells by the chlorophyll and other pigments involved in photosynthesis. Green and brown algae are common to wastewater ponds; red algae occur infrequently. The algal species that is dominant at any particular time is thought to be primarily a function of temperature, although the effects of predation, nutrient availability, and toxins are also important.

Green algae (Chlorophyta) include unicellular, filamentous, and colonial forms. Some green algal genera commonly found in facultative and aerobic ponds are *Euglena, Phacus, Chlamydomonas, Ankistrodesmus, Chlorella, Micractinium, Scenedesmus, Selenastrum, Dictyosphaerium* and *Volvox*.

Chrysophytes, or brown algae, are unicellular and may be flagellated, and include the diatoms. Certain brown algae are responsible for toxic red blooms. Brown algae found in wastewater ponds include the diatoms *Navicula* and *Cyclotella*.

Red algae (Rhodophyta) include a few unicellular forms, but are primarily filamentous (Gaudy and Gaudy, 1980; Pearson, 2005).

1.3.2.1 Importance of Interactions between Bacteria and Algae

It is generally accepted that the presence of both algae and bacteria is essential for the proper functioning of a treatment pond. Bacteria break down the complex organic waste components found in anaerobic and aerobic pond environments into simple compounds, which are then available for uptake by the algae. Algae, in turn, produce the O_2 necessary for the survival of aerobic bacteria.

In the process of pond reactions of biodegradation and mineralization of waste material by bacteria and the synthesis of new organic compounds in the form of algal cells, a pond effluent might contain a higher than acceptable TSS. Although this form of TSS does not contain the same constituents as the influent TSS, it does contribute to turbidity and needs to be removed before the effluent is discharged. Once concentrated and removed, depending on regulatory requirements, algal TSS may be used as a nutrient for use in agriculture or as a feed supplement (Grölund, 2002).

1.3.3 Invertebrates

Although bacteria and algae are the primary organisms through which waste stabilization is accomplished, predator life forms do play a role in wastewater pond ecology. It has been suggested that the planktonic invertebrate *Cladocera* spp. and the benthic invertebrate family Chironomidae are the most significant fauna in the pond community in terms of stabilizing organic matter. The cladocerans feed on the algae and promote flocculation and settling of particulate matter. This in turn results in better light penetration and algal growth at greater depths. Settled matter is further broken down and stabilized by the benthic feeding Chironomidae. Predators, such as rotifers, often control the population levels of certain of the smaller life forms in the pond, thereby influencing the succession of species throughout the seasons.

Mosquitoes can present a problem in some ponds. Aside from their nuisance characteristics, certain mosquitoes are also vectors for such diseases as encephalitis, malaria, and yellow fever, and constitute a hazard to public health which must be controlled. *Gambusia*, commonly called mosquito fish, have been introduced to eliminate mosquito problems in some ponds in warm climates (Ullrich, 1967; Pipes, 1961; Pearson, 2005), but their introduction has been problematic as they can out-compete native fish that also feed on mosquito larvae. There are also biochemical controls, such as the larvicides *Bacillus thuringiensis* israelensis (Bti), and Abate®, which may be effective if the product is applied directly to the area containing mosquito larvae. The most effective means of control of mosquitoes in ponds is the control of emergent vegetation.

1.3.4 Biochemistry in a Pond

1.3.4.1 Photosynthesis

Photosynthesis is the process whereby organisms use solar energy to fix CO_2 and obtain the reducing power to convert it to organic compounds. In wastewater ponds, the dominant photosynthetic organisms include algae, cyanobacteria, and purple sulfur bacteria (Pipes, 1961; Pearson, 2005).

Photosynthesis may be classified as oxygenic or anoxygenic, depending on the source of reducing power used by a particular organism. In oxygenic photosynthesis, water serves as the source of reducing power, with O_2 as a by-product. The equation representing oxygenic photosynthesis is:

$$H_2O + sunlight \longrightarrow 1/2O_2 + 2H^+ + 2e^- \qquad (1\text{-}1)$$

Oxygenic photosynthetic algae and cyanobacteria convert CO_2 to organic compounds, which serve as the major source of chemical energy for other aerobic organisms. Aerobic bacteria need the O_2 produced to function in their role as primary consumers in degrading complex organic waste material.

Anoxygenic photosynthesis does not produce O_2 and, in fact, occurs in the complete absence of O_2. The bacteria involved in anoxygenic photosynthesis are largely strict anaerobes, unable to function in the presence of O_2. They obtain energy by reducing inorganic compounds. Many photosynthetic bacteria utilize reduced S compounds or elemental S in anoxygenic photosynthesis according to the following equation:

$$H_2S \longrightarrow S^o + 2H^+ + 2e^- \qquad (1\text{-}2)$$

1.3.4.2 Respiration

Respiration is a physiological process by which organic compounds are oxidized into CO_2 and water. Respiration is also an indicator of cell material synthesis. It is a complex process that consists of many interrelated biochemical reactions (Stanier et al., 1963; Pearson, 2005). Aerobic respiration, common to species of bacteria, algae, protozoa, invertebrates and higher plants and animals, may be represented by the following equation:

$$C_2H_{12}O_6 + 6O_2 + enzymes \rightarrow 6CO_2 + 6H_2O + new\ cells \qquad (1\text{-}3)$$

The bacteria involved in aerobic respiration are primarily responsible for degradation of waste products.

In the presence of light, respiration and photosynthesis can occur simultaneously in algae. However, the respiration rate is low compared to the photosynthesis rate, which results in a net consumption of CO_2 and production of O_2. In the absence of light, on the other hand, algal respiration continues while photosynthesis stops, resulting in a net consumption of O_2 and production of CO_2.

1.3.4.3 Nitrogen Cycle

The N cycle occurring in a wastewater treatment pond consists of a number of biochemical reactions mediated by bacteria. A schematic representation of the changes in N speciation in wastewater ponds over a year is represented by Figure 1-1. See Chapter 6 for a more detailed discussion of the cycling of N species in ponds.

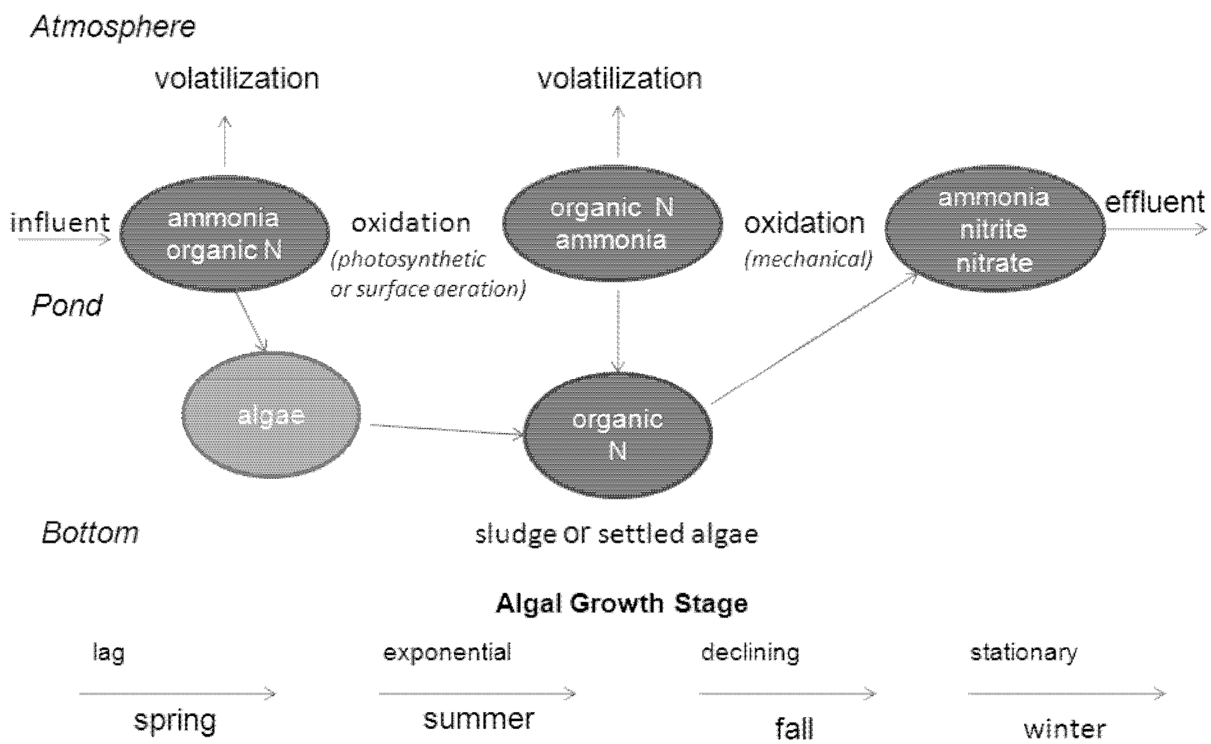


Figure 1-1. The nitrogen cycle in wastewater pond system.

Organic N and NH_3 enter with the influent wastewater. Organic N in fecal matter and other organic materials undergo conversion to NH_3 and ammonium ion NH_4^+ by microbial activity. The NH_3 may volatilize into the atmosphere. The rate of gaseous NH_3 losses to the atmosphere is primarily a function of pH, surface to volume ratio, temperature, and the mixing conditions. An alkaline pH shifts the equilibrium of NH_3 gas and NH_4^+ towards gaseous NH_3 production, while the mixing conditions affect the magnitude of the mass transfer coefficient.

Ammonium is nitrified to nitrite ($NO_2^=$) by the bacterium *Nitrosomonas* and then to NO_3^- by *Nitrobacter*. The overall nitrification reaction is:

$$NH_4^+ + 2O_2 \longrightarrow NO_3^- + 2H^+ + H_2O \qquad (1\text{-}4)$$

The NO_3^- produced in the nitrification process, as well as a portion of the NH_4^+ produced from ammonification, can be assimilated by organisms to produce cell protein and other N-containing compounds. The NO_3^- may also be denitrified to form $NO_2^=$ and then N gas. Several species of bacteria may be involved in the denitrification process, including *Pseudomonas, Micrococcus, Achromobacter,* and *Bacillus.* The overall denitrification reaction is

$$6NO_3^- + 5CH_3OH \rightarrow 3N_2 + 5CO_2 + 7H_2O + 6OH^- \qquad (1\text{-}5)$$

Nitrogen gas may be fixed by certain species of cyanobacteria when N is limited. This may occur in N-poor industrial ponds, but rarely in municipal or agricultural ponds (U.S. EPA, 1975a, 1993).

Nitrogen removal in facultative wastewater ponds can occur through any of the following processes: (1) gaseous NH_3 stripping to the atmosphere, (2) NH_4^+ assimilation in algal biomass, (3) NO_3^- uptake by floating vascular plants and algae, and (4) biological nitrification-denitrification. The removal of N is discussed in detail in Chapter 6. Whether NH_4^+ is assimilated into algal biomass depends on the biological activity in the system and is affected by several factors such as temperature, organic load, detention time, and wastewater characteristics.

1.3.4.4 Dissolved Oxygen (DO)

Oxygen is a partially soluble gas. Its solubility varies in direct proportion to the atmospheric pressure at any given temperature. DO concentrations of approximately 8 mg/L are generally considered to be the maximum available under local ambient conditions. In mechanically aerated ponds, the limited solubility of O_2 determines its absorption rate (Sawyer et al., 1994).

The natural sources of DO in ponds are photosynthetic oxygenation and surface re-aeration. In areas of low wind activity, surface re-aeration may be relatively unimportant, depending on the water depth. Where surface turbulence is created by excessive wind activity, surface re-aeration can be significant. Experiments have shown that DO in wastewater ponds varies almost directly with the level of photosynthetic activity, which is low at night and early morning and rises during daylight hours to a peak in the early afternoon. At increased depth, the effects of photosynthetic oxygenation and surface re-aeration decrease, as the distance from the water-atmosphere interface increases and light penetration decreases. This can result in the establishment of a vertical gradient. The microorganisms in the pond will segregate along the gradient.

1.3.4.5 *p*H and Alkalinity

In wastewater ponds, the H ion concentration, expressed as *p*H, is controlled through the carbonate buffering system represented by the following equations:

where:

$$CO_2 + H_2O \leftrightarrow H_2CO_3 \leftrightarrow HCO_3^- + H^+ \quad (1\text{-}6)$$

$$HCO_3^- \leftrightarrow CO_3^{-2} + H^+ \quad (1\text{-}7)$$

$$CO_3^{-2} + H_2O \leftrightarrow HCO_3^- + OH^- \quad (1\text{-}8)$$

$$OH^- + H^+ \leftrightarrow H_2O \quad (1\text{-}9)$$

The equilibrium of this system is affected by the rate of algal photosynthesis. In photosynthetic metabolism, CO_2 is removed from the dissolved phase, forcing the equilibrium of the first expression (1-6) to the left. This tends to decrease the hydrogen ion (H^+) concentration and the bicarbonate (HCO_3^-) alkalinity. The effect of the decrease in HCO_3^- concentration is to force the third equation (1-8) to the left and the fourth (1-9) to the right, both of which decrease total alkalinity. Figure 1-2 shows a typical relationship between *p*H, CO_2, HCO_3^{--}, CO_3^{-2}, and OH^-.

The decreased alkalinity associated with photosynthesis will simultaneously reduce the carbonate hardness present in the waste. Because of the close correlation between *p*H and photosynthetic activity, there is a diurnal fluctuation in *p*H when respiration is the dominant metabolic activity.

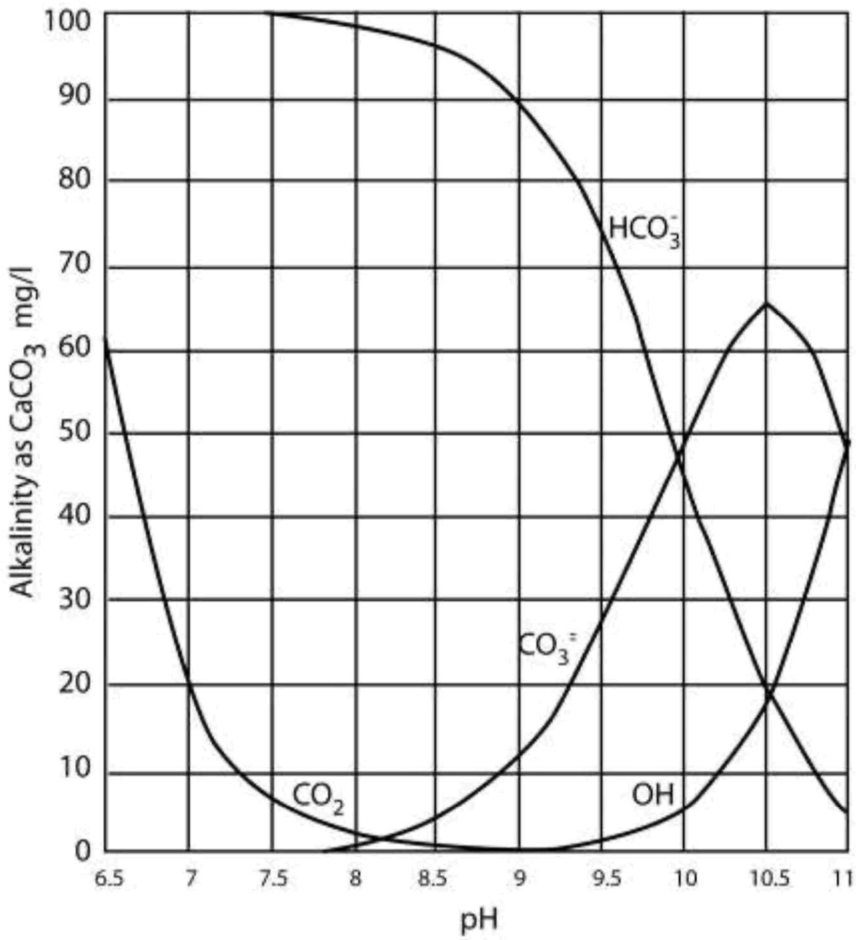


Figure 1-2. Relationship between *p*H and alkalinity (Sawyer et al., 1994).

1.3.5 Physical Factors

1.3.5.1 Light

The intensity and spectral composition of light penetrating a pond surface significantly affect all resident microbial activity. In general, activity increases with increasing light intensity until the photosynthetic system becomes light saturated. The rate at which photosynthesis increases in proportion to an increase in light intensity, as well as the level at which an organism's photosynthetic system becomes light saturated, depends upon the particular biochemistry of the species (Lynch and Poole, 1979; Pearson, 2005). In ponds, photosynthetic O_2 production has been shown to be relatively constant within the range of 5,380 to 53,800 lumens/m^2 light intensity with a reduction occurring at higher and lower intensities (Pipes, 1961; Paterson and Curtis, 2005).

The spectral composition of available light is also crucial in determining photosynthetic activity.

The ability of photosynthetic organisms to utilize available light energy depends primarily upon their ability to absorb the available wavelengths. This absorption ability is determined by the specific photosynthetic pigment of the organism. The main photosynthetic pigments are chlorophylls and phycobilins. Bacterial chlorophyll differs from algal chlorophyll in both chemical structure and absorption capacity. These differences allow the photosynthetic bacteria to live below dense algal layers where they can utilize light not absorbed by the algae (Lynch and Poole, 1979; Pearson, 2005).

The quality and quantity of light penetrating the pond surface to any depth depend on the presence of dissolved and particulate matter as well as the water absorption characteristics. The organisms themselves contribute to water turbidity, further limiting the depth of light penetration. Given the light penetration interferences, photosynthesis is significant only in the upper pond layers. This region of net photosynthetic activity is called the euphotic zone (Lynch and Poole, 1979; Pearson, 2005).

Light intensity from solar radiation varies with the time of day and difference in latitudes. In cold climates, light penetration can be reduced during the winter by ice and snow cover. Supplementing the treatment ponds with mechanical aeration may be necessary in these regions during that time of year.

1.3.5.2 Temperature

Temperature at or near the surface of the aerobic environment of a pond determines the succession of predominant species of algae, bacteria, and other aquatic organisms. Algae can survive at temperatures of 5 - 40°C. Green algae show most efficient growth and activity at temperatures of 30 - 35°C. Aerobic bacteria are viable within a temperature range of 10 - 40°C; 35 - 40°C is optimum for cyanobacteria (Anderson and Zweig, 1962; Gloyna et al., 1976; Paterson and Curtis, 2005; Crites et al., 2006).

As the major source of heat for these systems is solar radiation, a temperature gradient can develop in a pond with depth. This will influence the rate of anaerobic decomposition of solids that have settled at the bottom of the pond. The bacteria responsible for anaerobic degradation are active in temperatures from 15 - 65°C. When they are exposed to lower temperatures, their activity is reduced.

The other major source of heat is the influent water. In sewerage systems with no major inflow or infiltration problems, the influent temperature is higher than that of the pond contents. Cooling influences are exerted by evaporation, contact with cooler groundwater and wind action.

The overall effect of temperature in combination with light intensity is reflected in the fact that nearly all investigators report improved performance during summer and autumn months when both temperature and light are at their maximum. The maximum practical temperature of wastewater ponds is likely less than 30°C, indicating that most ponds operate at less than optimum temperature for anaerobic activity (Oswald, 1968b; Oswald, 1996; Paterson and Curtis, 2005; Crites et al., 2006).

During certain times of the year, cooler, denser water remains at depth, while the warmer water stays at the surface. Water temperature differences may cause ponds to stratify throughout their depth. As the temperature decreases during the fall and the surface water cools, stratification decreases and the deeper water mixes with the cooling surface water. This phenomenon is called *mixis*, or pond overturn. As the density of water decreases and the temperature falls below 4°C, winter stratification can develop. When the ice cover breaks up and the water warms, a spring overturn can also occur.

Pond overturn, which releases odorous compounds into the atmosphere, can generate complaints from property owners living downwind of the pond. The potential for pond overturn during certain times of the year is the reason why regulations may specify that ponds be located downwind, based on prevailing winds during overturn periods, and away from dwellings.

1.3.5.3 Wind

Prevailing and storm-generated winds should be factored into pond design and siting as they influence performance and maintenance in several significant ways:

- Oxygen transfer and dispersal: By producing circulatory flows, winds provide the mixing needed for O_2 transfer and diffusion below the surface of facultative ponds. This mixing action also helps disperse microorganisms and augments the movement of algae, particularly green algae.
- Prevention of short circuiting and reduction of odor events: Care must be taken during design to position the pond inlet/outlet axis perpendicular to the direction of prevailing winds to reduce short circuiting, which is the most common cause of poor performance. Consideration must also be made for the transport and fate of odors generated by treatment by-products in anaerobic and facultative ponds.
- Disturbance of pond integrity: Waves generated by strong prevailing or storm winds are capable of eroding or overtopping embankments. Some protective material should extend one or more feet above and below the water level to stabilize earthen berms.
- A study by Wong and Lloyd (2004) indicates that wind effects can reduce hydraulic retention time.

1.3.6 Pond Nutritional Requirements

In order to function as designed, the wastewater pond must provide sufficient macro- and micronutrients for the microorganisms to grow and populate the system adequately. It should be understood that a treatment pond system should be neither overloaded nor underloaded with wastewater nutrients.

1.3.6.1 Nitrogen

Nitrogen can be a limiting nutrient for primary productivity in a pond. Figure 1-3 represents the various forms that N typically takes over time in these systems. The conversion of organic N to various other N forms results in a total net loss (Assenzo and Reid, 1966; Pano and Middlebrooks, 1982; Middlebrooks et al. 1982; Middlebrooks and Pano, 1983; Craggs, 2005). This N loss may be due to algal uptake or bacterial action. It is likely that both mechanisms contribute to the overall total N reduction. Another factor contributing to the reduction of total N is the removal of gaseous NH_3 under favorable environmental conditions. Regardless of the specific removal mechanism involved, NH_3 removal in facultative wastewater ponds have been observed at levels greater than 90 percent, with the major removal occurring in the primary cell of a multicell pond system (Middlebrooks et al., 1982; Shilton, 2005; Crites et al., 2006).

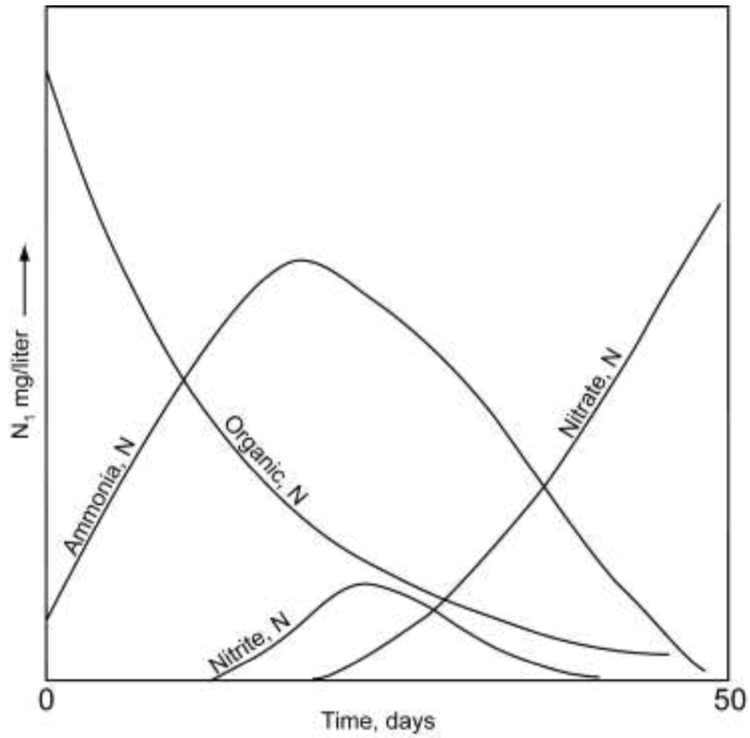


Figure 1-3. Changes occurring in forms of *N* present in a pond environment under aerobic conditions (Sawyer et al., 1994).

1.3.6.2 Phosphorus

Phosphorus (*P*) is most often the growth-limiting nutrient in aquatic environments. Municipal wastewater in the United States is normally enriched in *P* even though restrictions on *P*-containing compounds in laundry detergents in some states have resulted in reduced concentrations since the 1970s. As of 1999, 27 states and the District of Columbia had passed laws prohibiting the manufacture and use of laundry detergents containing *P*. However, phosphate (PO_4^{-3}) content limits in automatic dishwashing detergents and other household cleaning agents containing *P* remain unchanged in most states**.** With a contribution of approximately 15 percent, the concentration of *P* from wastewater treatment plants is still adequate to promote growth in aquatic organisms (Canadian Environmental Protection Act, 2009).

In aquatic environments, *P* occurs in three forms: (1) particulate *P*, (2) soluble organic *P*, and (3) inorganic *P*. Inorganic *P*, primarily in the form of orthophosphate ($OP(OR)_3$), is readily utilized by aquatic organisms. Some organisms may store excess *P* as polyphosphate. At the same time, some PO_4^{-3} is continuously lost to sediments, where it is locked up in insoluble precipitates (Lynch and Poole, 1979; Craggs, 2005; Crites et al., 2006).

Phosphorus removal in ponds occurs via physical mechanisms such as adsorption, coagulation, and precipitation. The uptake of *P* by organisms in metabolic functions as well as for storage can also contribute to its removal. Removal in wastewater ponds has been reported to range from 30 - 95 percent (Assenzo and Reid, 1966; Pearson, 2005; Crites et al., 2006).

Algae discharged in the final effluent may introduce organic *P* to receiving waters. Excessive algal "afterblooms" observed in waters receiving effluents have, in some cases, been attributed to *N* and *P* compounds remaining in the treated wastewater.

1.3.6.3 Sulfur

Sulfur (*S)* is a required nutrient for microorganisms, and it is usually present in sufficient concentration in natural waters. Because *S* is rarely limiting, its removal from wastewater is usually not considered necessary. Ecologically, *S* compounds such as hydrogen sulfide (H_2S and sulfuric acid (H_2SO_4) are toxic, while the oxidation of certain *S* compounds is an important energy source for some aquatic bacteria (Lynch and Poole, 1979; Pearson, 2005).

1.3.6.4 Carbon

The decomposable organic *C* content of a waste is traditionally measured in terms of its BOD_5, or the amount of O_2 required under standardized conditions for the aerobic biological stabilization of the organic matter over a certain period of time. Since complete treatment by biological oxidation can take several weeks, depending on the organic material and the organisms present, standard practice is to use the BOD_5 as an index of the organic carbon content or organic strength of a waste. The removal of BOD_5 is a primary criterion by which treatment efficiency is evaluated.

BOD_5 reduction in wastewater ponds ranging from 50 - 95 percent has been reported in the literature. Various factors affect the rate of reduction of BOD_5. A very rapid reduction occurs in a wastewater pond during the first five to seven days. Subsequent reductions take place at a sharply reduced rate. BOD_5 removals are generally much lower during winter and early spring than in summer and early fall. Many regulatory agencies recommend that pond operations do not include discharge during cold periods.

CHAPTER 2

PLANNING, FEASIBILITY ASSESSMENT AND SITE SELECTION

2.1 INTRODUCTION

During the early planning stages of a wastewater management project, it is prudent to consider as many alternatives as possible in order to select the technically appropriate and most cost effective process. The feasibility of using pond systems described in this manual depends significantly on site conditions, climate, and related factors. This chapter describes a sequential approach that first determines potential feasibility, land area requirements for treatment and potential sites. The second step is to evaluate these sites, based on technical and economic factors, and to select one or more for detailed investigation. The final step involves detailed field investigations, identification of the most cost-effective alternative and development of the criteria needed for final design. Additional information can be found in Borowitzka & Borowitzka (1988a, b), Crites et al., (2006), Reed et al., (1995) and Shilton (2005).

2.2 CONCEPT EVALUATION

Once the decision to use pond technology has been made, a further review of the types of ponds appropriate to the site should be undertaken. A number of factors must be considered, including but not limited to, required effluent quality, effluent discharge point, site topography, soils, geology, climate and groundwater conditions. Specific information is needed related to geotechnical characteristics, such as surface and groundwater hydrology, proximity to surface water for discharge, site permeability and lining requirements, feasibility of siting the ponds within or outside a flood plain, and presence of bedrock or groundwater within the depth of excavation (Crites et al., 2006).

2.3 RESOURCES REQUIRED

The identification of potential sites is made using the information contained in publicly available sources, such as existing maps and other published documents. Climate data, for example, can be obtained from the National Oceanic and Atmospheric Administration (NOAA) (http://www.noaa.gov/climate.html), at Worldclimate (http://www.worldclimate.com), and at Weather Base (http://www.weatherbase.com). Solar maps can be found at the National Renewable Energy Resources website (http://www.nrel.gov/gis/solar.html). Local or community maps should indicate such features as topographical features, water features such as ponds and streams, flood hazard zones, community layout and land use (e.g., residential, commercial, industrial, agricultural, forest), existing water supply and sewage systems, anticipated areas of growth and expansion, and soil types within the community and adjacent areas. Sources for these maps include the U.S. Geological Survey (USGS) (http://www.usgs.gov/pubprod/), the Natural Resources Conservation Service (NRCS) (http://www.soils.usda.gov/), state agencies, as well as local planning and zoning agencies. Much of this work can now be done using the Geographical Information System (GIS) and most of the layers are now available either for free or at low cost (http://www.gis.com).

2.3.1 Estimating Land Area Required for Treatment Ponds

The area estimate for a pond system will depend on the effluent quality required, the type of pond system proposed and the geographic location. A facultative pond or an integrated system of wastewater ponds in the southern United States will require less area than the same pond or integrated pond system in the northern states. The pond areas given in Table 2-1 are for total project area and include an allowance for dikes, roads and unused portions of the site (after Reed et al., 1995 and F.B. Green, pers. comm.).

The land area required for a community wastewater flow of 3,785 m^3/d (1 mgd) is estimated below for three types of locations: a cold climate, a temperate climate (the mid-Atlantic states), and a warm climate (the southern states). Allowances are made for any preliminary treatment that might be required and for unused portions of the general site area.

Table 2-1. Land Area Estimates for 3,785 m^3/d (1 mgd) Systems.

Treatment System	North (ha*)	Mid-Atlantic (ha*)	South (ha*)
Aerobic	NA**	NA**	13
Facultative	67	44	20
Controlled Discharge	65	65	65
Partial-Mix Aerated	20	15	12
Complete-Mix Aerated	2	2	1
AIWPS®***	12	8	6

* 1 ha = 2.471 ac
** NA = not applicable
***See discussion of land requirements in Chapter 4.

2.4 SITE IDENTIFICATION

The information collected should be used in conjunction with current maps of the community area to determine if there are potential sites for wastewater treatment within a reasonable distance to the source. The potential sites should be plotted on the community maps. Local knowledge regarding land use commitments and costs and a technical ranking procedure should be brought into the decision-making process. Critical factors at this point are how close the site is to the wastewater source and whether there is access to a reuse site (e.g., agricultural fields for use as irrigation supply) or to surface water for final discharge. Characterization of the site soils should be undertaken if percolation to groundwater is a disposal option.

2.4.1 Potential for Floods

Locating a wastewater system within a flood plain can be either an asset or a liability, depending on the approach used for planning and design. Flood-prone areas may be undesirable because of variable drainage characteristics and potential flood damage to the structural components of the system. On the other hand, flood plains and similar terrain may be the only deep soils in the area and the only location low enough to permit conveyance by gravity. If permitted by the regulatory authorities, utilization of such sites for wastewater or sludge storage can be an integral part of a flood-plain management plan. Off-site storage of wastewater or sludge should be included as a design feature if the site is to be flooded on an as-needed basis. An example of a design of a wastewater treatment system located in a floodplain can be found in Chapter 4.

Maps of flood-prone areas have been produced by the USGS for many areas of the UnitedStates as part of the Uniform National Program for Managing Flood Losses. The maps are based on the standard 7.5' USGS topographic sheets. They identify areas with a potential of a 1-in-100 chance of flooding in a given year. The hydrologic maps can be obtained from USGS (http://edc2.usgs.gov/pubslists/booklets/usgsmaps/usgsmaps.php). Other detailed flood information is available from local offices of the U.S. Army Corps of Engineers and flood-control districts. If the screening process identifies potential sites in flood-prone areas, local authorities must be consulted to identify regulatory requirements before beginning any detailed site investigation. At the very least, in designing a system within a flood plain, incorporated tank walls, structural openings, motor drives and pumps should be raised so that they are above the 100-year flood level.

2.4.2 Water Rights

Riparian water laws, primarily in states east of the Mississippi River, protect the rights of landowners to use the water along a watercourse. Appropriative water rights laws in the western states protect the rights of prior users of the water basin. Adoption of any of the pond concepts for wastewater treatment can have a direct impact on water rights concerns:

- Site drainage, both quantity and quality, may be affected.
- A zero discharge system, or a new discharge location, will affect the quantity of flow in a body of water where the discharge previously existed.
- Operational considerations for land treatment systems may alter the pattern and the quality of discharges to a water body.

In addition to surface waters in well-defined channels or basins, many states also regulate or control other superficial waters and the groundwater beneath the surface. State and local discharge requirements for the proposed project should be determined prior to the development of the design. If the project has the potential to generate legal questions, a water rights attorney should be consulted.

2.5 SITE EVALUATION

The next phase of the site and system selection process involves developing field surveys to confirm map data and field testing in order to provide the data needed for design. It also includes making an estimate of capital and operation and maintenance costs so that the sites identified can be compared. A design concept and a site are selected for final design based on these results. Each site evaluation must include the following information:

- Property ownership, physical dimensions of the site, current and future land use
- Surface and groundwater conditions: location and depth of water supply wells and injection wells, surface water flooding or surface water bodies within one mile of the proposed site, fluctuation in groundwater levels, other potential drainage problems
- Quality and use of groundwater, e.g., is area designated as a wellhead protection area or other critical recharge zone?

- Characterization of the soil profile to the depth of the first limiting condition such as the seasonal high water table, aquitard, or bedrock, or bottom of the excavation, whichever is deeper
- Reclamation of the site describing the existing vegetation, historical causes for disturbance, previous reclamation efforts, historical site contamination from anthropogenic or natural sources, need for grading or other terrain modification
- Current and future land use of adjacent properties
- Environmental impact and habitat evaluation

2.5.1 Soil and Groundwater Characterization

Table 2-2 presents a sequential approach to field testing to define the physical and chemical characteristics of the on-site soils. In addition to the on-site test pits and borings, exposed soil profiles in road cuts, borrow pits, and plowed fields on or near the site should be examined and a preliminary geotechnical investigation of the highest ranked potential sites should be undertaken.

Backhoe test pits to a 3 m depth, or to 6 - 8 m for deeper ponds, such as AIWPS® Advanced Facultative Pond with stable methane fermentation zones (Oswald and Green, 2000), are recommended, where soil conditions permit, in each of the major soil types on the site. These samples should be reserved for future testing. The walls of the test pit should be carefully examined to define the characteristics of the soil (Reed and Crites, 1984a; U.S. EPA, 1980c; U.S. EPA, 1984; Silva-Tulla and Flores-Berrones, 2005; Crites et al., 2006). The test pit should be left open long enough to determine if there is groundwater seepage and the highest level attained should be recorded. Equally important is the observation of any indication of seasonally high groundwater, most typically demonstrated by mottled or hydric soils (Vasilas et al., 2010).

Soil borings should penetrate to below the groundwater table if it is within 10 - 15 m of the surface. At least one boring should be located in every major soil type on the site. If generally uniform conditions prevail, one boring for every 1 - 2 ha is recommended for large-scale systems. For small systems (<5 ha), three to five shallow borings spaced over the entire site should be sufficient.

Groundwater encountered during test borings should be analyzed for general chemistry (pH, conductivity, nitrate, metals, and major ions using drinking water methods (see http://www.epa.gov/ogwdw/methods/methods_inorganic.pdf or 40 CFR 141) to establish background conditions. Seeps, perched saturated zones, depth of mottled zones, and depth to the seasonal high water table should be recorded on the site plan.

2.5.2 Buffer Zones

Prior to the site investigation, state and local requirements for buffer zones or setback distances should be researched to ensure that there is adequate area on site or that additional acreage can be obtained. Most requirements for buffer zones or separation distances are based on aesthetic considerations and to avoid potential complaints. A number of studies have been conducted at both conventional and land treatment facilities on aerosols and the results indicate that there is very little, if any, health risk to adjacent populations (Sorber et al., 1976; Reed, 1979; Sorber et al., 1984). Therefore, designing extensive buffer zones for aerosol containment is not recommended.

Most wastewater ponds and natural lakes are holo- or dimictic, overturning for a period during the spring and fall, which brings deeper anaerobic or anoxic water and bacterial solids to the surface, releasing volatile, odiferous compounds into the atmosphere. A typical requirement in these cases is to locate such ponds at least 0.4 km from human habitations.

Table 2-2. Sequence of Field Testing, Typical Order, reading from left to right (Crites in Asano and Pettygrove, 1984)

Comments	**Test Pits**	**Soil Borings**	**Infiltration Tests**[a]	**Soil Chemistry**[b]
Type of Test	Backhoe pit, inspect road cuts	Drill or auger log review of local wells for soil data and water level	Basin method[c] if possible	NRCS[d] Surveyed
Data needed	Depth of profile, texture, structure, restriction layers	Depth to ground water, depth to barrier	Infiltration rate	Nitrogen, phosphorus, metals, other potential site specific contaminants
Estimate	Need for hydraulic conductivity tests	Groundwater flow direction	Hydraulic capacity	Soil amendments, crop limitations
Then test for	Hydraulic conductivity, if needed	Horizontal conductivity, if needed		Quality of any percolate
Estimate	Loading rates	Groundwater mounding, drainage needs		Depends on site, soil uniformity, character of waste
Number of tests	3-5/site, more for large sites, lack of soil uniformity	3-5/site, more for lack of soil uniformity	2/site, more for large site or lack of soil uniformity	

[a]Required only for land application of wastewater; some definition of subsurface permeability needed for pond and sludge systems
[b]Typically needed for land application of sludges or wastewaters
[c]Crites et al., 2006
[d]Natural Resources Conservation Service

2.6 SITE AND PROCESS SELECTION

At this point, the evaluation procedure will have identified potential sites for a particular treatment alternative and field investigations will have been conducted to obtain data for the feasibility determination. Evaluation of the field data will determine whether the site requirements are adequate. If site conditions are favorable, it can be concluded that the site is at least a candidate for the intended concept. If only one site and related treatment concept emerge from this screening process, the focus can shift to final design and perhaps additional detailed field tests to support the design process. If more than one site for a particular concept, and/or more than one concept remains technically viable after the screening process, it will be necessary to do a preliminary analysis to identify the most cost-effective alternative.

The design criteria presented in later chapters should be used to develop the preliminary design of the concept. The design should then be used as the basis for a preliminary cost estimate for capital and operation/maintenance that should include the cost of purchasing the land as well as pumping or transport costs to move the wastes from sources to the site. In many cases the final selection of process or type of pond system will also be influenced by the social and institutional acceptability of the proposed site and treatment facility to be developed on it.

2.7 DESIGN CRITERIA OF MUNICIPAL WASTEWATER TREATMENT PONDS

Most states have design criteria for wastewater treatment ponds, but the depth of detail provided by each state varies widely (see Appendix A). Detailed sets of criteria are provided for the State of Nebraska, and the State of Iowa as examples. The Recommended Standards for Wastewater Facilities, known as the 10 States Standards, published by The Great Lakes-Upper Mississippi River Board of State and Provincial Public Health and Environmental Managers (Health Research, Inc., 2004), or a modification of these standards, is often cited as a reference.

2.8 STATE DESIGN STANDARDS

2.8.1 10 States Standards

The 10 States Standards recommend a minimum separation of 1.2 m between the bottom of the pond and the maximum groundwater elevation and a minimum separation of 3 m between the pond bottom and any bedrock formation. For a conventional facultative treatment pond system design, an average five day biochemical oxygen demand (BOD_5) loading from 17 - 40 kg/ha/d for the primary pond(s) with a detention time of 90 - 120 d is recommended. Controlled discharge facultative treatment pond systems have different requirements (see Chapter 7).

For the development of final design parameters for aerated treatment pond systems, it is recommended that actual experimental data be developed; however, the minimum detention time may be estimated using the following formula applied separately to each aerated cell:

$$t = E / [2.3k_1 \times (100 - E)] \qquad (2\text{-}1)$$

where: t = detention time in days; E = percent of BOD_5 to be removed in an aerated pond; and k_1 = reaction coefficient, aerated pond, base 10. For normal domestic wastewater, the k_1 value may be assumed to be 0.12/d at 20 °C and 0.06/day at 1 °C.

Additional storage volume should be considered for sludge, and in northern climates, for ice cover. If aeration equipment is used, it should be capable of maintaining a minimum dissolved oxygen (DO) level of 2 mg/L in the ponds at all times (Health Research, Inc., 2004).

The 10 States Standards recommend that, at a minimum, a wastewater treatment pond system consist of three cells designed to facilitate both series and parallel operations. The maximum size of a conventional pond cell should be 16 ha. Two-cell systems may be utilized in very small installations. Guidance is also provided on pond construction details.

2.8.2 Summary of Other Criteria

Other criteria to be considered are briefly discussed here.

Freeboard: The minimum and maximum recommended freeboard varies from 0.6 - 0.9 m. Some states allow a 0.3 m freeboard for small systems, while others specify 0.6 m.

Pond Bottom: The majority of the states include a detailed description of the materials that are acceptable for sealing the pond bottom and sides of the dikes. Permissible seepage rate or hydraulic conductivity is specified in all pond criteria; an emphasis is placed on groundwater protection. Natural earth, bentonite, asphalt, concrete and synthetic liners are acceptable in most cases.

Flow Distribution: Design of structures split hydraulic and organic loads effectively between two primary cells is a common requirement. This is frequently expanded to include multiple inlet points to accomplish even distribution of the flow. Most states allow one discharge point from secondary cells, but frequently recommend multiple outlets from primary cells.

Influent Discharge Apron: A common requirement for the influent discharge to a primary cell is that the flow should enter a shallow, saucer-shaped depression and that the end of the discharge line rest on a concrete apron large enough to prevent soil erosion.

Piping and Pipe Connections: In most states, the acceptable type of piping materials is specified, such as ductile iron, plastic or lined pipes. Where pipes penetrate the pond seal, anti-seep collars or similar devices should be used to prevent leaks around the pipes.

Hydraulic capacity frequently is specified as 250 percent of the design maximum day flow rate of the system. Most states specify that the piping must allow for parallel and series operation of multi-cell systems, and that provisions for by-passing each cell be provided. Provisions for draining each cell are also usually required.

Settling Ponds: Settling ponds may sometimes be referred to as polishing ponds, but they are not synonymous. A polishing pond is any pond in the treatment train that follows the facultative pond. A settling pond is usually placed at the beginning of the treatment series, but may also be a pond at the end of the system. The amount of time that effluent is retained in a settling pond can vary from 24 hours to some proportion of the time it takes the water to move through entire system. This can result in a hydraulic residence time (HRT) greater than 10 - 15 d. Most state standards distinguish between the two, but may not provide an explanation of the need for a correctly designed settling pond to help control algae in the effluent before it is discharged (Green, 2009). An HRT of 2 days at average design flow rate will provide better control.

Miscellaneous: All of the criteria specify that some type of fencing be put in place to limit access and discourage trespassing. Some states require only a fence with a few strands of barbed wire to prevent animals from entering the site. Others are more conservative and specify that a chain-link fence with barbed wire strands at the top be installed to discourage access. Gates should be of sufficient width to allow maintenance vehicles to enter the facility and should be provided with a lock.

An all-weather road to the pond site should be built and maintained to allow year-round access for operation and maintenance. The requirement that permanent warning signs are to be placed conspicuously around the site designating the nature of the facility is included in all the state criteria. Signs should be posted every 150 m along the perimeter of the facility.

Flow measurement parameters vary, but in all cases, some type of flow measuring device is recommended or required. Groundwater monitoring wells are required by most states. Pond level gauges are specified by most states. A service building that contains a laboratory and space for storage and maintenance of equipment is required in most criteria.

2.8.3 Criteria for Types of Ponds

As shown in Appendix A, many state guidances do not indicate what criteria are specific to the type and proposed operation schedule of a pond system (e.g., anaerobic, partial-mix, complete-mix, controlled discharge or hydrographically controlled). When seeking advice as to the factors that need to be considered in a specific pond design, it is advisable to consult with the relevant state regulatory agency. For general guidance, the Minnesota, Nebraska, South Dakota, Montana, Wyoming, Tennessee and several other state criteria provide sufficiently detailed information that can be used to develop an appropriate design.

CHAPTER 3

DESIGN OF MUNICIPAL WASTEWATER TREATMENT PONDS

3.1 INTRODUCTION

Wastewater treatment ponds existed and provided adequate treatment long before they were acknowledged as an "alternative" technology to mechanical plants in the United States. With legislative mandates to provide treatment to meet certain water quality standards, engineering specifications designed to meet those standards were developed, published and used by practitioners. The basic designs of the various pond types are presented in this chapter. Design equations and examples are found in the Appendix C.

3.2 ANAEROBIC PONDS

An anaerobic pond is a deep impoundment, essentially free of DO. The biochemical processes take place in deep basins, and such ponds are often used as preliminary treatment systems. Anaerobic ponds are not aerated, heated or mixed.

Anaerobic ponds are typically more than eight feet deep. At such depths, the effects of oxygen (O_2) diffusion from the surface are minimized, allowing anaerobic conditions to dominate. The process is analogous to that of a single-stage unheated anaerobic digester. Preliminary treatment in an anaerobic pond includes separation of settleable solids, digestion of solids and treatment of the liquid portion. They are conventionally used to treat high strength industrial wastewater or to provide the first stage of treatment in municipal wastewater pond treatment systems.

Anaerobic ponds have been especially effective in treating high strength organic wastewater. Applications include industrial wastewater and rural community wastewater treatment systems that have a significant organic load from industrial sources. BOD_5 removals may reach 60 percent. The effluent cannot be discharged due to the high level of BOD_5 that remains. Anaerobic ponds are not an appropriate design for locations that do not have sufficient land available. The potential to give off odors, if not properly managed, makes them less a reliable choice for municipal wastewater treatment. Finally, the anaerobic process may require long retention times, especially in cold climates, as anaerobic bacteria are inactive below 15° C. As a result, anaerobic ponds are not widely used for municipal wastewater treatment in the northern United States.

Because anaerobic ponds are deep and generally have a relatively longer hydraulic residence time (HRT), so solids settle, retained sludge is digested, and BOD_5 concentration is reduced. Raw wastewater enters near the bottom of the pond and mixes with the active microbial mass in the sludge blanket. Anaerobic conditions prevail except for a shallow surface layer in which excess undigested grease and scum are concentrated. Sometimes aeration is provided at the surface to control odors. An impervious crust that retains heat and odors will develop if surface aeration is not provided. The discharge is located near the side opposite the influent. Anaerobic ponds are usually followed by aerobic or facultative ponds to provide additional treatment.

The anaerobic pond is usually preceded by a bar screen and a Parshall flume with a flow recorder to determine the inflow. A cover can be provided to trap and collect CH_4, a by-product of the process, for use elsewhere.

3.2.1 Microbiology

Anaerobic microorganisms convert organic materials into stable products, such as CO_2 and CH_4. The degradation process involves two separate but interrelated phases: acid formation and methane production. During the acid phase, bacteria convert complex organic compounds (carbohydrates, fats, and proteins) to simple organic compounds, mainly short-chain volatile organic acids (acetic, propionic, and lactic acids). The anaerobic bacteria involved in this phase are called "acid formers," and are classified as non-methanogenic microorganisms. During this phase, the chemical oxygen demand (COD) is low and BOD_5 reduction occurs, because the short-chain fatty acids, alcohols, and other organic compounds can be used by many aerobic microorganisms.

The methane production phase involves an intermediate step. First, bacteria convert the short-chain organic acids to acetate, hydrogen gas (H_2), and CO_2. This intermediate process is referred to as acetogenesis. Subsequently, several species of strictly anaerobic bacteria called "methane formers" convert the acetate, H_2, and CO_2 into CH_4 through one of two major pathways. This process is referred to as methanogenesis. During this phase, waste stabilization occurs, indicated by the formation of CH_4. The two major pathways of methane formation are

1) the breakdown of acetic acid to form methane and carbon dioxide:

$$CH_3COOH \rightarrow CH_4 + CO_2 \tag{3-1}$$

and

2) the reduction of carbon dioxide by hydrogen gas to form methane:

$$CO_2 + 4H_2 \rightarrow CH_4 + 2\,H_2O. \tag{3-2}$$

3.2.2 Equilibrium

When the system is working properly, the two phases of degradation occur simultaneously in dynamic equilibrium. The volatile organic acids are converted to methane at the same rate that they are formed from the more complex organic molecules. The growth rate and metabolism of the methanogenic bacteria can be adversely affected by small fluctuations in *p*H substrate concentrations and temperature, but the performance of acid-forming bacteria is more tolerant of a wide range of conditions. When anaerobic ponds are stressed by shock loads or temperature fluctuations, CH_4 bacteria activity occurs more slowly than the acid formation and an imbalance occurs. Intermediate volatile organic acids accumulate and the *p*H drops. The methanogens are further inhibited and the process eventually fails without corrective action. For this reason, the CH_4 formation phase is the rate-limiting step and must not be inhibited. For an anaerobic pond to function properly, the design must incorporate the limiting characteristics of these methanogens.

3.2.3 Establishing and Maintaining Equilibrium

The system must operate at conditions favorable for the performance of methanogenic bacteria. Ideally, temperatures should be maintained within the range of 25 to 40° C. Anaerobic activity decreases rapidly at temperatures below 15° C, and virtually ceases when water temperature drops below freezing (0^{o} C). The *p*H value should range from 6.6 to 7.6, and should not drop below 6.2 as CH_4 bacteria cannot function below this level. Sudden fluctuations of *p*H will upset methanogenic activity and inhibit pond performance. Alkalinity should range from 1,000 to 5,000 mg/L.

Volatile acid concentration is an indicator of process performance. Ideally, volatile acid concentrations will be low if the pond system is working properly and dynamic equilibrium between acid formation and consumption is maintained. As a general rule, concentrations should be less than 250 mg/L. Inhibition occurs at volatile acid concentrations in excess of 2,000 mg/L. Table 3-1 presents optimum and extreme operating ranges for CH_4 formation. The rate of CH_4 formation drops dramatically outside these ranges. In addition to adhering to these guidelines, sufficient nutrients, such as *N* and *P* must be available. Concentrations of inhibitory substances, including NH_3 and calcium, should be kept to a minimum. High concentrations of these inhibitors will reduce biological activity. Concentration of free NH_3 in excess of 1,540 mg/L will result in severe toxicity, but concentrations of NH_4^+ must be greater than 3,000 mg/L to produce the same effect. Maintaining a *p*H of 7.2 or below will ensure that most NH_3 will be in the form of NH_4^+, so that higher concentrations can be tolerated with little effect. Table 3-2 provides guidelines for acceptable ranges of other inhibitory substances.

Table 3-1. Ideal Operating Ranges for Methane Fermentation.

Variable	Optimal	Extreme
Temperature, °C	30-35	25-40
*p*H	6.8-7.4	6.2-7.8
Oxidation-Reduction Potential, MV	-520 to -530	-490 to -550
Volatile Acids, mg/L as Acetic	50-500	2000
Alkalinity, mg/L as $CaCO_3$	2000-3000	1000-5000

Table 3-2. Concentrations of Inhibitory Substances (Parkin and Owen, 1986.)

Substance	Moderately Inhibitory (mg/L)	Strongly Inhibitory (mg/L)
Sodium	3,500-5,500	8,000
Potassium	2,500-4,500	12,000
Calcium	2,500-4,500	8,000
Magnesium	1,000-1,500	3,000
Sulfides	200	>200

Anaerobic ponds produce undesirable odors unless provisions are made to oxidize the escaping gases. Gas production must be minimized (sulfate [SO_4^{-2}]) concentration must be reduced to less than 100 mg/L) or aeration should be provided at the surface of the pond to oxidize the escaping gases. Aerators must not introduce DO to depths below the top 0.6 - 0.9 m (2 - 3 ft) so that anaerobic activity at depth is not inhibited.

Another option is to locate the pond in a remote area. A relatively long detention time is required for organic stabilization due to the slow growth rate of the CH_4 formers and sludge digestion. Wastewater seepage into the groundwater may be a problem. Providing a liner for the pond can help avoid this problem.

Advantages and Disadvantages
The advantages of anaerobic ponds are several: sludge removal is infrequently needed; 80-90 percent BOD_5 removal can be expected; the energy requirements to run the plant are low or none; and operation and maintenance (O&M) is relatively uncomplicated.

On the other hand, they are not designed to produce effluent that can be discharged; the ponds can emit unpleasant odors; and the rate of treatment is dependent on climate and season.

3.2.4 Design Criteria

The design of anaerobic ponds is not well defined and a widely accepted overall design equation does not exist. Design is often based on organic loading rates, surface or volumetric loading rates and HRT derived from pilot plant studies and observations of existing operating systems. States in which ponds are commonly used often have regulations governing their design, installation, and management. For example, state regulations may require specific organic loading rates, detention times, embankment slope ratio of 1 to 3 to 1 to 4, and maximum allowable seepage of 1 to 6 mm/d.

3.2.5 Performance

System performance depends on loading, temperature, and whether the *p*H is maintained within the optimum range. Tables 3-3 and 3-4 show expected removal efficiencies for municipal wastewaters. In cold climates, detention times as great as 50 days and volumetric loading rates as low as 0.04 kg $BOD_5/m^3/d$ may be required to achieve 50 percent reduction in BOD_5. Effluent TSS will range between 80 and 160 mg/L. The effluent is not suitable for direct discharge to receiving waters. Pond contents that are black indicate that it is functioning properly.

Table 3-3. BOD_5 Reduction as a Function of Detention Time for Temperatures Greater than 20 $^{\circ}$C (World Health Organization, 1987)

Detention Time (Days)	BOD_5 Reduction (Percent)
1	50
2.5	60
5	70

Table 3-4. BOD_5 Reduction as a Function of Detention Time and Temperature (World Health Organization, 1987)

Temperature (°Celsius)	Detention Time (Days)	BOD_5 Reduction (Percent)
10	5	0-10
10-15	4-5	30-40
15-20	2-3	40-50
20-25	1-2	40-60
25-30	1-2	60-80

3.2.6 Operation and Maintenance

Operation and maintenance requirements of an anaerobic pond are minimal. A daily grab sample of influent and effluent should be taken and analyzed to ensure proper operation. Aside from sampling, analysis, and general upkeep, the system is virtually maintenance-free. Solids accumulate in the pond bottom and require removal infrequently (5-10 years), depending on the amount of inert material in the influent and the temperature. Sludge depth should be measured annually.

3.2. Costs

The primary costs associated with constructing an anaerobic pond are the cost of the land, building earthwork appurtenances, constructing the required service facilities, and excavation. Costs for forming the embankment, compacting, lining, service road and fencing, and piping and pumps must also be considered. Operating costs and energy requirements are minimal.

3.2.8 Design Models and Example Calculations

Anaerobic treatment ponds are typically designed on the basis of volumetric loading rate and HRT. Although often done, it is probably inaccurate to design on the basis of surface loading rate. Design should be based on the volumetric loading rate, temperature of the liquid, and the HRT. Areal loading rates that have been used around the world are shown in Table 3-5. It is possible to approximate the volumetric loading rates by dividing by the average depth of the ponds and converting to the proper set of units.

In climates where the temperature exceeds 22 °C, the following design criteria should yield a BOD_5 removal of 50 percent or better (World Health Organization, 1987).

Volumetric loading up to 300 g $BOD_5/m^3/d$
HRT of approximately 5 d
Depth between 2.5 and 5 m

In cold climates, detention times as great as 50 d and volumetric loading rates as low as 40 g $BOD_5/m^3/d$ may be required to achieve 50 percent reduction in BOD_5.

Table 3-5. Design and Operational Parameters for Anaerobic Ponds Treating Municipal Wastewater (See p. xiv for Conversion Table)

ALR BOD_5		Est. VLR		Removal		Depth	HRT	Refs.
lbs/ac/d		lbs/1000 ft^3		Percent		Ft	D	
Summer	Winter	Summer	Winter	Summer	Winter			
360		2.34		75		3-4		Parker, 1970
280		1.84		65		3-4		Parker, 1970
100		0.66		86		3-4		Parker, 1970
170		1.11		52		3-4		Parker, 1970
560	400	3.67	2.62	89	60	3-4		Parker, 1970
400	100			70				Oswald, 1968 b
900-1200	675	5.17-6.89	3.88	60-70		3-5	2-5	Parker et al., 1959
						8-10	30-50	Eckenfelder, 1961
220-600			0.51-1.38				15-160	Cooper, 1968
500			1.15	70		8-12	5	Oswald et al., 1967
						8-12	2 (s) 5 (w)	Malina and Rios, 1976

ALR = areal loading rate
VLR = volumetric loading rate
See p. xiv for conversion table.

An example of an approach to the design of anaerobic ponds has been presented by Oswald (1996) (Figure 3-1). In his Advanced Integrated Wastewater Pond System® (see Chapter 4), Oswald incorporates a deep anaerobic pond within a facultative pond. The anaerobic pond design is based on organic loading rates that vary with water temperature in the pond, and the design is checked by determining the volume of anaerobic pond provided per capita, which is one of the methods used for the design of separate anaerobic digesters. An example of this design approach is presented in Appendix C (Example C-3-1), along with another example using volumetric loading or detention times (see Example C-3-2) (Crites et al., 2006).

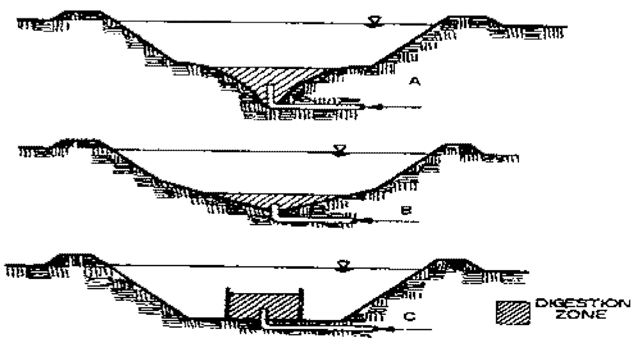


Figure 3-1. Method of creating a digestion chamber in the bottom of an anaerobic pond (Oswald, 1968).

3.3 FACULTATIVE PONDS

3.3.1 Description

The technology associated with conventional facultative ponds to treat municipal and industrial wastewater has been in widespread use in the United States for 100 years. These ponds are usually 1.2 - 2.4 m in depth and are not mechanically mixed or aerated. The layer of water near the surface contains sufficient DO from atmospheric re-aeration and photosynthetic oxygenation by microalgae growing in the photic zone to support the growth of aerobic and facultative bacteria that oxidize and stabilize wastewater organics. The bottom layer of a conventional facultative pond includes sludge deposits that are decomposed by anaerobic bacteria. These shallow ponds tend to integrate carbon and primary solids undergoing acetogenic fermentation but only intermittent methane fermentation. The intermediate anoxic layer, called the facultative zone, ranges from aerobic near the top to anaerobic at the bottom. These three strata or layers may remain stable for months due to temperature-induced water density differentials, but normally twice a year during the spring and fall seasons, conventional facultative ponds will overturn, and the three strata will mix bottom to top, top to bottom. This dimictic overturn inhibits CH_4 fermentation by O_2 intrusion into the bottom anaerobic stratum, and, as a result, C is integrated rather than being converted into biogas (Oswald et al., 1994).

The presence of algae, which release O_2 as they disassociate water molecules photochemically to assimilate hydrogen during photosynthesis, is essential to the successful performance of conventional, as well as advanced, facultative ponds. On warm, sunny days, the O_2 concentration in the aerobic zone can exceed saturation levels. As the algae take up CO_2, the *p*H of the near-surface water can exceed 10, creating conditions favorable for ammonia removal via volatilization (see Chapter 5). At night, when the algae are not photosynthesizing, O_2 levels decrease. Oxygen and *p*H levels shift together from a maximum in daylight hours to a minimum at night. The O_2 in the upper layers of the facultative pond is used by aerobic and facultative bacteria to stabilize organic material. Anaerobic fermentation, which takes place in the absence of O_2, is the dominant activity in the bottom layer of the pond. In cold climates, both oxygenation and fermentation reaction rates are significantly slower during the winter and early spring so that effluent quality may be reduced to the equivalent of primary effluent when an ice cover persists on the water surface. As a result, northern United States and Canadian provinces prohibit discharge from facultative ponds in the winter months.

3.3.2 Applicability

Conceptually, conventional facultative ponds are well suited for rural communities and industries where land costs are not a limiting factor. Conventional facultative ponds have been used to treat raw, screened, or primary settled municipal wastewater as well as higher strength biodegradable industrial wastewater. They represent a reliable and easy-to-operate process that is cost effective.

3.3.3 Advantages and Disadvantages

The advantages of facultative ponds include infrequent need for sludge removal; effective removal of settleable solids, BOD_5, pathogens, fecal coliform, and, to a limited extent, NH_3. They are easy to operate and require little energy, particularly if designed to operate with gravity flow.

The disadvantages include higher sludge accumulation in shallow ponds or in cold climates and variable seasonal NH_3 levels in the effluent. Emergent vegetation must be controlled to avoid creating breeding areas for mosquitoes and other vectors. Shallow ponds require relatively large areas. During spring and fall dimictic turnover, odors can be an intermittent problem.

3.3.4 Design Criteria

Facultative pond systems may be relatively simple mechanically, but the biological and chemical reactions taking place within them are more complex than those in conventional mechanical wastewater treatment systems. Typical design features needed to operate facultative ponds include the use of linings to control seepage to groundwater and emergent plant growth; proper design and location of inlet and outlet structures; and hydraulic controls, floating dividers, and baffles.

Many existing conventional facultative ponds are large, single-cell systems with inlets located near the center of the cell. This configuration can result in short-circuiting and ineffective use of the system design volume. A multiple-cell system with at least four cells in series, with appropriate inlet and outlet structures, is strongly recommended (Mara and Cairncross, 1989).

Most states have design criteria that specify the areal or surface organic loading rate expressed in kg/ha/d or lbs/ac/d and/or the hydraulic loading rate expressed in m/d or ft/d residence time. Typical organic loading values range from 15 - 80 kg/ha/d. Detention times range from 20 - 180 days, and can approach 200 days in northern climates where discharge restrictions prevail. Effluent $BOD_5 < 30$ mg/L can usually be achieved, while effluent TSS may range from < 30 mg/L to more than 100 mg/L, depending on the algal concentrations and discharge structure design.

A number of empirical and rational models exist for the design of simple conventional and in-series facultative ponds. These include first order plug flow, first order complete mix, and models proposed by Gloyna (1976), Marais (1961), Oswald (1968b), and Thirumurthi (1974).

All provide reasonable designs, as long as the basis for the formula is understood, appropriate parameters are selected, and the hydraulic detention and sludge retention characteristics of the system are known. This last element is of critical importance because short-circuiting in a poorly designed cell can result in detention time of 50 percent or less than the theoretical design value.

3.3.5 Design Methods

3.3.5.1 Areal Loading Rate Method

A series of detailed evaluations of facultative pond systems conducted by EPA remains a useful data set for pond systems performance in the United States (U.S. EPA, 1975). Studies of systems in other countries bring the literature up to date (Racault and Boutin, 2001; Kotsovinos et al., 2004; Oliveira and von Sperling, 2008; von Sperling and Oliveira, 2009). A comparison of the state design criteria for each location and actual design values for organic loading and HRT for four facultative pond systems evaluated by the EPA (Middlebrooks et al., 1982) are presented in Table 3-6. Many of the design flaws in the systems referenced in Table 3-6 have been corrected since 1983.

The following surface organic loading rates for various climatic conditions are recommended for use in designing facultative pond systems. For average winter air temperatures above 15 °C, a BOD_5 loading rate range of 45 - 90 kg/ha/d is recommended. When the average winter air temperature ranges between 0 - 15 °C the organic loading rate should range between 22 - 45 kg/ha/d. For average winter temperatures below 0 °C the organic loading rates should range from 11 - 22 kg/ha/d.

A review of design standards in 2006 showed that most states have design criteria for organic loading and/or HRT for facultative ponds with many now incorporating NH_4 conversion and P removal requirements. The principal changes since a survey by Canter and Englande (1970) are the nutrient removal requirements.

Table 3-6. Design and Performance Data from U.S. EPA Pond Studies (Middlebrooks et al., 1982).

Location	Organic Load (kg BOD_5/ha/d)			Theoretical Detention Time			
	State Design	Design	Actual	State Design	Design	Actual	Month 30 mg/L exceeded
P'borough[1]	39.3	19.6	16.2	None	57	107	10, 2,3,4
Kilmichael[2]	56.2	43.0	17.5	None	79	214	11, 7
Eudora[3]	38.1	38.1	18.8	None	47	231	3, 4, 8
Corinne[4]	45.0*	36.2	29.7*	180	180	70	None
			14.6**			88***	

[1]New Hampshire; [2]Mississippi; [3]Kansas; [4]Utah.
*Primary cell; ** Entire system; ***Estimated from dye study.

The BOD_5 loading rate in the first cell is usually limited to 40 kg/ha/d or less, and the total HRT in the system is 120 - 180 days in climates where the average winter air temperature is below 0

°C. In mild climates, where the winter temperature is greater than 15 °C, loadings on the primary cell can be 100 kg/ha/d (see Example C-3-3 in Appendix C).

3.3.5.2 Comparison of Facultative Pond Design Models

Because there are many possible approaches to the design of facultative ponds and given the lack of adequate performance data for the latest designs, it is not possible to recommend one approach over the others. An evaluation of the design methods presented above, with operational data referenced in Table 3-6 did not indicate that any of the models are superior to the others in predicting performance (Middlebrooks, 1987). Other reviews of facultative pond systems based on more limited data sets have reached similar conclusions (Pearson and Green, 1995). Each of the models was used to design a facultative pond for the conditions presented in Example C-3-3; the results are summarized at the end of the example (see Appendix C).

While it is difficult to make direct comparisons, an examination of the HRTs and total volume requirements calculated by all of the methods show considerable consistency if the reaction rates are selected carefully. The major limitation of all these methods is the selection of a reaction rate constant or other factors in the equations. Appropriate reaction rates must be selected, but if the pond hydraulic system is designed and constructed so that the theoretical HRT is approached, reasonable success can be assured with all of the design methods. Short-circuiting is the greatest deterrent to consistent pond performance. The importance of the hydraulic design of a pond system cannot be overemphasized.

The surface loading rate approach to design requires a minimum of input data, and is based on operational experiences in various geographical areas of the United States. It is probably the most conservative of the design methods, but the hydraulic design should be included as well.

The Gloyna loading design method achieves 80 to 90 percent BOD_5 removal efficiency, and it assumes that solar energy for photosynthesis is above the saturation level. Provisions for removal outside this range are not anticipated; however, adjustments for other solar conditions can be calculated. Mara (1975) provides a detailed critique of the method.

3.3.6 Performance

Overall, facultative pond systems are simple to operate, but may be variable in performance; BOD_5 removal can range up to 75 percent; TSS may exceed 150 mg/L; NH_3 removal can be significant (up to 90 percent) depending on temperature, *p*H and detention time in the system, except in winter; approximately 50 percent P removal can be expected under high *p*H conditions; and pathogens and coliform removal is effective, depending on temperature and detention time.

Limitations to be considered include the fact that algae in the effluent may increase TSS above the 30 mg/L limit for TSS; low temperatures and ice formation will limit process efficiency; and odors may be a problem in the spring and fall.

3.3.7 Operation and Maintenance

Most facultative ponds are designed to operate by gravity flow. The system requires less maintenance and has lower associated energy costs because pumps and other electrically powered devices may not be required. Although some analysis is essential to ensure proper operation, an extensive sampling and monitoring program is usually not necessary. Regular observation of impoundment earth works must be performed to monitor for excavation by burrowing animals. See Chapter 9 for more details on operation and maintenance.

3.3.8 Common Modifications

A common modification to facultative ponds is to operate them in the controlled discharge mode, where discharge is prohibited during the winter months in cold climates and/or during peak algal growth periods in the summer. In this approach, each cell in the system is isolated and discharged sequentially. A similar modification, the hydrograph controlled release (HCR), retains treated wastewater in the pond until flow volume and conditions in the receiving stream are adequate for discharge. A recently developed physical modification uses plastic curtains, supported by floats and anchored to the bottom, to divide ponds into multiple cells and/or to serve as baffles to improve hydraulic conditions. Another modification uses a floating plastic grid to support the growth of duckweed (*Lemna* spp.) on the surface of the final cell in the pond system, which restricts light penetration and reduces algal growth (with sufficient detention time, >20 d), improving the final effluent quality. These types of modifications are discussed in detail in Chapter 7.

3.3.9 Costs

Cost information for facultative ponds varies significantly. Construction costs include land purchase, excavation, grading, berm construction, and inlet and outlet structures. If the soil is permeable, an additional cost for lining should be considered. See Chapter 8 for discussion of costs associated with construction of pond systems.

3.4 AERATED POND SYSTEMS

3.4.1 Partial Mix Aerated Ponds

Aerobic ponds are classified by the amount and source of O_2 supplied. In aerated systems, O_2 is supplied mainly through mechanical or diffused aeration rather than by algal photosynthesis. The submerged systems can include perforated tubing or piping, with a variety of diffusers attached. A partial mix system provides only enough aeration to satisfy the O_2 requirements of the system. It does not provide energy to keep all solids in suspension. In some cases, the initial cell in a system might be a complete mix unit followed by partial mix and settling cells. A complete mix system requires about 10 times the amount of energy needed for a similarly sized partial mix system.

Some solids in partial mix ponds are kept in suspension to contribute to overall treatment. This allows for anaerobic fermentation of the settled sludges. Partial mix ponds are also called facultative aerated ponds and are generally designed with at least three cells in series; total detention time depends on water temperature. The ponds are constructed to have a water depth of up to 6 m to ensure maximum O_2 transfer efficiency. In most systems, aeration is not applied uniformly over the entire system.

Typically, the most intense aeration (up to 50 percent of the total required) is used in the first cell. The final cell may have little or no aeration to allow settling to occur. In some cases, a small separate settling pond is provided after the final cell. Diffused aeration equipment typically provides about 3.7 - 4 kg O_2/kW/hr and mechanical surface aerators are rated at 1.5 - 2.1 kg O_2/kW/hr. Consequently, diffused systems are somewhat more efficient than non-aerated ponds, but also require a significantly greater installation and maintenance effort.

Aerobic ponds can reliably produce an effluent to achieve BOD_5 and TSS < 30 mg/L if a settling pond is in place at the end of the system. Additionally, significant nitrification will occur during the summer if there is adequate DO. Many systems designed only for BOD_5 removal fail to meet discharge standards during the summer because of a shortage of DO. Both nitrification of NH_3 and BOD_5 removal require O_2. To achieve regulatory limits for the two parameters in heavily loaded systems, pond volume and aeration capacity beyond that provided for BOD_5 removal alone are required. It is generally assumed that 1.5 kg of O_2 will treat 1 kg of BOD_5. About 5 kg of O_2 are theoretically required to convert 1 kg of NH_3 to NO_3^-.

3.4.1.1 Applicability

Aerated ponds are well suited for small communities and industries and require less land. They are usually designed with a shorter retention time. They have been used to treat raw, screened or primary settled municipal wastewater, as well as higher strength biodegradable industrial wastewater. The process is reliable, relatively easy to operate and cost effective.

3.4.1.2 Advantages and Disadvantages

The advantages include reliable BOD_5 removal; significant nitrification of NH_3 possible with sufficient mean cell resident time; treatment of influent with higher BOD_5 in less space; and reduced potential for unpleasant odors.

Aerated ponds are more complicated to design and construct, which increases capital and O&M costs. A larger staff is needed for whom training must be provided on a regular basis. Finally, sludge removal is more frequent and requires secondary treatment for disposal off-site.

3.4.1.3 Design Methods

The basic approach to the design of partial mix aerobic ponds has not changed since the early 1980's. The most notable innovations have been the placement of floating plastic partitions in the ponds to improve the hydraulic characteristics and the development of a wider selection of more efficient aeration equipment (Water Environment Federation, 2001). Given the importance of the hydraulic characteristics, retaining redundancy in the design of aerobic pond systems is still strongly encouraged. Operation and maintenance costs associated with aerobic pond systems often are not included when communities compare system options. The initial cost of a system built without redundancy is lower in the short term. Systems that include flexibility in operation in the long run, however, greatly reduce the actual cost to the environment and the owner.

In partial mix systems, the aeration serves to provide only an adequate O_2 supply, and there is no attempt to keep all of the solids in suspension. Although some of the solids are suspended, anaerobic degradation of the organic matter that settles does occur.

3.4.1.4 Partial Mix Design Model

Although the pond is partially mixed, it is conventional to estimate the BOD_5 removal using a complete mix model and first order reaction kinetics. Studies by Middlebrooks et al. (1982) have shown that a plug flow model and first order kinetics more closely predict the performance of these ponds when either surface or diffused aeration is used. However, most of the ponds evaluated in this study were lightly loaded and the calculated reaction rates are very conservative, as it seems that the rate decreases as the organic loading decreases (Neel et al., 1961). Without additional data to support theoretical design reaction rates, it is necessary to design partial mix ponds using complete mix kinetics.

The design model using first order kinetics and operating *n* number of equal sized cells in series is given by Equation 3-3 (Middlebrooks et al. 1982; 10 States Standards, 2004; Water Environment Federation, 2001; Crites et al., 2006).

$$\frac{C_e}{C_0} = \frac{1}{[1 + (kt/n)]^n} \qquad (3\text{-}3)$$

Where:

C_n = effluent BOD_5 concentration in cell *n*, mg/L
C_o = influent BOD_5 concentration, mg/L
k = first order reaction rate constant /d
= 0.276 day^{-1} at 20° C (assumed to be constant in all cells)
t = total hydraulic residence time in pond system, d
n = number of cells in the series

If other than a series of equal volume ponds are to be employed and varying reaction rates are expected, the following general equation should be used:

$$\frac{C_n}{C_0} = \left(\frac{1}{1+k_1t_1}\right)\left(\frac{1}{1+k_2t_2}\right)\cdots\left(\frac{1}{1+k_nt_n}\right) \qquad (3\text{-}4)$$

where $k_1, k_2,...k_n$ are the reaction rates in cells 1 through *n* (all usually assumed to be equal without additional data) and $t_1, t_2,...t_n$ are the hydraulic residence times in the respective cells.

Mara (1975) has shown that a number of equal volume reactors in series is more efficient than unequal volumes; however, due to site topography or other factors, there may be sites where it is necessary to construct cells of unequal volume.

3.4.1.5 Temperature Effects

The influence of temperature on the reaction rate is defined by Equation 3-5.

$$k_T = k_{20}\theta^{T_w-20} \qquad (3\text{-}5)$$

Where:

k_T = reaction rate at temperature T/d
k_{20} = reaction rate at 20° C/d

θ = temperature coefficient = 1.036
T_w = temperature of pond water, °C

The pond water temperature (T_w) can be estimated using the following equation developed by Mancini and Barnhart (1976).

$$T_w = \frac{AfT + QT_I}{Af + Q} \qquad (3\text{-}6)$$

Where:

T_w = pond water temperature, °C
T_a = ambient air temperature, °C
A = surface area of pond, m^2
f = proportionality factor = 0.5
Q = wastewater flow rate, m^3/d

An estimate of the surface area is made based on Equation 3-4, corrected for temperature, and the temperature is calculated using Equation 3-6. After several iterations, when the water temperature used to correct the reaction rate coefficient agrees with the value calculated with Equation 3-6, the detention time in the system can be determined.

3.4.1.6 Selection of Reaction Rate Constants

The selection of a k value is the critical decision in the design of any pond system. A design value of 0.12 /d at 20 °C and 0.06/d at 1 °C is recommended by the 10 States Standards (2004). Studies of systems in Texas have empirically derived the value of the temperature coefficient, θ, for soluble organic removal in complete mix ponds to be 1.03-1.04 (Wang and Pereira, 1986.)

3.4.1.7 Influence of Number of Cells

When using the partial mix design model, the number of cells in series has a pronounced effect on the size of the pond system required to achieve the specified degree of treatment. The effect can be demonstrated by rearranging Equation 3-1 and solving for t:

$$t = \frac{n}{k}\left[\left(\frac{C_o}{C_n}\right)^{\frac{1}{n}} - 1\right] \qquad (3\text{-}7)$$

All terms in this equation have been defined previously.

3.4.1.8 Pond Configuration

The ideal configuration of a pond designed on the basis of complete mix hydraulics is a circular or square pond. However, even though partial mix ponds are designed using the complete mix model, it is recommended that the cells be configured with a length-to-width ratio of 3:1 or 4:1. This is because it is recognized that the hydraulic flow pattern in partial mix systems more closely resembles the plug flow condition. The dimensions of the cells can be calculated using Equation 3-8.

$$V = [LW + (L - 2sd)(W - 2sd) + 4(L - sd)(W - sd)]\frac{d}{6} \qquad (3\text{-}8)$$

Where:

V = volume of pond or cell, m^3
L = length of pond or cell at water surface, m
W = width of pond or cell at water surface, m
s = slope factor (e.g., with 3:1 slope, s = 3)
d = depth of pond, m

3.4.1.9 Mixing and Aeration

The O_2 requirements control the energy input required for partial mix pond systems. There are several rational equations available to estimate the O_2 requirements for pond systems; these can be found in Benefield and Randall (1980), Gloyna (1976, 1971), and Metcalf and Eddy (1991, 2003). In most cases, partial mix system design is based on the strength of the BOD_5 entering the system. After calculating the required rate of O_2 transfer, information contained in equipment manufacturers' catalogs should be consulted to determine the zone of complete O_2 dispersion by surface, helical, or air gun aerators or the proper spacing of perforated tubing. Schematic sketches of several of the various types of aerators used in pond systems are shown in Figure 3-2A and B. Photographs of installed aeration equipment are shown in Figure 3-3.

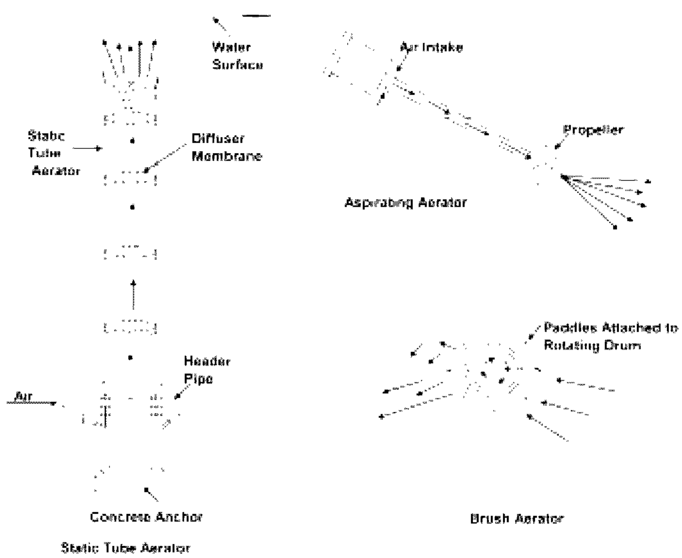


Figure 3-2A. Static Tube, Brush and Aspirating Aerators. (Reynolds and Richards, 1996).

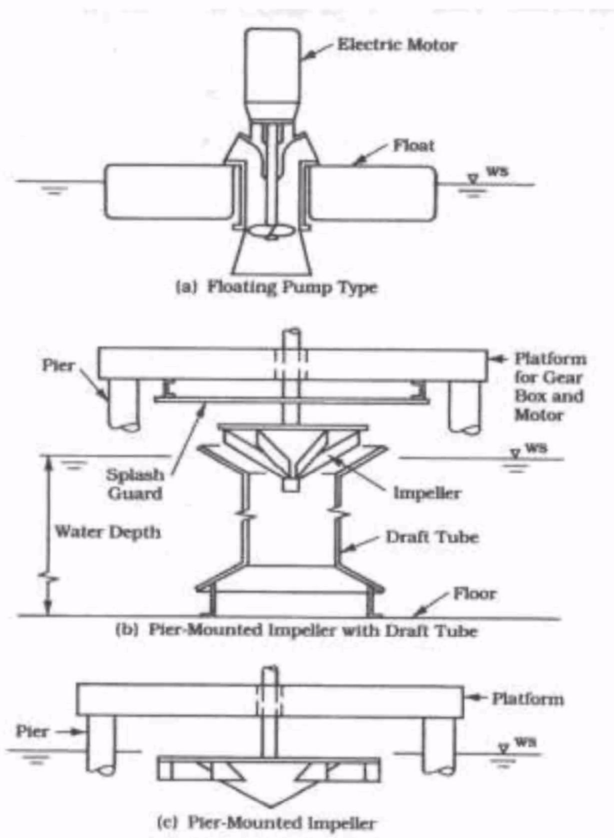


Figure 3-2B. Floating pump, pier-mounted impeller with draft tube and pier-mounted impeller (Reynolds and Richards, 1996).

See Appendix C for mixer and diffuser design calculations.

Surface aeration equipment is subject to potential icing problems in cold climates, but there are many options available to avoid this problem (see Figure 3.3 and Chapter 4). Improvements have been made in fine bubble perforated tubing, but a diligent maintenance program is still the best policy. In the past, a number of systems experienced clogging of the perforations, particularly in hard water areas, and corrective action required purging with *HCl* gas.

Figure 3-3. Floating aerators in summer and winter operation.

The final element recommended in this partial mix aerobic pond system is a settling cell with a 2 d HRT at the average design flow rate.

3.4.1.10 Performance

Reliable BOD_5 removal up to 95 percent can be expected. Effluent TSS can range from 20 to 60 mg/L, depending on the design of the settling basin and the concentration of algae in the effluent. Removal of NH_3 is less effective due to shorter detention times, but nitrification of NH_3 can occur in aerated ponds if the system is designed for that purpose. Phosphorus removal is only 15 - 25 percent. Removal of total and fecal coliform depends on length of detention time and temperature. If effluent limits are < 200 MPN/100 mL, disinfection may be needed.

3.4.1.11 Limitations and Operation and Maintenance

Depending upon the rate of aeration and the environment, ice may form on the surface of aerobic ponds during cold weather. Rates of biological activity slow down during cold weather. If properly designed, a system will continue to function and produce acceptable effluents under these conditions. The potential for ice formation on floating aerators may encourage the use of submerged diffused aeration in very cold climates.

The use of submerged perforated tubing for diffused aeration requires maintenance and cleaning on a routine basis to maintain design rates. There are numerous types of submerged aeration equipment that can be used in warm or cold climates, and these should be considered for all designs. In submerged diffused aeration, the routine application of hydrochloric acid (HCl) gas in the system is used to dissolve accumulated material on the diffuser units. Any earthen structures used as impoundments must be periodically inspected. Typically, operation occurs by gravity flow. Energy is required for the aeration devices, the amount depending on the intensity of mixing desired. Partial mix systems require between 1 - 2 W/m^3 capacity, depending on the depth and configuration of the system. See design example C-3-7 in Appendix C for a method of calculating the energy requirements for partial mix systems.

3.4.1.12 Modifications

One physical modification to an aerobic pond is the use of plastic curtains supported by floats and anchored to the bottom to divide existing ponds into multiple cells and/or serve as baffles to improve hydraulic conditions. A recently developed approach suspends a row of submerged diffusers from flexible floating booms, which move in a cyclic pattern during aeration activity. This treats a larger volume with each aeration line. Effluent is periodically recycled within the system to improve performance. If there is sufficient depth for effective O_2 transfer, aeration is used to upgrade existing facultative ponds and is sometimes used on a seasonal basis during periods of peak wastewater discharge (e.g., seasonal food processing wastes) to the pond.

3.4.1.13 Costs

Construction costs associated with partially mixed aerobic ponds include cost of the land, excavation, and inlet and outlet structures. If the soil where the system is constructed is permeable, there will be an additional cost for lining. Excavation costs vary, depending on whether soil must be added or removed. Operating costs of partial mix ponds include operation and maintenance of surface or diffused aeration equipment.

3.4.2 Complete Mix Aerated Ponds (Subset of Aerated Pond System)

Complete mix systems rely on mechanical aeration to introduce enough O_2 to completely degrade all BOD_5. In addition to that, however, the additional mixing suspends the solid material to enhance biodegradation.

3.4.2.1 Applicability

See Section 3.4.1.1.

3.4.2.2 Advantages/Disadvantages

See Section 3.4.1.2.

3.4.2.3 Design models and example calculations

Complete mix ponds are smaller than partial mix ponds and all solids in the aeration cell are kept in suspension. The system is designed using first order kinetics and a complete mix model. Most states specify the formulation shown in Equation 3-7 and used in the design example to size the aeration cell and specify the size of the settling cell. Typically a plastic, clay or other impervious lining is required to protect groundwater. A multiple cell system with at least three cells in series is recommended, with appropriate inlet and outlet structures to maximize effectiveness of the design volume. Hydraulic residence times are generally are less than 3 d except where high strength wastewaters are treated. An HRT range of 2 - 4 d is recommended so that the microbial community has sufficient time to grow (von Sperling and de Lemos Chernicharo, 2005).

3.4.2.4 Design Equation

The design model using first order kinetics and operating *n* number of equal sized cells in series is given in Section 3.4 by Equation 3-3 and if a series of non-equal volume ponds or ponds with varying reaction rates are to be designed, use Equation 3-4.

3.4.2.5 Temperature Effects

See Section 3.4.1.5.

3.4.2.6 Selection of Reaction Rate Parameters

See Section 3.4.1.6.

3.4.2.7 Influence of Number of Cells

See Section 3.4.1.7. An example (C-3-4) can be found in Appendix C.

3.4.2.8 Pond Configuration

The ideal configuration of a pond designed on the basis of complete mix hydraulics is a circular or a square pond; however, it is recommended that the cells be configured with a length to width ratio of 3:1 or 4:1 because the hydraulic flow pattern in complete mix systems actually more closely resembles the plug flow model. The dimensions of the cells can be calculated using Equation 3-8 in Section 3.4.1.8.

3.4.2.9 Mixing and Aeration

The mixing requirements usually control the energy input required for complete mix pond systems. There are several rational equations available to estimate the O_2 requirements for pond systems (see Section 3.4.1.9 for references). Complete mix systems are designed by estimating the strength of the BOD_5 entering the system and then calculated to ensure that adequate energy is available to provide complete mixing. Once the required rate of O_2 transfer is known, the equipment manufacturers' catalogs should be consulted to determine the zone of complete mixing and O_2 dispersion. The aerators used in complete mix systems are the same as those used in partial mix systems.

Equation 3-9 is used to estimate O_2 transfer rates.

$$N = \frac{N_a}{\alpha\left[\frac{(C_{sw} - C_L)}{C_s}\right](1.025)^{(T_W - 20)}} \qquad (3\text{-}9)$$

where

N = equivalent O_2 transfer to tap water at standard conditions, kg/hr
N_a = O_2 required to treat the wastewater, kg/hr (usually taken as 1.5 x the organic loading entering the cell)
a = (O_2 transfer in wastewater)/(O_2 transfer in tap water) = 0.9
C_L = minimum DO concentration to be maintained in the wastewater, assume 2 mg/L
C_s = O_2 saturation value of tap water at 20 °C and one atmosphere pressure = 9.17 mg/L
T_w = wastewater temperature, °C
C_{sw} = β(C_{ss})P = O_2 saturation value of the waste, mg/L
β = (wastewater saturation value)/(tap water O_2 saturation value)
C_{ss} = tap water O_2 saturation value at temperature T_W
P = ratio of barometric pressure at the pond site to barometric pressure at sea level, assume 1.0 for an elevation of 100 m

Equation 3-6 can be used to estimate the water temperature in the pond during the summer months, which is the most active period of biological activity. However, as energy to provide complete mixing is assumed to be available, DO should be at adequate levels throughout the year. The complete mix design procedure is illustrated in Example 3-5 found in Appendix C. The four-cell system can be simulated by using floating plastic partitions (see Chapter 4).

3.4.2.10 Performance
See Section 3.4.1.10.

3.4.2.11 Modifications
There are many configurations of complete mix pond systems. Examples that will be discussed in Chapter 4 include the High-Performance Aerated Pond Systems and the BIOLAC® Process.

An examination of Example C-3-3 (Appendix C) will show the similarity between the design for the High-Performance Aerated Pond System and the complete mix design when the final three cells of the complete mix design are supplied with only enough DO to meet the BOD_5. This is not to imply that the designs are identical, but only to point out that they have some common features.

3.4.2.12 Costs
See Section 3.4.1.12.

CHAPTER 4

PHYSICAL DESIGN AND CONSTRUCTION

4.1 INTRODUCTION

No matter how carefully coefficients are evaluated and biological or kinetic models reviewed, if sufficient consideration is not given to optimization of the pond layout and construction, the actual efficiency of the system may be far less than the calculated efficiency. The biological factors affecting wastewater pond performance must be understood so that a reasonable estimate of the hydraulic residence time required to achieve a specified efficiency is incorporated into the design. But it is the physical factors, such as length to width ratio, placement of inlet and outlet structures and redundancy in design that determine the actual treatment efficiency that can be achieved (Crites et al., 2006; Shilton, 2005).

The danger of groundwater contamination frequently imposes seepage restrictions, necessitating lining or sealing the pond. Reuse of the pond effluent in dry areas where all water losses are to be avoided may also dictate the use of linings. Layout and construction criteria should be established to reduce dike erosion from wave action, weather and burrowing animals. Transfer structure placement and size affect flow patterns within the pond and determine operational ability to control the water level and discharge rate. These important physical design considerations are discussed in the following sections.

4.2 DIKE CONSTRUCTION

Dike stability is most often affected by erosion caused by wind-driven wave action or rain and rain-induced weathering. Dikes may also be destroyed by burrowing animals. A good design with proper maintenance, will anticipate these problems and provide a stable, reliable system.

4.2.1 Wave Protection

Erosion protection should be provided on all slopes; however, if winds are predominantly from one direction, protection should be enhanced for those areas that receive the full force of the wind-driven waves. Protection should always extend from at least 0.3 m below the minimum water surface to at least 0.3 m above the maximum water surface (U.S. EPA, 1977b; Kays, 1986; U.S. Department of Interior [USDI], 2001). Wave height is a function of wind velocity and fetch (the distance over which the wind acts on the water). The size of riprap needed depends on the fetch length (Uhte, 1974; Kays, 1986). Riprap varies from river run rocks that are 15 - 20 cm to quarry boulders that are 7 - 14 kg. Uniformly graded river run material, when used for riprap, can be quite unstable. River run rocks, if not properly mixed with smaller material and carefully placed, can be loosened by wave action and slip down the steep sloped dikes. Broken concrete pavement can often be used for riprap but can make mechanical weed control difficult. Asphalt, concrete, fabric, and low grasses can also be used to provide protection from wave action. When riprap is used for wave protection, the designer must take into consideration its effect on weed and animal control and routine dike maintenance.

4.2.2 Weather Protection

Dike slopes must be protected from weather induced erosion as much as from wave erosion in many areas of the country. The most common method of weather erosion protection employs grass when large dike areas are involved. Because variations in depth develop in total containment ponds, they often have large sloped dike areas that cannot be protected in a more cost-effective way. Ponds that have only minimum freeboard and constant water depth may be protected more cost-effectively if the riprap is carried right to the top of the slope where it can serve as wave and weather protection.

In some cases climate and soil conditions are suitable for completely bare dike slopes without major weather erosion problems. Figure 4-1 shows the erosion effects on the bare slopes of a treatment pond.

Weather caused erosion, unlike wave erosion, can also affect the top and outside slopes of the pond diking system. The designer should make sure that the all-weather road system for the top of the dike is of sufficient width to allow traffic to pass over every part of the surface. Too narrow a road will result in ruts that can create runoff erosion problems in areas of high rain intensity. Final grading should be specified to minimize rutting and frequent maintenance should be required to control surface runoff and erosion.

It is also necessary to protect the exterior surface of dikes. A thin layer of gravel may be used; placement of topsoil and seeding for native groundcover is recommended. Local highway department experience on erosion control for cut-and-fill slopes should be used as a guide.

Figure 4-1 An example of eroded dike slopes.

4.2.3 Animal Protection

If a treatment pond is located in an area that supports burrowing animals, such as muskrats and nutria, design elements can be put in place to control this threat to dike stability. Broken concrete or other riprap that does not completely cover the dike soil can become a home for burrowing animals. Riprap design and placement should emphasize limiting the creation of voids that allow them to burrow near the water surface (Crites et al., 2006).

Varying pond water depth can discourage muskrat infestation (U.S.EPA, 1977b; Crites et al., 2006). Muskrats prefer a partially submerged tunnel, so design provisions to vary the water level over a several-week period will discourage them from burrowing in the dike. Such provisions will often add to the expense of riprap placement for wave protection, but can greatly reduce operation and maintenance expenses.

Figure 4.2 Evidence of burrowing at the edge of a treatment pond (Mayo et al., 2010).

4.2.4 Seepage

Dikes should be designed and constructed to minimize seepage. Vegetation and porous soils should be removed and the embankment should be well compacted. Use of conventional construction equipment is usually suitable for this purpose.

Seepage collars should be provided around any pipes penetrating the dike (Kays, 1986; Thomas et al., 1966). The seepage collars should extend a minimum of 0.6 m from the pipe. Proper installation of transfer pipes can be assured by building up the dike above the pipe elevation, digging a trench for the pipe and seepage collar, backfilling the trench, and compacting the backfill.

In some circumstances it may be necessary to control seepage and ensure bank stability at the exterior toe. A filter blanket material can be used (Middlebrooks, et al. 1978; Kays, 1986). Another method of preventing seepage where embankment material cannot be adequately compacted is placement of an impervious core in the levee with imported material.

4.3 POND SEALING

4.3.1 Introduction

The need for a well-sealed treatment pond has impacted modern pond design, construction, and maintenance, and sealing is required in most design situations. The primary motive for sealing ponds is to prevent seepage. Seepage affects treatment capabilities by causing fluctuation in the water depth and can cause pollution of groundwater. Although many types of pond sealers exist, they can be classified into three major categories: (1) synthetic and rubber liners, (2) earthen and cement liners, and (3) natural and chemical treatment sealers. Within each category there exists a wide variety of application characteristics. Choosing the appropriate lining for a specific site is a critical issue in pond design and seepage control. Detailed information is available from other publications (Kays, 1986; Middlebrooks et al., 1978; USDA, 1997; USDI, 2001, Koerner and Koerner, 2009).

4.3.2 Seepage Rates

Most regulatory agencies limit the amount of seepage from ponds, so it may be important to be able to estimate seepage rates. Stander et al. (1970) presented a summary of information (Table

4.1) on measured seepage rates in wastewater treatment ponds. Seepage rates in irrigation channels can be found in U.S. DI (1991). Seepage is a function of a number of variables; it is difficult to anticipate or predict rates even with extensive soil tests. Careful evaluations must be conducted along with a review of manufacturers' information to determine whether a lining is required and which type. This should be done before the ponds are constructed.

The Minnesota Pollution Control Agency (Hannaman et al., 1978) initiated an intensive study to evaluate the effects of treatment pond seepage from five municipal systems. The five communities were selected for study on the basis of geologic setting, age of the system, and past operating history of the pond. The selected ponds were representative of the major geomorphic regions in the state, and the age of the systems ranged from 3 to 17 years.

Estimates of seepage were calculated by two independent methods for each of the five pond systems. Water balances were calculated by taking the difference between the recorded inflows and outflows, and pond seepage was determined by conducting in-place field permeability tests of the bottom soils at each location. Good correlation was obtained with both techniques.

Table 4-1. Reported Seepage Rates From Pond Systems (from Stander et al., 1970)[a].

Location	Pond Base	Initial Seepage Rate cm/d ($m^3/m^2/d$)	Hydraulic Load $m^3/m^2/d$	Seepage Rate as percent of Hydraulic Load	Settling-in Period	Eventual Seepage Rate cm/d ($m^3/m^2/d$)	Hydraulic Load $m^3/m^2/d$	Seepage Rate as percent of Hydraulic Load
Mojave[1]	Desert soil (sandy soil)	22.4 (0.19)	0.30	63	9 mo	0.9 (0.007)	0.36	2
Kearney[2b]	Sand and gravel	14.0 (0.12)	0.13	90	1 yr	1.5 (0.013)	0.04	29
Filer City[3]	Sandy soil				Average over 5 yr	0.9 (0.007)	0.009	84
Pretoria[4c]	Clay loam and shale	(0.13)	0.05	Exceeded inflow rate	Approx. 1 yr	0.8 (0.006)	0.05	13

[1]California; [2]Nebraska; [3]Michigan; [4]South Africa
[a]Courtesy of Ann Arbor Science Publishers, Inc., Ann Arbor, MI.
[b]Evaporation and rainfall effects apparently not corrected for. Seepage losses also influenced at times by a high water table.
[c]Constructed in sandy soil for the express purpose of seeping away Paper Mill NSSC liquor.

Field permeability tests indicated that the additional sealing from the sludge blanket was insignificant in locations where impermeable soils were used in the construction process. In the case of more permeable soils, it appeared that the sludge blanket reduced the permeability of the bottom soil from an initial level of 10^{-4} or 10^{-5} cm/sec to the order of 10^{-6} cm/sec. At all five systems evaluated, the treatment pond was in contact with the local groundwater table. Local groundwater fluctuations had a significant impact on seepage rates. Reducing the groundwater gradient resulted in a reduction of seepage losses at three of the sites. Contact with groundwater possibly explains the reduction in seepage rates in many ponds; in the past this reduction in seepage rates has been attributed totally to a sludge buildup. (Stander, et al.)

In an area underlain by permeable material where little groundwater mounding occurs, there is probably little influence from the water table on seepage rates. The buildup of sludge on the bottom of a pond appears to improve the quality of the seepage water leaving the pond. Sludge accumulation apparently increases the cation exchange capacity of the bottom of the pond. Groundwater samples obtained from monitoring wells did not show any appreciable increases in N, P, or fecal coliform over the background levels after 17 years of operation. The seepage from the ponds did show an increase in soluble salts as great as 20 times over background levels. Concentrations of 25 mg/L to 527 mg/L of chloride were observed.

A comparison of observed seepage rates for various types of liner material is presented in Table 4.2 (Kays, 1986). If an impermeable liner is required, one of the synthetic materials must be used. The East Bay Municipal Utility District, Oakland California, developed the following formula for leakage tolerance, which can be modified by inserting more stringent factors in the denominator, e.g., 100, 200 and so forth. The equation is empirical and its use must be based on experience:

$$Q = \frac{A\sqrt{H}}{80} \qquad (4\text{-}1)$$

Where:

Q = maximum permissible leakage tolerance, L/min
A = lining area, m^2
H = maximum water depth, m

4.3.3 Natural and Chemical Treatment Sealing

The most complex techniques of pond sealing, either separately or in combination, are natural pond sealing and chemical treatment sealing (Thomas et al., 1966; Bhagat and Proctor, 1969; Seepage Control, Inc., 2005).

Natural sealing of ponds occurs via three mechanisms: (1) physical clogging of soil pores by settled solids; (2) chemical clogging of soil pores by ionic exchange; and (3) biological and organic clogging caused by microbial growth at the pond lining. Which mechanism should be used depends on the characteristics of the wastewater being treated.

Infiltration characteristics of anaerobic ponds were studied in New Zealand (Hill, 1976). Certain soil additives were employed (bentonite, sodium carbonate, sodium triphosphate) in 12 pilot ponds with varying water depth, soil type and compacted bottom soil thickness. It was found that chemical sealing was effective for soils with a minimum clay content of 8 percent and a silt content of 10 percent. Effectiveness increased with clay and silt content.

Four different soil columns were placed at the bottom of an animal wastewater pond to study physical and chemical properties of soil and sealing of ponds (Chang et al., 1974; U.S. DA, 1972). It was discovered that the initial sealing which occurred in the top 5 cm of the soil columns was caused by the trapping of suspended matter in the soil pores. This was followed by a secondary mechanism of microbial growth that completely sealed off the soil from water intrusion.

A similar study performed in Arizona (Wilson et al., 1973) also found this double mechanism of physical and biological sealing. Physical sealing of the pond was enhanced by the use of an organic polymer mixed with bentonite clay. This additive could have been applied with the pond full or empty, although it was more effective when the pond was empty.

Table 4-2. Seepage Rates for Various Liners[a] (Kays, 1986)[b].

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

NA = not available
[a] The data are based on actual installation experience. The chemical and bentonite treatments

depend on seepage rates, and in the table loose earth values are assumed.
[b]Courtesy of John Wiley & Sons, Inc., New York, NY.

An experiment was performed in South Dakota (Matthew and Harms, 1969) in an effort to relate the sodium adsorption ratio (SAR) of the in situ soil to the sealing mechanism of treatment ponds. The general observation was made that the equilibrium permeability ratio decreases by a factor of 10 as SAR varies from 10 to 80. Polymeric sealants have been used to seal both filled and unfilled ponds (Rosene and Parks, 1973; Seepage Control, Inc., 2005). Unfilled ponds have been sealed by admixing a blend of bentonite and the polymer directly into the soil lining. Filled ponds have been sealed by spraying the fluid surface with alternate slurries of the polymer and bentonite. It has been recommended that the spraying take place in three subsequent layers: (1) polymer, (2) bentonite, and (3) polymer. The efficiency of the sealant has been found to be significantly affected by the characteristics of the impounded water. Most importantly, calcium ions in the water exchange with sodium ions in the bentonite and cause failure of the compacted bentonite linings.

Davis et al., (1973) found that for liquid dairy waste, the biological clogging mechanism predominated. In a San Diego County study site located on sandy loam, the infiltration rate of a virgin pond was measured. A clean water infiltration rate for the pond was 122 cm/d. After two weeks of manure water addition, infiltration averaged 5.8 cm/d; after four months, 0.5 cm/d.

A study performed in southern California (Robinson, 1973) showed similar results. After waste material was placed in the unlined pond in an alluvial silty soil, the seepage rate was reduced. The initial 11.2 cm/d seepage rate dropped to 0.56 cm/d after three months, and to 0.30 cm/d after six months.

4.3.4 Design and Construction Practice

4.3.4.1 Lining Materials

Information about current commercial sources of lining materials is available elsewhere (see Section 4.3.1). Design and construction methods are available from these sources. A general presentation of recommended pond sealing design and construction procedures is presented below. The methods are divided into two categories: (1) bentonite, asphalt, and soil cement liners, and (2) thin membrane liners. Although there are major differences between the two application techniques, there are some similarities between the application of asphalt panels and elastomer liners.

Regardless of the difference between the type of material selected, there are many common design, specification, and construction practices. A summary of the effective design practices in cut-and-fill reservoirs is given below. Most of these practices are common sense observations, yet experience shows that these practices are very often ignored.

Summary of Effective Design Practices for Placing Linings in Cut-and-Fill Reservoirs

- Lining must be placed on a stable soil foundation or structure.

- Facility design and inspection should be the responsibility of professionals with backgrounds in liner applications and experience in geotechnical engineering.

- A continuous underdrain of perforated piping or other configuration to collect groundwater below the lining that operates at atmospheric pressure should be put in place.
- A leakage tolerance should be included in the specifications.
- Continuous, thin, impermeable-type linings should be placed on a smooth surface of concrete, earth, gunite, or asphalt concrete.
- Except for asphalt panels, all field joints should be made perpendicular to the toe of the slope. Some materials can run in any direction, but generally joints run perpendicular to the toe of the slope.
- Formal or informal anchors may be used at the top of the slope.
- Inlet and outlet structures must be sealed properly.
- All lining punctures and cracks in the support structure should be sealed.
- Emergency discharge quick-release devices should be provided in large reservoirs.
- Wind problems with exposed thin membrane liners can be controlled by installing vents so that they are built into the lining.
- Adequate protective fencing must be installed to control vandalism.

Bentonite, Asphalt, and Soil Cement
The application of bentonite, asphalt, and soil cement as lining materials for reservoirs and wastewater ponds has a long history (Kays, 1986). The following summary includes consideration of the materials, costs, evaluations of durability, and effectiveness in limiting seepage. The cost analysis is somewhat arbitrary, since it depends primarily on the availability of the materials. Most states have developed standards relating to the application of these types of materials, and detailed discussions of these materials are presented elsewhere (Middlebrooks et al., 1978; Koerner and Koerner, 2009).

Bentonite
Bentonite is a sodium montmorillonite clay that exhibits a high degree of swelling, imperviousness, and low hydraulic conductivity. The variety of ways in which bentonite may be used to line ponds are listed below:

- A suspension of bentonite in water (with a bentonite concentration of approximately 0.5 percent of the water weight) is placed over the area to be lined. The bentonite settles to the soil surface, forming a thin blanket.
- The same procedure as above, except frequent harrowing of the surface produces a uniform soil-bentonite mixture on the surface of the soil. The amount of bentonite used in this procedure is approximately 4.5 kg/m^2.
- A gravel bed approximately 15 cm deep is first prepared and the bentonite application performed as in the first method. The bentonite will settle through the gravel layer and seal the void spaces.
- Bentonite is spread as a membrane 2.5 - 5 cm thick and covered with a 20 - 30 cm

blanket of soil and gravel to protect the membrane. A mixture of soil and gravel is more satisfactory than soil alone, because the stability is increased and there is greater resistance to erosion.
- Bentonite is mixed with a sand ratio of approximately 1:8. A layer 5 - 10 cm in thickness is placed on the reservoir bottom and covered with a protective cover of sand or soil. This method takes about 13.5 kg/m of bentonite.

In the last two methods listed above, the following construction practices are recommended:
- The section must be over-excavated (30 cm) with drag lines or graders.
- Side slopes should not be steeper than two horizontal to one vertical.
- The sub-grade surface should be dragged to remove large rocks and sharp angles. Usually two passes with adequate equipment are sufficient to smooth the sub-grade.
- Sub-grade should be rolled with a smooth steel roller.
- The sub-grade should be sprinkled to eliminate dust problems. The bentonite or soil-bentonite membrane should then be applied.
- The protective cover should contain sand and small gravel, in addition to cohesive, fine grained material, so that it will be erosion resistant and stable.

The performance of bentonite linings is greatly affected by the quality of the bentonite. Some natural bentonite deposits may contain quantities of sand, silt and clay impurities. Poor quality bentonite deteriorates rapidly in the presence of hard water, and tends to erode in the presence of currents or waves. Bentonite linings must often be put in place manually, which can add considerably to the cost. Wyoming-type bentonite, which is a high-swelling sodium montmorillonite clay, has been found to be satisfactory.

Fine-ground bentonite is generally more suitable for the lining than pit-run bentonite. If the bentonite is finer than a No. 30 sieve, it may be used without specifying size gradation. But if the material is coarser than the No. 30 sieve, it should be well graded. Bentonite should contain a moisture content of less than 20 percent. This is especially important for thin membranes. Some disturbance, and possibly cracking of the membranes, may take place during the first year after construction due to settling of the sub-grade upon saturation. A proper maintenance program, especially at the end of the first year, is a necessity.

Sodium bentonite linings may be effective if they have an adequate amount of exchangeable sodium. Deterioration of the linings has been observed to occur in cases where magnesium or calcium has replaced sodium as the adsorbed ions. A thin layer, less than 15 cm, of bentonite on the soil surface tends to crack if allowed to dry. Because of this, a bentonite soil mixture with a cover of fine grained soil on top, or a thicker bentonite layer, is recommended (Dedrick, 1975). Surface bentonite cannot be expected to be effective longer than two to four years. A buried bentonite blanket may last from 8 to 15 years (U.S. EPA, 1978; U.S. DA, Natural Resources Conservation Services, 2010).

Seepage losses through buried bentonite blankets are approximately 0.2 - 0.25 $m^3/m^2/d$. This figure is for thin blankets and represents about a 60 percent improvement over ponds with no lining.

Asphalt
Asphalt linings may be buried on the surface and may be composed of fresh asphalt or a prefabricated asphalt. Some variations include:

- An asphalt membrane is produced by spraying asphalt at high temperatures. This lining may be either on the surface or buried. Special equipment is needed for installation. A useful life of 18 years or greater has been observed when these membranes are carefully applied and covered with an adequate layer of fine grained soil.
- Buried Asphaltic Membrane. This is similar to the first asphalt membrane, except that a gravel-sand cover is applied over the asphaltic membrane. This cover is usually more expensive and less effective in discouraging plant growth.
- Built-up Linings. These include several different types of materials. One type could be a fiberglass matting, which is applied over a sprayed asphalt layer and then sprayed or swept over with a sealed coat of asphalt or clay. A 280 g jute burlap has also been used as the interior layer between two hot-sprayed asphalt layers. In this case, the total asphalt application should be about 11.3 L/m^2. The prefabricated lining may be on the surface or buried. If it is buried, it could be covered with a layer of soil or, in some cases, a geotextile coating.
- Prefabricated Linings. These linings consist of a fiber or paper material coated with asphalt. This type of liner can be exposed or covered with soil. Joints between the material are sealed with asphaltic mastic. When the asphaltic material is covered, it is more effective and durable. When it is exposed, it should be coated with aluminized paint every three to four years to retard degradation. This is especially necessary above the water line. Joints also have to be maintained if they are not covered with fine-grained soil. Prefabricated asphalt membrane lining is approximately 0.32 - 0.64 cm thick. It may be handled in much the same way as rolled roofing, with lapped and cemented joints. Cover for this material is generally soil and gravel, although shot-crete and macadam may also be used.

Installation procedures for prefabricated asphalt membrane linings and for buried asphalt linings are similar to those for buried bentonite linings (U.S. DI, 2001). The preparation of the sub-grade is important; it should be stable and adequately smooth before the lining is put in place. Linings of bentonite and asphalt are sometimes unsuitable in areas of high weed growth, since weeds and tree roots readily puncture the membranes). Many lining failures occur as a result of rodent and crayfish holes in embankments. Asphalt membrane lining tends to decrease the damage, but, in some cases, harder surface linings are necessary to prevent water loss from embankment failures. Linings of hot applied buried asphalt membrane provide one of the tightest linings available. These linings last longer than other flexible membrane linings. Asphalt linings composed of prefabricated buried materials are best for small jobs, since there is a minimum amount of special equipment and labor connected with installation. For larger jobs sprayed asphalt is more economical. When fibers and filler composed of organic materials are used in building asphalt membranes, the membranes have a shorter lifetime. Inorganic fibers are, therefore, recommended. Typical seepage volume through one buried asphalt membrane after 10 years of service was consistently 0.02 $m^3/m^2/d$ (2.3×10^{-7} cm/s). Asphalt membrane linings can be constructed at any time of the year, but fall and winter installation may dictate the use of the buried asphalt membrane lining.

Buried asphalt membranes usually perform satisfactorily for more than 15 years. When these linings fail, it is generally due to one or more of the following causes:

- Placement of lining on unstable side slopes
- Inadequate protection of the membrane
- Weed growth
- Surface runoff
- Type of sub-grade material
- Cleaning operations
- Scour of cover material Membrane puncture

Soil Cement

The best results are obtained with soil cement when the soil mixed with the cement is sandy and well graded to a maximum thickness of about 2 cm. Soil cement should not be laid down in cold weather. It should be cured for about seven days after placement. Some variations of the soil cement lining are listed below.

- **Standard soil cement** is compacted using a water content of the optimum moisture content of the soil. (Moisture content is expressed in percent dry weight at which a given soil can be compacted to its maximum density by means of a standard method of compaction.) The mixing process is accomplished by traveling mixing machines and can be handled satisfactorily in slopes up to 4:1. Standard soil cement may be on the surface or buried.
- **Plastic soil cement** (surface or buried) is a mixture of soil and cement with a consistency comparable to that of Portland cement concrete. This requires the addition of a considerable amount of water. Plastic soil cement contains from three to six sacks of cement per cubic meter and is approximately 7.5 cm thick.
- **Cement modified soil** contains two to six percent volume of cement. This may be used with plastic fine grained soils. The treatment stabilizes the soil in sections subject to erosion. The lining is constructed by spreading cement on top of loose soil layers by a fertilizer-type spreader. The cement is then mixed with loose soil by a rotary traveling mixer and compacted with a sheeps-foot roller. A 7-day curing period is necessary for a cement modified soil. Soil cement has been used successfully in some cases in mild climates. Where wetting or drying is a factor, or if freezing-thawing cycles are present, the lining will deteriorate rapidly (U.S. DI, 2001).

Thin Membrane Liners

Plastic and elastomeric membranes are popular in applications requiring essentially zero permeability. These materials are economical, resistant to most chemicals if selected and installed properly, available in large sheets simplifying installation, and essentially impermeable. As environmental standards become more stringent, the application of plastic and elastomeric membranes will increase because of the need to guarantee protection against

seepage. This is particularly true for sealing ponds containing toxic wastewaters or the sealing of landfills containing toxic solids.
Most regulatory agencies have general standards for the application of liners, as do most manufacturers. Searching the Internet using key words such as "liners," "plastic liners," "seepage prevention," "sealing," "water proofing," or "membranes" will yield the most current information. Detailed drawings showing the correct method for the application of linings are presented in Kays (1986) and manufacturer's literature. These sources of information should be consulted before designing a liner.

The most difficult design problem encountered in liner application involves placing a liner in an existing pond. Effective design practices are essentially the same as those used in new systems, but the evaluation of the existing structure must be done carefully to achieve the required results. Lining materials must be selected for their compatibility with the existing structure. For example, if a badly cracked concrete lining is to be covered with a flexible synthetic material, it must be properly sealed and the flexible material placed in such a way so that any movement will not destroy the integrity of the new liner. Sealing around existing columns and footings must also be considered.

Protection of a thin membrane lining is essential. Kays (1986) recommends that a fence at least 2 m high be placed on the outside berm slope with the top of the fence below the top elevation of the dike to keep the membrane out of sight.

There are many firms specializing in the installation of lining materials. Most installation companies and manufacturers publish specifications and installation instructions and design details. The manufacturers' and installers' recommendations are similar, but there are differences worthy of consideration when designing a system requiring a liner. For details, consult the Internet for manufacturers using the key words "Pond Liners".

New products are always being developed, and with each new material the options available to designers expand. If the liners are chosen and installed with care, control of seepage and associated pollution should become a minor operation and maintenance element.

4.3.4.2 Mechanisms of Failure

Kays (1986) classified the principal types of failures observed in cut-and-fill reservoirs (Table 4-3). The list is extensive and case histories involving all of the categories are available; however, the most frequently observed failure mechanisms were the lack of integrity in the lining support structure and abuse of the liner. For example, exposed thin membrane liners must be protected from aerator damage, contact with sharp objects, and excess foot traffic. In general, unless a protective cover is provided, it is advisable to use reinforced membranes or thick materials recommended for exposure to the elements.

Table 4-3. Classification of the Principal Failure Mechanisms for Cut-and-Fill Reservoirs (Kays, 1986)[a].

Supporting Structure	Lining
Underdrains	**Mechanical**
Substrate	Field seams
Compaction	Fish moughts[b]
Texture	Structure seals
Voids	Bridging
Subsidence	Porosity
Holes and cracks	Holes and pinholes
Groundwater	Tear strength
Expansive clays	Tensile strength
Out gassing	Extursion and extension
Sloughing	Animals including burrowing birds
Slope anchor stability	Insects
Mud	Weeds
Frozen ground and ice	
	Weather
Operations	General weathering
Cavitation	Wind
Impingement	Wave erosion
Lack of regular cleaning	Ozone
Reverse hydrostatic uplift	
Vandalism	
Seismic activity	

[a]Courtesy of John Wiley and Sons, Inc., New York, NY.
[b]Separation of butyl-type cured sheets at the joint due to unequal tension between the sheets.

4.3.4.3 Cover Material

The cost of linings for ponds vary with the type and the quality required to ensure against seepage problems. Contacting individual suppliers will yield accurate and up-to-date cost information.

Placing cover material over buried membranes is the most expensive part of the procedure. The cover material should, therefore, be as thin as possible, while still providing adequate protection for the membrane. If a significant hydraulic current is present in the pond, the depth of coverage should be greater than 25 cm, and this minimum depth should only be used when the material is erosion resistant and cohesive. Such a material as a clay gravel is suitable. If the material is not cohesive, or if it is fine grained, a higher amount of cover is needed.

Maintenance costs for different types of linings are difficult to estimate. Maintenance should include repair of holes, cracks, and deterioration and damage caused by animals and pond cleaning, as well as weed control expenses. Climate, type of operation, type of terrain, and surface conditions also influence maintenance costs.

Synthetic liners are most practical where zero or minimum seepage regulations are in effect, a facility is treating industrial waste that might degrade concrete or earthen liners and/or there are extremes in climatic conditions

4.4 POND HYDRAULICS

4.4.1 Inlet and Outlet Configuration

In the past, the majority of ponds were designed to receive influent wastewater through a single pipe, usually located toward the center of the pond. Hydraulic and performance studies (Mangelson, 1971; Ewald, 1973; Mangelson and Watters, 1972; Finney and Middlebrooks, 1980; Middlebrooks et al., 1982; Shilton, 2005; Crites et al., 2006) have shown that the use of centralized inlet structures is an inefficient method of introducing wastewater to a pond, often resulting in less than ideal residence time. Multiple inlet arrangements are preferred, even in small ponds (<0.5 ha) and preferably by means of a long splitter box with multiple outlets large enough to avoid plugging by influent solids. The splitter box should be located at approximately mid-depth above the sludge blanket. Outlets should be located as far as possible from the inlets. The inlets and outlets should be placed so that flow through the pond has a uniform velocity profile between the next inlet and outlet.

Single inlets can be used successfully if the inlet is located at the greatest distance possible from the outlet structure and baffled or the flow directed to avoid currents and short circuiting. Outlet structures should be designed for multiple depth withdrawal, and all withdrawals should be a minimum of 0.6 m below the water surface.

4.4.1.1 Pond Transfer Inlets and Outlets

Pond transfer inlets and outlets should be constructed to minimize head loss at peak recirculation rates, assure uniform distribution to all pond areas at all recirculation rates, and maintain water-surface continuity between the supply channel, the ponds, and the return channel. See Section 4.5 for further discussion of recirculation.

Transfer pipes should be numerous and large enough to limit peak head loss to about 7-10 cm with the pipes flowing about two-thirds to three-quarters full. Supply- and return-channel sizing should assure that the total channel loss is no more than one-tenth of the transfer-pipe losses. When such a ratio is maintained, uniform distribution is assured.

By operating with the transfer pipes less than full, unobstructed water surface is maintained between the channels and ponds and scum does not build up in any one area. If the first cell is designed to remove scum, then the transfer pipes must be submerged.

Transfer inlets and outlets usually are made of plastic pipe or bitumastic-coated, corrugated metal pipe, and have seepage collars located near the midpoint. These types of pipe are inexpensive, but strong enough to allow for the differential settlement often encountered in pond-dike construction.

Specially made fiberglass plugs can be provided to close the pipes. The plugs may be installed from a boat. Such plugs permit any pipe to be closed without expensive construction of sluice gates and access platforms at each transfer point. Launching ramps into each pond and channel are recommended to assure easy boat access for sampling, aquatic plant control and pond maintenance.

4.4.2 Baffling

Better treatment is obtained when the flow is guided through the pond. Treatment efficiency, economics and esthetics play an important role in deciding whether or not baffling is desirable. Because there is little horizontal force on baffling except that caused by wave action, the baffle structure need not be particularly strong. The lateral spacing and length of the baffle should be specified so that the cross-sectional area of the flow is as close to a constant as possible.
It may also be placed below the pond surface to help overcome esthetic objections. A typical type of baffle might be a submerged fence attached to posts driven into the pond bottom and covered with a flexible, heavy plastic membrane. Commercial float-supported plastic baffling for ponds also is available.

Baffling has additional advantages. The spiral action induced when flow occurs around the end of the baffles enhances mixing and tends to break up or prevent any stratification or tendency to stratify. Reynolds et al. (1975) and Polprasert and Agarwalla (1995) have discussed the advantages of biomass distribution and attachment to baffles leading to improved pond efficiency. It should be mentioned that winter ice can damage or destroy baffles in cold climates.

4.4.3 Wind Effects

Wind generates a circulatory flow in bodies of water. To minimize short circuiting due to wind, the pond inlet-outlet axis should be aligned perpendicular to the prevailing wind direction. If for some reason the inlet-outlet axis cannot be oriented properly, baffling can be used to control, to some extent, the wind-induced circulation. Where the pond depth is constant, the surface current is in the direction of the wind and the return flow is in the upwind direction along the bottom.

4.4.4 Stratified Ponds

Ponds that are stratified because of temperature differences between the inflow and the pond contents tend to behave differently in winter and summer. In summer, the inflow is generally colder than the pond water. It sinks to the pond bottom and flows toward the outlet. In the winter, the reverse is true. The inflow rises to the surface and flows toward the outlet. A likely consequence of this is that the effective volume of the pond is reduced to that of the stratified inflow layer (density current). The result can be a drastic decrease in detention time and an unacceptable level of treatment.

One strategy is to use selective pond outlets positioned vertically so that outflow is drawn from the layer with density different from that of the inflow. For example, under summer conditions, the inflow will occur along the pond bottom. Hence, the outlets should draw from water near the pond surface.

Another approach is to premix the inflow with pond water while in the pipe or diffuser system, thereby decreasing the density difference. This could be accomplished by regularly constricting the submerged inflow diffuser pipe and locating openings in the pipe at the constrictions. The low pressure at the pipe constrictions would draw in pond water and mix it with the inflow to alter the density. However, in this case, clogging of openings with solid material could be a problem.

4.5 POND RECIRCULATION AND CONFIGURATION

Pond recirculation involves inter- and intra-pond recirculation as opposed to mechanical mixing in the pond cell. The effluents from pond cells are mixed with the influent to the cells. In intra-pond recirculation, effluent from a single cell is returned to the influent to that cell. In inter-pond recirculation, effluent from another pond is returned and mixed with influent to the pond (Figure 4-3).

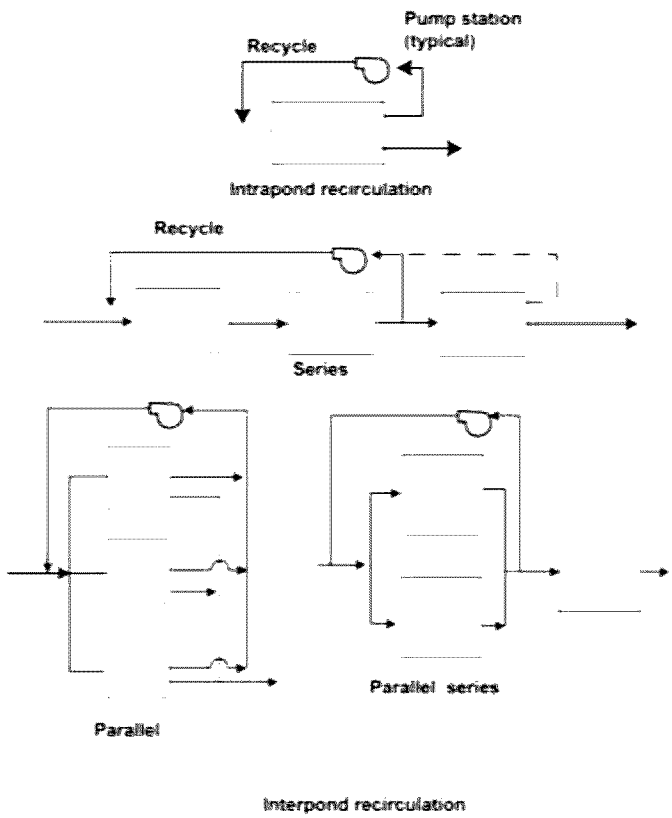


Figure 4-3. Common pond configurations and recirculation systems.

Both methods return active algal cells to the feed area to provide photosynthetic oxygen that can treat some of the organic load. The principal benefit is to control odors and anaerobic conditions in that area of the pond. Inter- and intra-pond recirculation can also affect stratification in ponds. Pond recirculation, however, is not as efficient as mechanical mixing in facultative ponds.

Excessive organic loading of primary ponds and the associated odors can be mitigated by recirculating treated effluent to the overloaded ponds. Recirculation dilutes high BOD_5 concentrations and essentially pushes BOD_5 into downstream ponds, spreading the load over a greater pond surface area in a shorter time than influent flow could do alone. If the recirculated water originates in ponds with high DO, some of this DO mass is brought into the overloaded ponds. However, conventional aeration is likely to be a more energy efficient means of providing DO mass than pumping recirculation water.

Another effect of recirculation is inoculation of primary ponds with microalgae and other organisms from downstream ponds. Inoculation may help maintain algal populations in primary ponds, leading to increased photosynthetic DO and odor control. To promote photosynthesis, water should be recirculated to the surface of the primary ponds in an attempt to form a cap of algae-rich water. Ideally, this recirculated water is warmer than the bulk of the water in the primary ponds. In this way, a surface layer of recirculated water is promoted. However, such a surface cap may be short-lived if night time cooling of the surface allows the surface water to cool and sink. Three common types of inter-pond recirculation systems (series, parallel, and parallel series) are shown in Figure 4-3.

Recirculation in series

Recirculating wastewater through the pond series dilutes the organic mass in the first cell by increasing the flow rate. Neither the mass of material entering the cell nor the surface loading rate (mass/unit area/d) is reduced by this configuration. Intra-cell recirculation does reduce the HRT of the water in the cell receiving the recirculated flow, but not the overall HRT of the system. The method attempts to flush the influent through the pond system faster than it would travel without recirculation, thereby reducing the concentration in the reactor. The reduction in HRT might offset any advantages gained other than odor control. This reduction in the first-pass HRT of the influent and recycled mixture in the first, most heavily loaded, pond in the series system is:

$$t = \frac{V}{(1 + r)Q} \qquad (4\text{-}2)$$

Where:

V = the volume of pond cell
Q = the influent flow rate
r = the recycle flow rate
r/Q = the recycle ratio
t=time

Another effect of recirculation in the series configuration is to reduce the BOD_5 in the mixture entering the pond, and is given by the expression:

$$S_m = \frac{S_{in}}{1+r} + \left(\frac{r}{1+r}\right)S_e \qquad (4\text{-}3)$$

Where:

S_m = the BOD_5 of the mixture
S_e = the effluent BOD_5 from the third cell
S_{in} = the influent BOD_5
r = recycle flow rate

S_m would be only 20 percent of S_{in} with a 4:1 recyle ratio, as S_e would be negligible in almost all cases. Thus, the application of organic load in the pond is spread more evenly throughout the ponds, and organic loading and odor generation near the feed points are reduced. Recirculation in series has been used to reduce odors in those cases where the first pond is anaerobic. The recirculation ratio is selected based on the loading rate applied to the cell that will not cause a nuisance.

Recirculation in parallel

The parallel configuration more effectively reduces pond loadings than does the series configuration, because the mixture of influent is spread evenly across all ponds instead of the first pond. For example, consider three ponds, either in series or parallel. In the parallel configuration, the surface loading (kg BOD_5/ha/d) on the three ponds is one-third that of the first pond in the series configuration. The parallel configuration, therefore, is less likely to produce odors than the series configuration. However, the hydraulic improvements in design using a series configuration generally will offset the benefits of reduced loading in parallel configuration.

Based upon the analyses of performance data from selected aerated and facultative ponds, four ponds in series give the best BOD_5 and fecal coliform removals for ponds designed as plug flow systems. Good performance can be obtained with a smaller number of ponds if baffles or dikes are used to optimize the hydraulic characteristics of the system.

Recirculation usually is accomplished with high-volume, low-head propeller pumps. Figure 4-4 presents a simplified cross-section of such an installation. In this design, the cost and maintenance problems associated with large discharge flap gates are eliminated by the siphon discharge. An auxiliary pump with an air eductor maintains the siphon. Siphon breaks are provided to ensure positive backflow protection.

Pumping stations of this type can be designed to maintain full capacity with minimal increase in horsepower even when the inlet and discharge surface levels fluctuate over a range of 1.0 - 1.2 m. Multiple- and/or variable-speed pumps are used to adjust the recirculation rate to seasonal load changes.

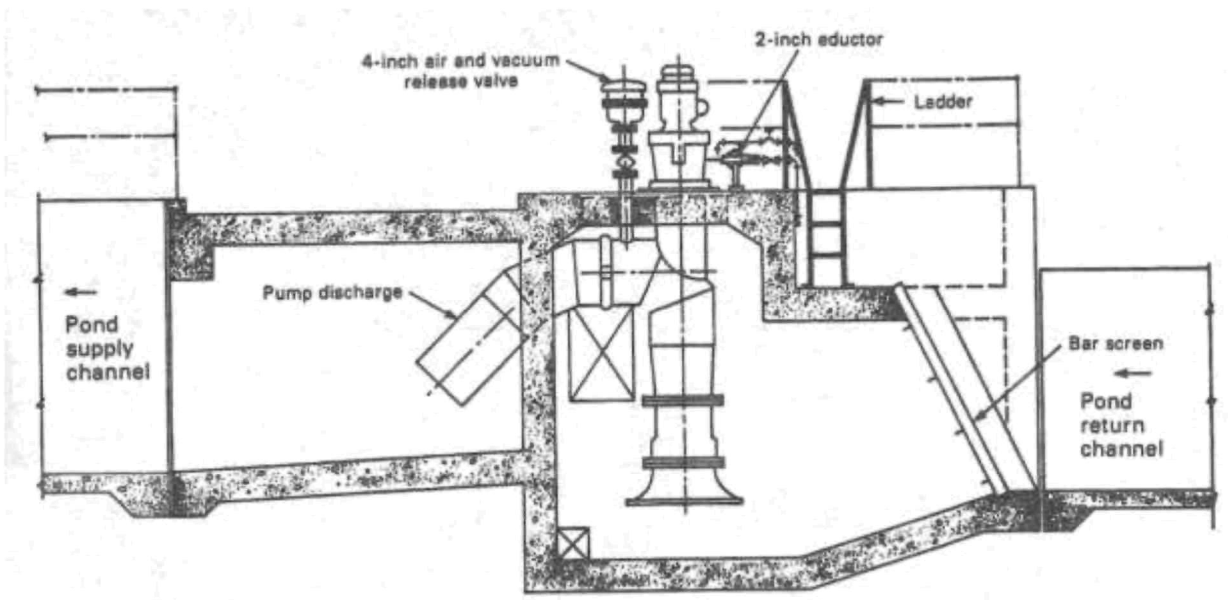


Figure 4-4. Cross-sectional view of a typical recirculation pumping station.

Pond configuration should allow full use of the wetted pond area. Transfer inlets and outlets should be located to eliminate dead spots and short circuiting that may be detrimental to photosynthetic processes. Wind directions should be studied and transfer outlets located to prevent dead pockets where scum will tend to accumulate. Pond size need not be limited, as long as proper distribution is maintained.

CHAPTER 5

ADVANCES IN POND DESIGN

5.1 INTRODUCTION

In Chapter 3, the basic design of wastewater treatment ponds was described. In this chapter, we discuss design innovations that have come about through a better understanding of the biological processes occurring in these systems and experience building them under different environmental conditions. Many of these innovations have allowed communities to retain their pond system with modifications rather than abandoning them in favor of mechanized systems. Examples include fermentation pits in facultative ponds to protect anaerobic digestion, streamlined footprints, improved aerators or mixers and additions of covers to retard algal growth in polishing ponds.

5.2 ADVANCED INTEGRATED WASTEWATER POND SYSTEMS® (AIWPS®)

5.2.1 Description

The AIWPS® Technology (Oswald, 1977; Oswald and Green, 2000) has evolved over a 50-year period of research by Dr. William J. Oswald at the University of California, Berkeley and other locations. The majority of the research and operations experience has involved facilities in areas with moderate climates. Certain aspects of the process are currently patent-protected, with new patents pending. The salient features of the patent reflect the evolution of the design, including a smaller land footprint, an increased depth in the primary pond, and appropriately sized aperture for the in-pond digesters (IPDs). The evolution of AIWPS® design vs. size is demonstrated in Table 5-1 and shown in Figure 5-1.

Table 5-1. Evolution of the Design of Selected AIWPS® Wastewater Treatment Facilities 1965 to present.

AIWPS® WWTF Location, Type, and Design Date	Design Capacity in MLD	Treatment Area in hectares	Treatment Area/ Design Capacity ha/MLD
Type I			
St Helena, CA 1965	1.1	9.3	8.1
Hollister, CA 1977	7.6	19.2	2.5
Varanasi, Sota, INDIA 1997	200	220	1.1
Delhi, CA 1998	5.7	7.9	1.4
Hilmar, CA 1999	3.8	7.7	1.9
Wanganui, NEW ZEALAND 2002	23	49	2.1
Rosamond, CA 2009	7.6	16.7	2.2
Varanasi, Ramana, INDIA	40	45	1.1

2010			
Type II			
Napa, CA 1966	45.4	142.5	3.1
Bolinas, CA 1973	0.2	2.2	3.7
Ridgemark Estates, CA 1975	0.5	1.7	3.4

See p.xiv for conversion table.

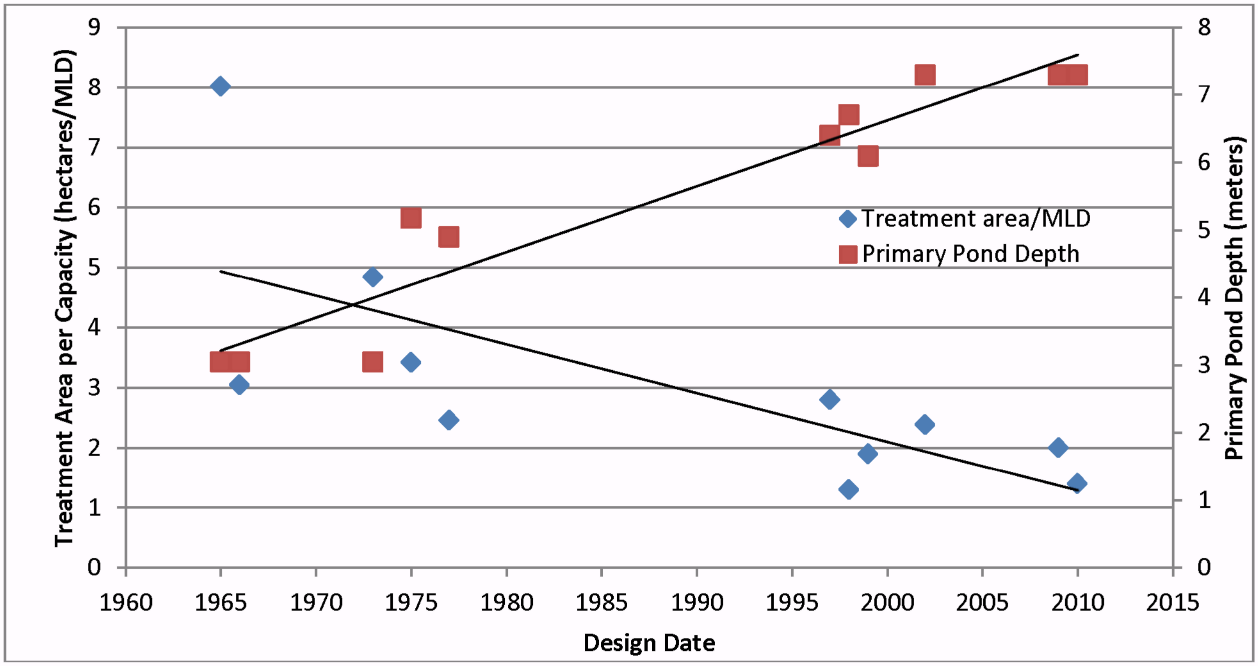


Figure 5-1. AIWPS® Facilities over time showing the design trend of increasing primary pond depth in meters and decreasing footprint area per treatment capacity in hectares per million liters per day (MLD) of capacity.

Typically, the AIWPS® technology consists of compacted earthen dikes, such as those used in conventional pond systems, with a minimum of four types of ponds in series. These separate unit processes in the treatment scheme reduce the impact of short-circuiting in the ponds and provide for depth control in areas where evaporation exceeds precipitation. While the unit processes in the AIWPS® do not differ from those in conventional wastewater treatment pond systems (i.e., primary sedimentation, flotation, anaerobic digestion, methane fermentation, aeration, photosynthetic oxygenation, nutrient removal, secondary sedimentation, nutrient removal, storage and final disposal or reclamation and reuse), their application and some patented design features do differ from those in most conventional pond systems. Figure 5-2 is a plan view and a schematic profile of a Type 1 AIWPS® facility. This is followed by a description of the functions of the individual ponds.

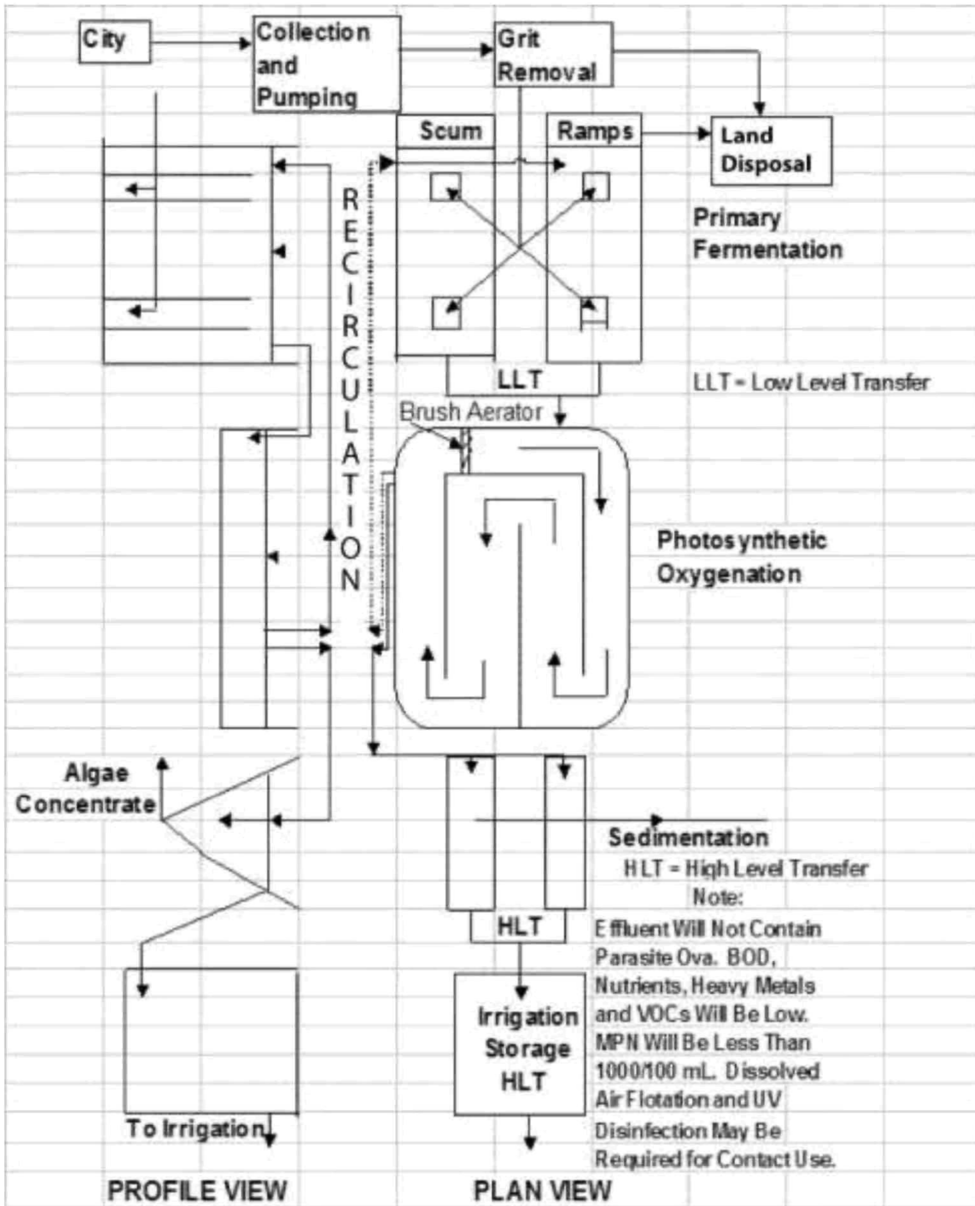
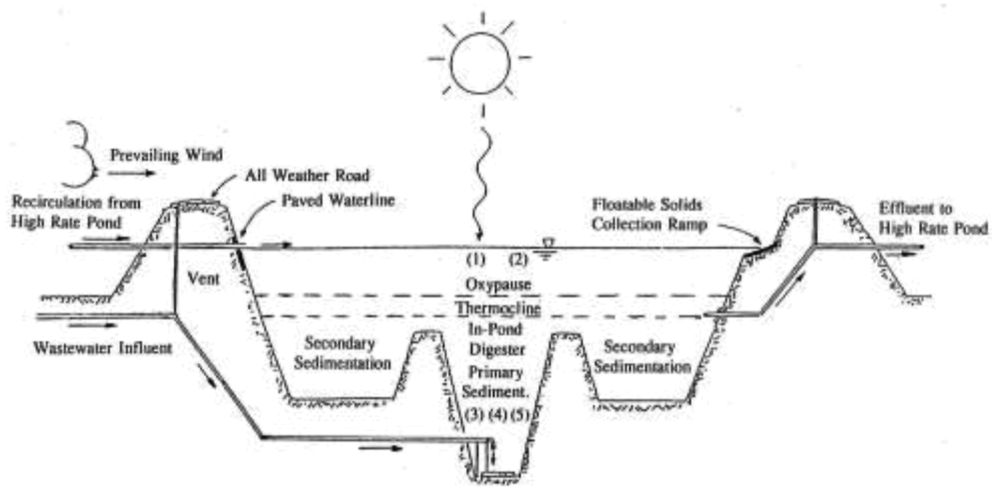


Figure 5-2. Type I Advanced Integrated Wastewater Pond System®.

The first pond in the series is an Advanced Facultative Pond (AFP) that contains one of the unique features of an AIWPS® Facility. The AFP contains deep anaerobic digestion and methane fermentation pits with a minimum depth of 6 - 8m through which all of the influent wastewater upflows. Figure 5-3 shows a schematic cross section of the AFP. The great depth and certain design elements prevent DO from entering the fermentation pit or IPD, thereby improving methane fermentation and reducing solids transfer to subsequent ponds. Most settleable solids are retained in the pit and undergo anaerobic digestion and methane fermentation. The overflow velocity is limited to less than 2.0 m/d to optimize sedimentation and to prevent most helminth ova and ova cysts from leaving the fermentation pit. Sludge fermentation has been shown to be sufficiently complete to essentially eliminate sludge handling in moderate climates for many decades. A public works budget should maintain a line item for removal activities against the time when it will be necessary to remove sludge and a plan should be in place to compost it, preferably on site.



(1) Aerobic Oxidation
(2) Photosynthesis
(3) Organic Acid formation (Putrefaction)
(4) Methane formation
(5) Carbon dioxide reduction

Figure 5-3. Schematic Cross-section of primary facultative pond of an Advanced Integrated Wastewater Pond System® (Oswald, 1996).

Limited data are available for cold climate operation, but indications are that fermentation occurs in areas with weather as severe as that in the high country of the Rocky Mountains. Floatables are collected on a down-wind concrete scum ramp and removed periodically. An AFP will reportedly remove at least 60 percent of the influent BOD_5 and a greater percentage of TSS. In the southwestern United States, removals up to 80 percent have been observed. Outlets from the AFP are located 1 – 1.3 m below the surface to prevent discharge of floatables into the secondary pond. Recirculation with algal-laden waters with levels of O_2 and supplementary mechanical aeration controls odors.

The second pond in series may be a Secondary Facultative Pond (SFP) or a High Rate Pond (HRP), depending on the desired level of treatment. Figure 5-4 shows a schematic drawing of the raceway in the HRP. The HRP is used to produce high concentrations of algae and DO. With recirculation, the DO can be used for odor control at the surface of the AFP. The algae remove nutrients and can also serve useful purposes (Woertz et al., 2009). HRP systems are usually less than one meter in depth and mixed with low-speed paddle wheel mixers (brush aerators in Figure 5-2) at a mean surface velocity of approximately 15 cm per second. The elevated *p*H in the HRP along with the intensity of the sunlight it receives contributes to maintaining high kill rates of bacteria present in the influent. These high *p*H values and temperature in the summer contribute to NH_3 removal from the wastewater by volatilization, although this is not the only NH_3 removal mechanism. Effluent from the HRP is withdrawn from the surface to obtain the highest quality water with high DO concentrations and high *p*H values.

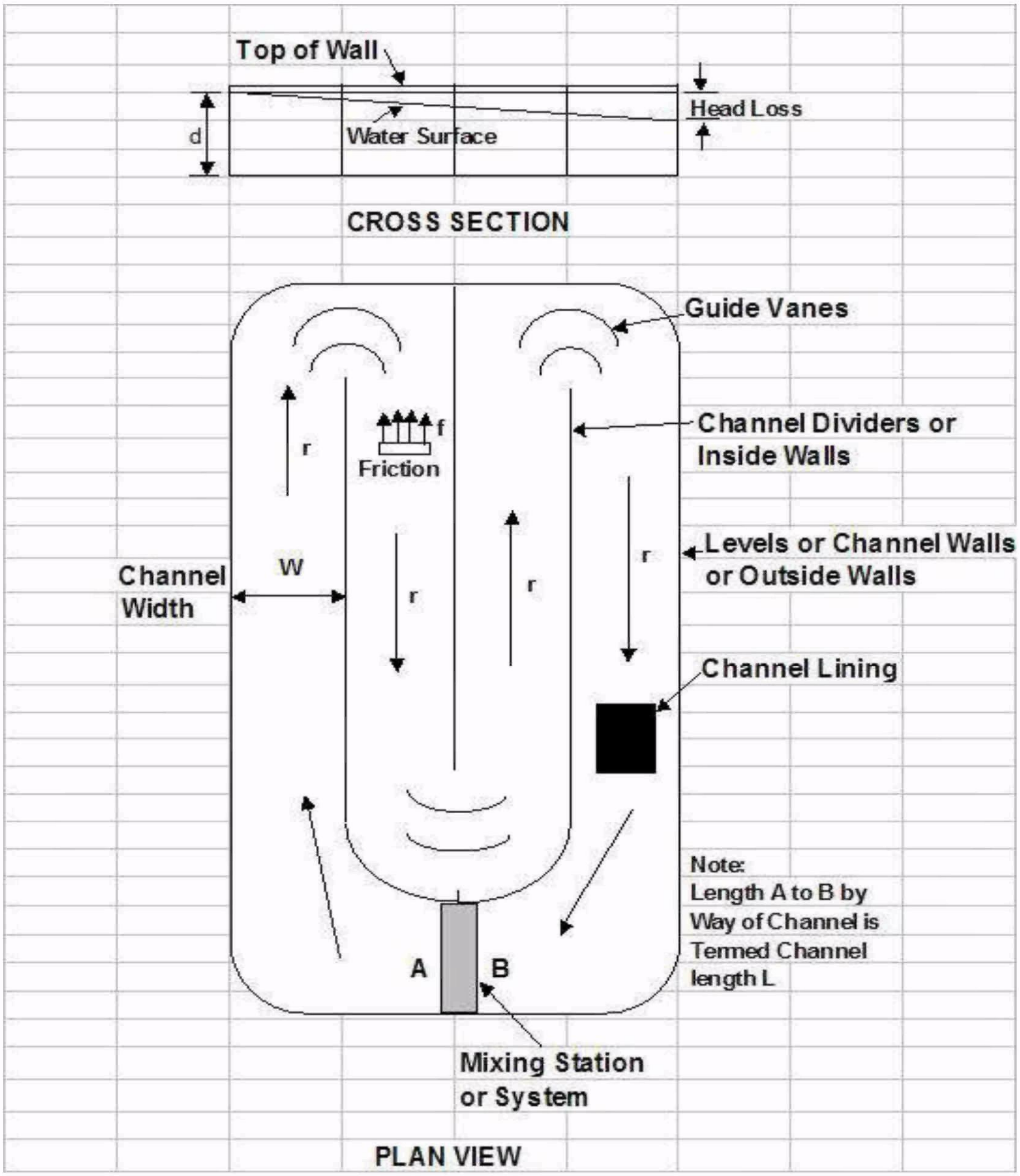


Figure 5-4. Schematic of raceway to cultivate microalgae for O_2 production. (Oswald, 1996).

Oswald does not recommend chlorination of the effluent from AIWPS® systems because of low MPN counts in the effluent and the fact that chlorine doses above 10 mg/L are required to kill algae, potentially releasing their nutrients and BOD_5 back into the water column. However, it is likely that many states will still require chemical disinfection of these effluents, and if such a requirement is set, Oswald recommends that algae should be removed prior to chlorination. Algae can tolerate ozone disinfection.

The third pond is used for algal sedimentation and collection for drying, while the fourth is for storage, further disinfection and TSS removal. In areas where advanced treatment may not be feasible and human contact is expected, deep maturation ponds with detention times of 10 - 20 days following treatment in AFP, HRP and Algae Settling Pond (ASP) in series will provide reliable control of pathogenic microorganisms of human origin.

5.2.2 Modifications

The Type 2 AIWPS® modification provides supplementary aeration to replace algae as the source of DO in the primary pond surface. The primary pond (AFP) is designed exactly as the primary pond in the AIWPS® Type 1. In Type 2 AIWPS® facilities has less land, but more energy for mechanical aeration is required.

5.2.3 Applicability

Where land costs are not a limiting factor, the concept is well suited for the treatment of municipal wastewater and biodegradable industrial and agricultural wastewaters. New configurations of the design have reduced the footprint and thus the land requirements (Table 5-2).

Table 5-2. AIWPS® Types I and II with Treatment Areas (in acres)[1].

AIWPS® WWTF	Design Flow (MGD)	Total Treatment Wet Surface Area (not including roads) (acres)	Total Treatment Wet Surface Area/MGD design capacity (acres/MGD)
St. Helena CA Type I)	0.3-3.0	30.86	10.3
Napa CA (Type II)[2]	12	342.2	28.5
Ridgemark Estates CA (Type II)	0.13	4.1	31.5
Bolinas CA (Type II)	0.05-0.12	5.44	45.3
Varanasi Uttar Pradesh India	54	566.70	10.5
Delhi CA	1.5	19.86	13.2
Hilmar CA	1.0	17.8	17.8
Wanganui New Zealand	6.2	117.87	19
Melbourne Australia 25 W		586.26	
Melbourne Australia 55E		505.5	
Ramana[2] Uttar Pradesh India	10	91.9	9

[1]See p. xiv for conversion to hectares.
[2]From Napa to Ramana, the treatment system footprint has decreased by 200 percent, from Napa to Delhi, by 100 percent.

5.2.4 Advantages and Disadvantages

The advantages of the AIWPS® technology are reliable treatment, much reduced need for sludge disposal, lower energy and land requirements, which result in considerable cost reduction for operation and maintenance. Additional information about the process and operational data are available in the following sources (Oswald, 1990a; 1990b, 1995, 1996, 2003; Oswald et al., 1994; Nurdogan and Oswald, 1995; Green et al., 1995a; Green et al., 1995b; Green et al., 1996; Green et al., 2003; U.S. EPA, 2000a; Downing et al., 2002).

Another advantage of the AIWPS® technology is the ability of the sludge blanket to retain parasite eggs (ova), adsorb toxic materials, and convert organic N into N_2. Toxicity tests conducted over forty years demonstrate that the AIWPS® is able to produce an effluent from municipal/industrial wastewaters that will satisfy most regulatory requirements.

One disadvantage of the AIWPS® technology is that land is required for disposal of the effluent at certain times of the year to remove algae from the effluent. If there is no recirculation of the settling pond effluent, odors may be present in the advanced facultative pond. The use of surface mechanical aeration in the primary pond at those times will correct the situation.

5.2.5 Design Criteria

Design criteria can be obtained in "A Syllabus on Advanced Integrated Pond Systems" (Oswald, 1996). Further information can be found in Crites et al., 2006. The exact number of operating AIWPS® systems is unknown, but many are in operation around the world. Four complete full-scale AIWPS® units have been closely studied in California, and large-scale pilot units have been built and studied in the Philippines, Australia, Tunisia, Kuwait, South Africa, France, Indonesia, Thailand, Morocco, and Spain.

5.2.6 Performance

Some examples of Type I systems are listed in Table 5-3. More detailed descriptions of AIWPS® include St. Helena, Bolinas, Delhi and Hilmar, all in California.

Table 5-3. Type I Advanced Integrated Wastewater Pond Systems (AIWPS®).

Name of Site	St. Helena	Delhi	Hilmar
Location in CA	Napa County, Napa Valley, adjacent to Napa River	Merced County, San Joaquin Valley, Merced River Basin	Merced County, San Joaquin Valley, Merced River Basin
Climate	3-28° C	3 - 35° C	3 - 35° C
Population Served	5,000	7,000	5,000
Date Commissioned	1967	1998	2004
Type	1st Type I AIWPS®	Type I AIWPS®	Type I AIWPS®
Design Capacity	0.3 MGD	1.2 MGD	1.0 MGD
Influent Characteristics			
Present Flow	0.5 MGD	0.67 MGD	0.45 MGD
BOD_5	430 mg/L	200 mg/L	240 mg/L
Type	Municipal with two small seasonal wineries	Municipal	Municipal with seasonal FOG
Effluent Disposal	Beneficial Reuse for irrigation adjacent to Napa River	Percolation disposal and beneficial reuse for agricultural irrigation	Percolation disposal
Total WWTF Area	124 acres incl. 90 for irrigation reuse	37 acres including 9 for effluent percolation	40 acres including 12 for effluent percolation
Residual Solids Disposal	No removal to date	No accumulation to date	No accumulation to date
Pond Design			
Treatment Wet Surface Area	31 acres	20 acres	18 acres
Advanced Facultative Pond(s)	Yes (1 with baffle-protected fermentation crib)	Yes (2 operated in parallel each with one fermentation pit)	Yes (2 operated in parallel each with two In-Pond Digesters)
High Rate Pond(s)	Yes (1 serpentine)	Yes (2 peripheral)	Yes (2 peripheral)
Algae Settling	Yes (1)	Yes (2 in parallel)	Yes (2 in parallel)

Pond(s)			
Maturation Pond(s)	Yes (2 in series)	Yes (3 in series)	Yes (2 in series)
Algae Drying Beds	None	Yes (4)	Yes (4)
Operators	2	2 (part-time)	2 (part-time)

5.2.6.1 St. Helena, California

The St. Helena, California system was the first full-scale Type 1 AIWPS® Facility designed and constructed under the supervision of Dr. Oswald. It has been in operation for over 40 years. The site includes a spray irrigation and secondary reclamation field for land application for use during the dry season (May to October). The system discharges to the Napa River intermittently during the winter.

Figure 5-5 shows a plan view of the St. Helena system, not including the irrigated pasture adjacent to the treatment facility. Table 5-3 summarizes the characteristics of the facility. The wastewater treatment system serves about 6,000 people (2000 Census) at an average flow rate of 1.5 - 1.9 ML/d (400 - 500, 000 g/d) on about 6 ha. There are approximately 36 ha of grassland available for summer time use.

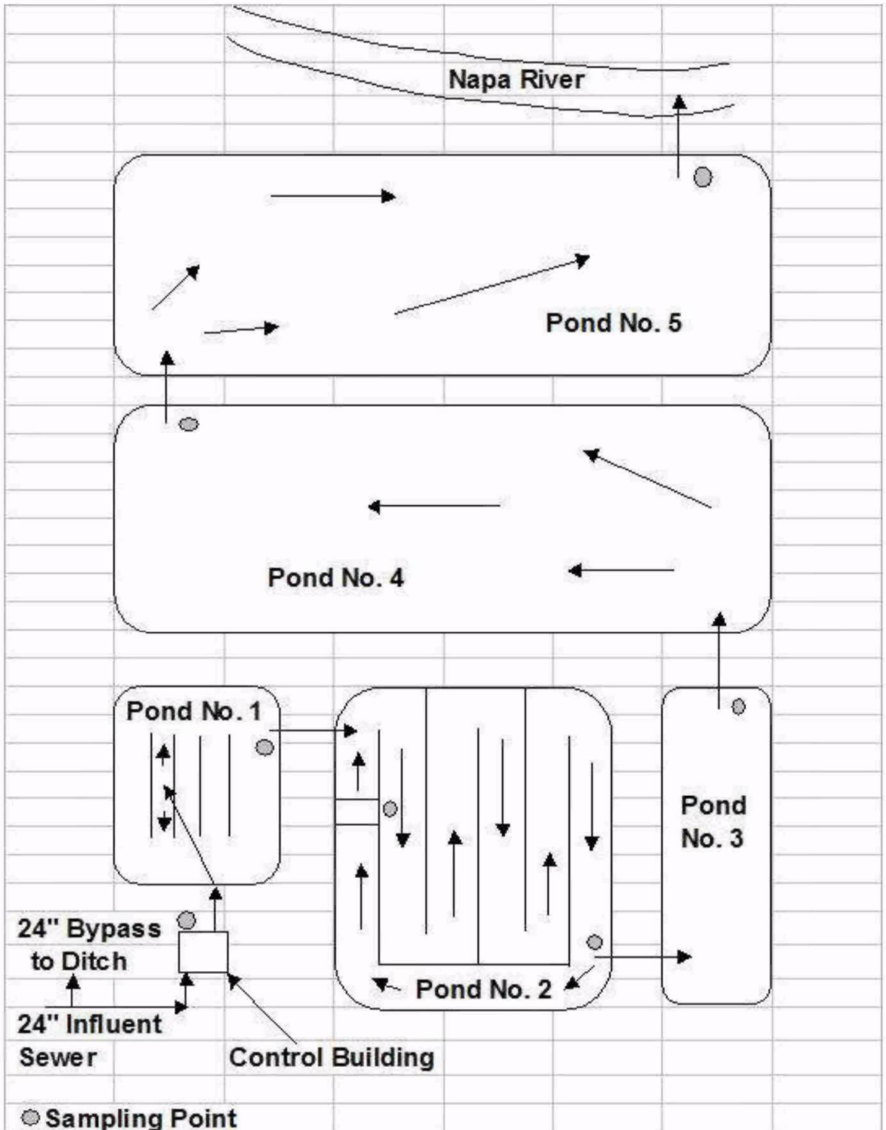


Figure 5-5. Plan view of St. Helena, California, AIWPS® (Oswald, 1996).

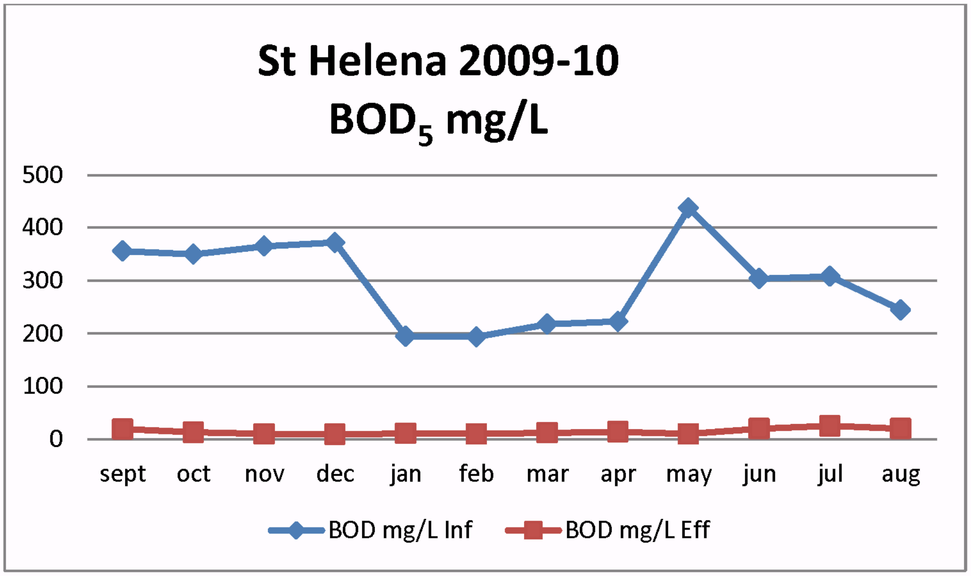


Figure 5-6. St. Helena biochemical oxygen demand.

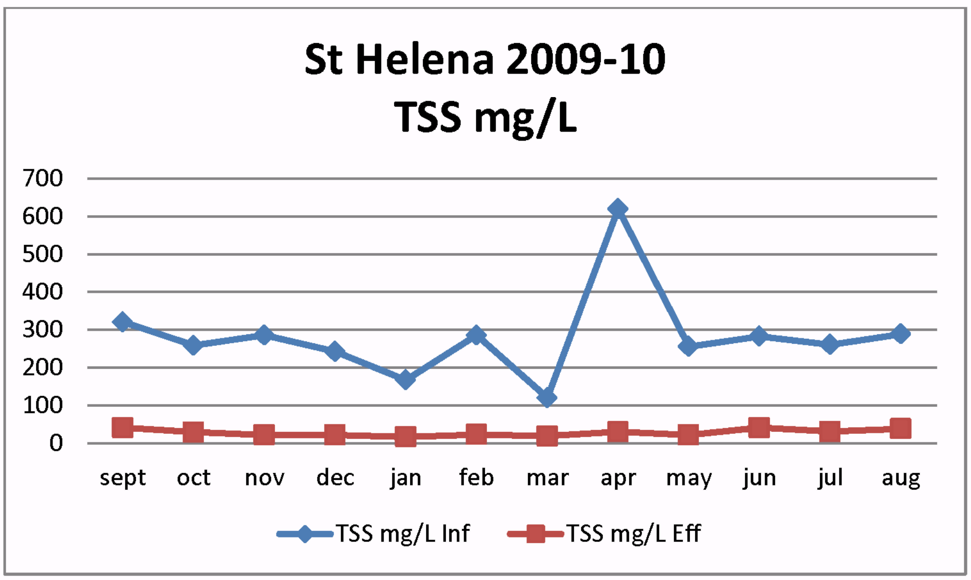


Figure 5-7. St. Helena total suspended solids.

Figures 5-6 and 5-7 present the BOD_5 and TSS performance data for the St. Helena system. Figure 5-8 presents a summary of annual means for total P, total N and BOD_5 (Meron, 1970) for the various stages of the St. Helena system.

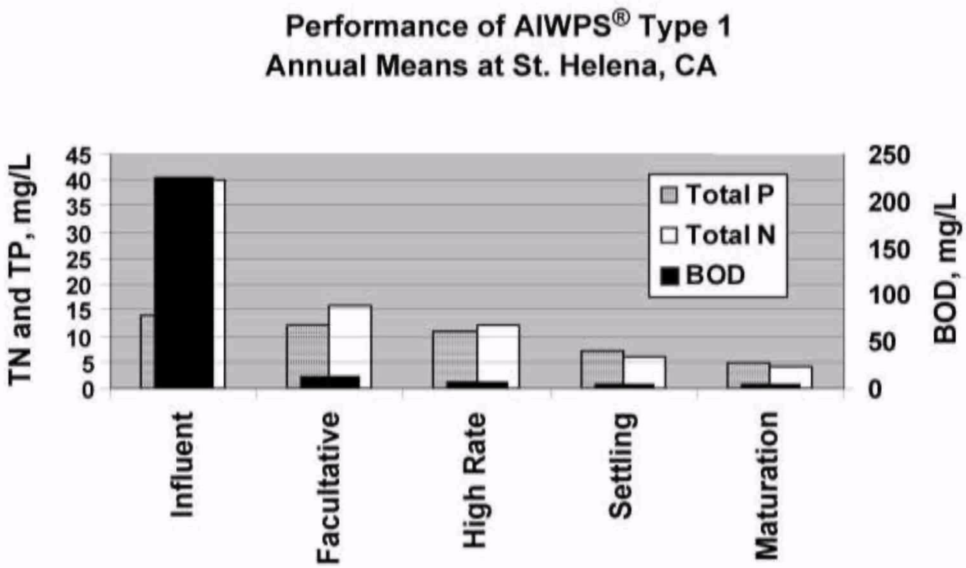


Figure 5-8. Performance of AIWPS® Type 1: annual means at St. Helena (Meron, 1970).

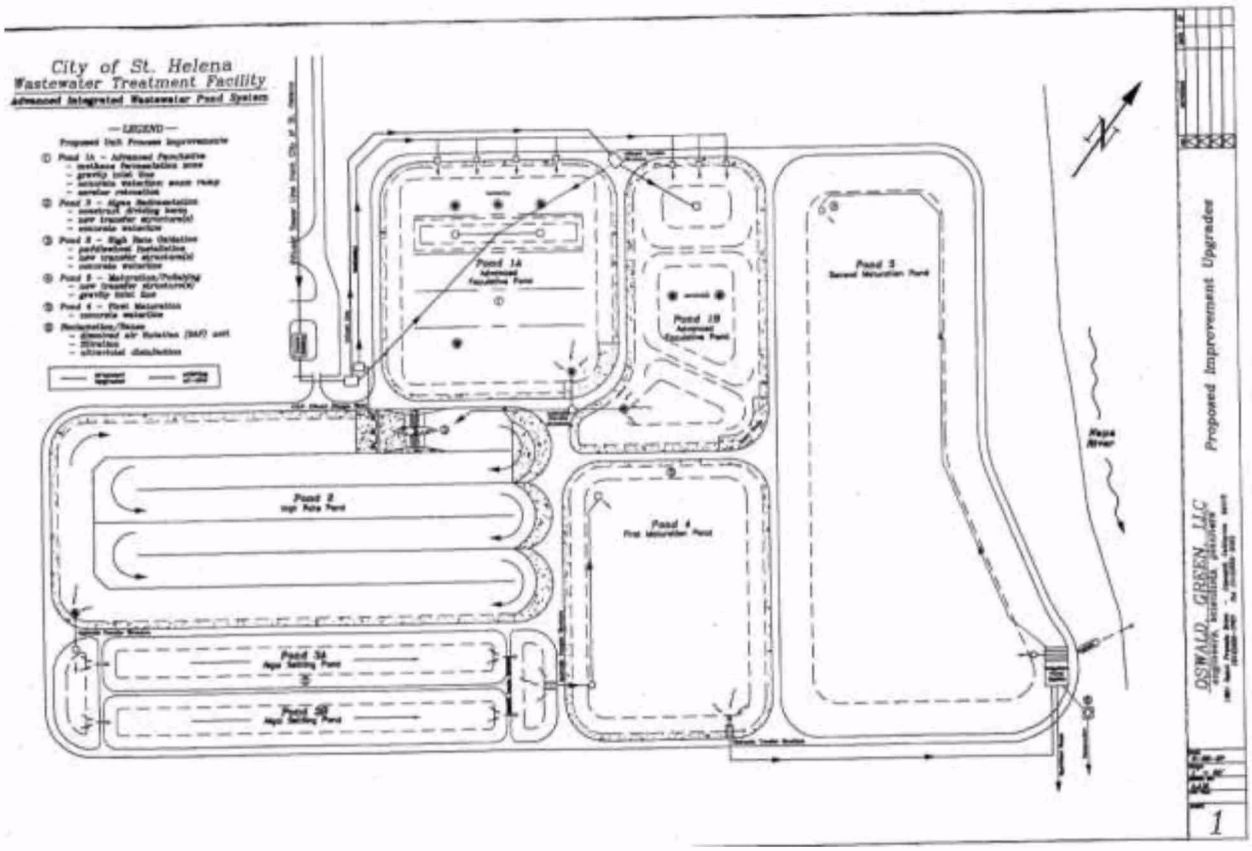


Figure 5-9A. Configuration of the St. Helena AIWPS®. Pond 4 has been divided into 1B and 4. 1B was designed to treat seasonal industrial waste from small winery operations. Currently all influent is sent to 1B.

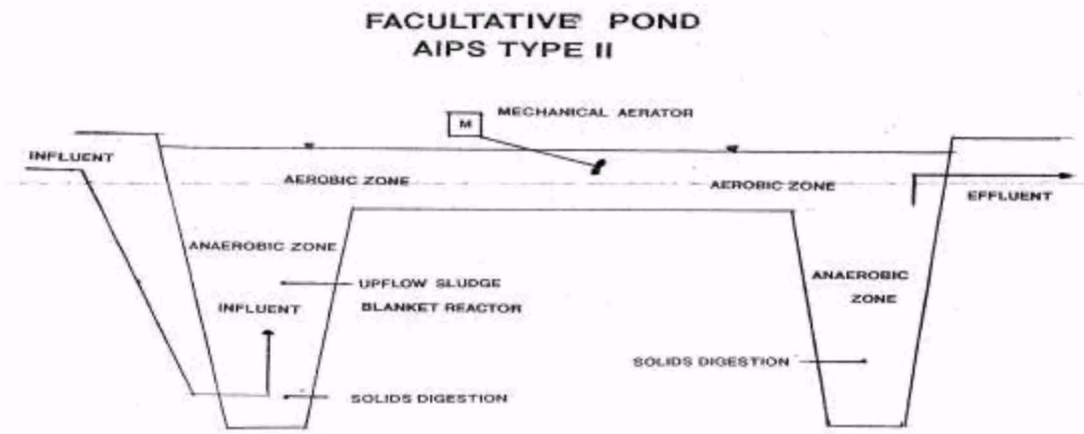


Figure 5-9B. Configuration of Pond 1B (as of 1994).

The effluent quality can vary depending on whether growth conditions are favorable for algae in the HRP and ASP, but the data for 2010 show consistently acceptable water quality. When the effluent is discharged to land, TSS is not a compliance parameter, though the high concentration of algae can reduce the effectiveness of the disinfection process. Based upon published data, the process can produce excellent quality effluent if algae are removed before the effluent is disinfected and discharged to land or ambient water.

5.2.6.2. Delhi and Hilmar, California

Delhi and Hilmar are two small towns (populations of 10,000 and 8,000, respectively) in the Central Valley, a large ancient marine lake bed that lies between the Coast Range and the Sierra Nevada. It is the site of industrial-scale agricultural enterprises that include poultry and dairy operations. Nitrogen infiltration of groundwater is a valley-wide problem. The temperature ranges from 3 °C in winter to 35 °C in summer. Rain fall is low (highly variable average: 150 mm) and is limited to the winter months. Both towns had outgrown their original treatment systems and decided to replace them with AIWPS®. Figure 5-10A and B show the plans for the Delhi and Hilmar systems.

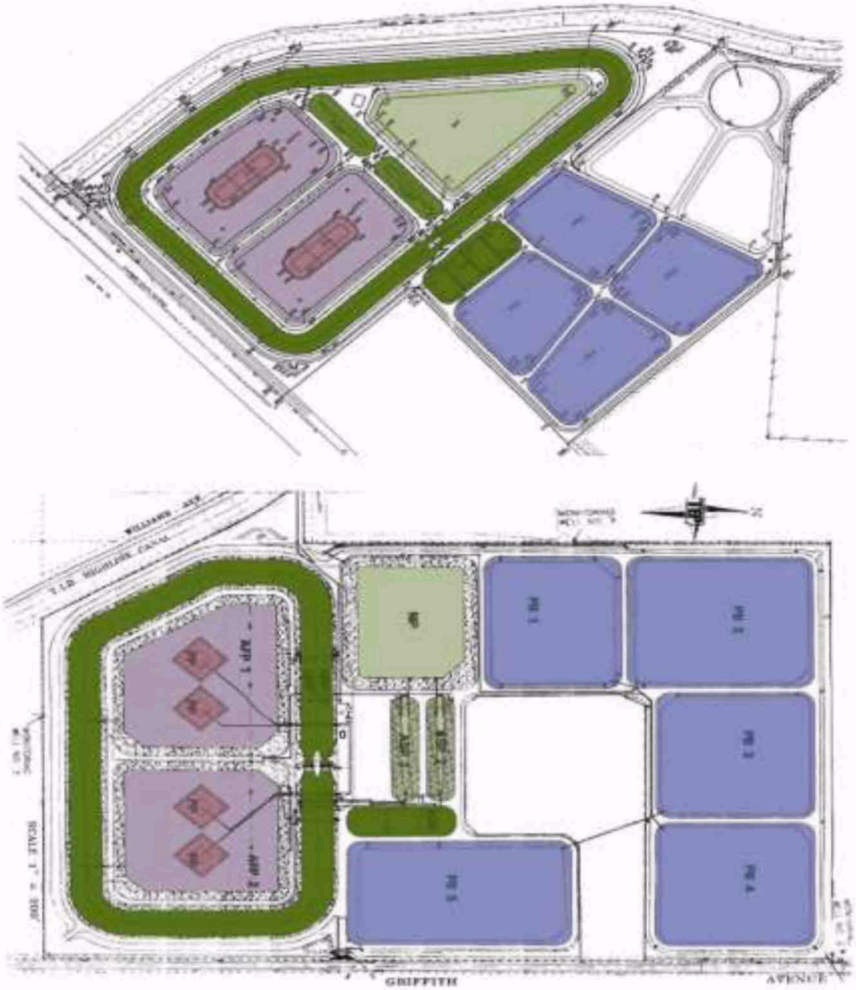
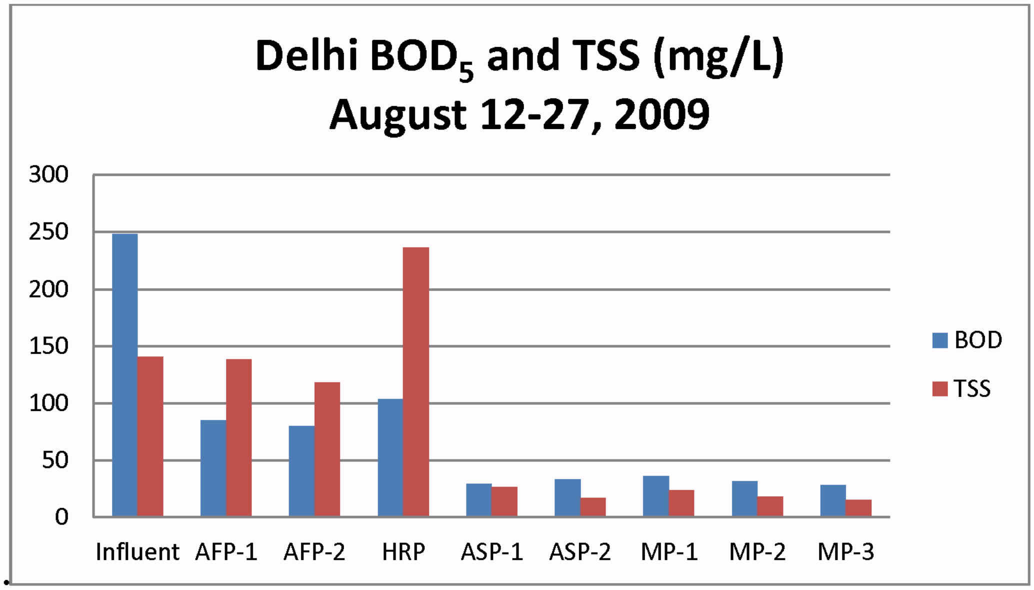


Figure 5-10A and B. A. Delhi, California AIWPS®; B. Hilmar, California AIWPS® (AFP=Advanced Facultative Pond, FP= Fermentation Pit, HRP=High Rate Pond, ASP=Algal Settling Pond, ADB-Algal Drying Bed, MP=Maturation Pond, PB=Percolation Bed).

In 2009-10, the EPA Region 9 Laboratory, Richmond, California, conducted a study at Delhi comparing dry (August) and wet (January) season treatment throughout the system. BOD_5 and TSS removal are presented in Figures 5-11A and B. Total coliform and *E. coli* results, are presented in Figures 5 – 12A and B.

A



B.

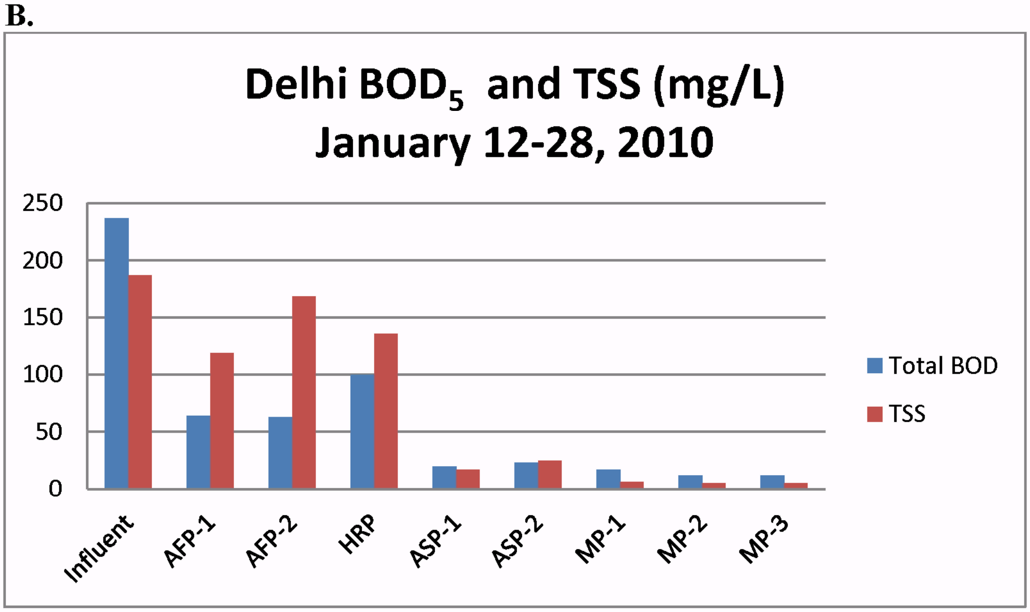
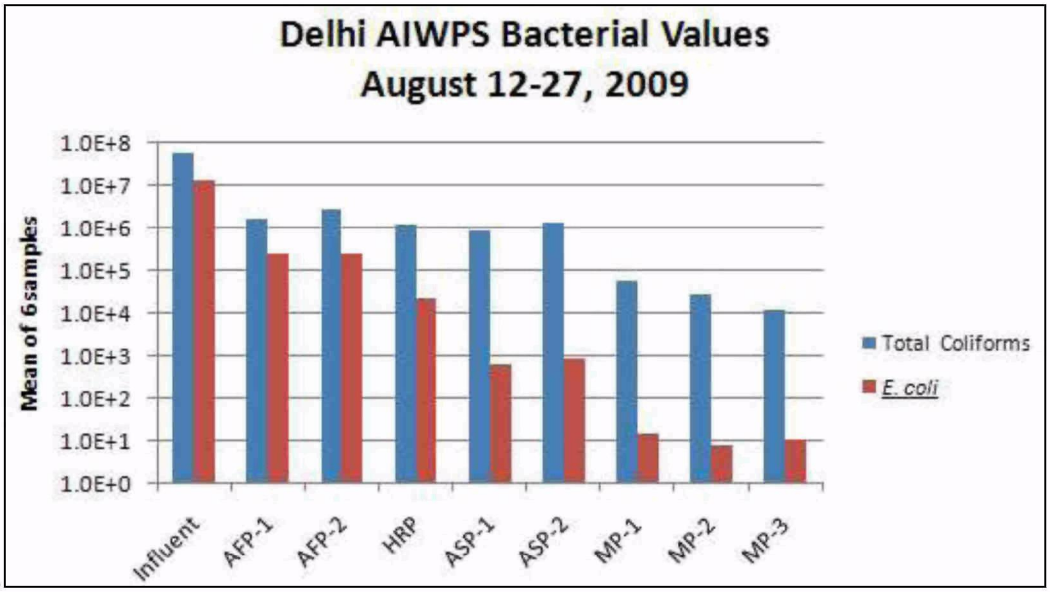


Figure 5-11A and 11B. Delhi AIWPS® BOD_5/TSS Study, 1A) August 2009, 1B) January 2010. (AFP=Advanced Facultative Pond, HRP=High Rate Pond, ASP=Algal Settling Pond, MP=Maturation Pond).

A



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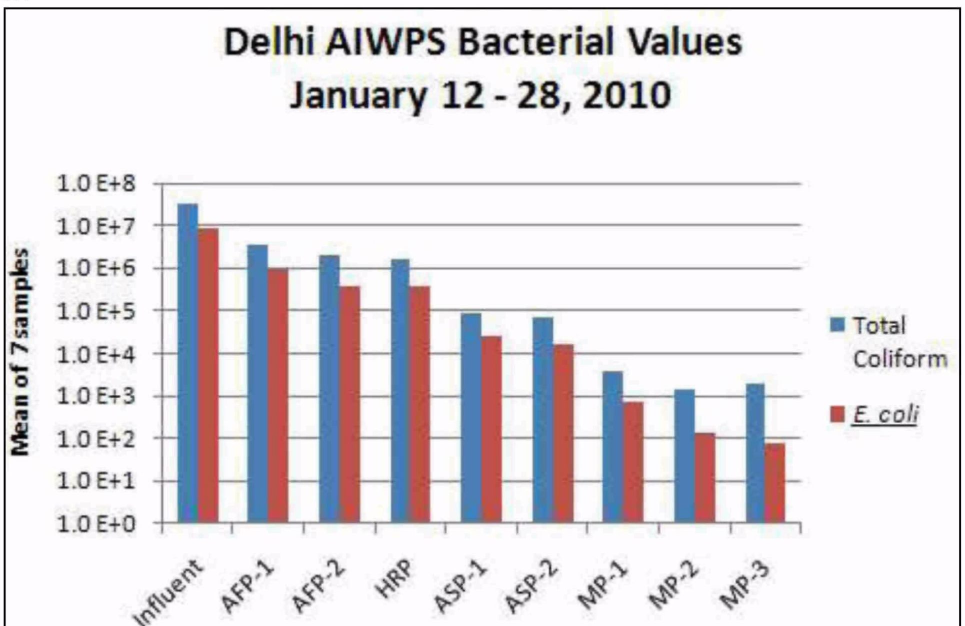


Figure 5 – 12A and B. Delhi AIWPS® Coliform Study. A. Summer, B. Winter. (AFP=Advanced Facultative Pond, HRP=High Rate Pond, ASP=Algal Settling Pond, MP=Maturation Pond).

5.2.6.3 Type II AIWPS®

A variation on the AIWPS® design, Type II, does not include the high rate pond or algae settling basin. Details of these systems are presented in Table 5-4.

Table 5-4. Type II AIWPS®

	Napa	Bolinas	Ridgemark Estates
Location in California	Napa County adjacent to the Napa River near San Francisco Bay	Marin County, Pacific Coast in the Bolinas Bay watershed	San Benito County near the City of Hollister
Climate	3-28° C	6 – 29° C	6 – 29° C
Population served	77,000	500	1,000
Date commissioned	1968	1973	1973
Type	II (4 ponds operated in series)	II (3 ponds operated in series)	II (4 ponds operated in series)
Design Capacity	12 MGD	0.03 MGD	0.1 MGD
Influent Characteristics			
Present Flow	12 MGD	0.03 MGD	0.1 MGD
BOD_5	320 mg/L	160 mg/L	220 mg/L
Type	Municipal	Municipal	Municipal
Effluent disposal and/or reuse	Discharge to Napa River and beneficial irrigation reuse	On-site disposal by spray irrigation to land and wetlands	Beneficial reuse for landscape irrigation
Total WWTF Area	360 acres	40 acres including 34 acres of on-site disposal	6 acres
Residual Solids Disposal	None	Land application of residual solids removed from Pond 1A in 2005	None
Pond Design			
Treatment Wet Surface Area	340 acres	5.4 acres	4.1 acres
Primary Facultative Pond(s)	Yes (1) with internal fermentation trenches	Yes (2) with baffle-protected fermentation pit (Ponds 1A and 1B are operated in parallel)	Yes (1)
Secondary Facultative Pond(s)	Yes (1)	Yes (1)	Yes (1)
Algae Settling Ponds	None	None	None
Maturation Pond(s)	Yes (2 in series)	Yes (1)	Yes (2 in series)
Algae Drying Beds	None	None	None
Operators	4 (part-time)	2 (part-time)	1 (part-time)

5.2.6.3.1 Bolinas, California

The Bolinas Type II AWIPS® has been in operation since 1973. The system consists of four ponds (2 facultative; 1 maturation; and 1 storage) (Figure 5-13) that occupies 1.6 ha of a 35.6 ha community-owned area. Treated effluent is land applied during the dry season (May - October) and stored in Pond 3 during the winter. Figure 5 – 14 shows the treatment of BOD_5 throughout the treatment system in 2010. The population served is approximately 500, as only part of the town is connected to the system. Restrictions on additional water connections, as well as a strong voluntary water conservation program, postponed the need to design for increased capacity indefinitely and improvements to the collection system have reduced inflow and infiltration (I&I). The plant accepts septage from the rest of the community. Pond 1A was dewatered and accumulated biosolids were treated on adjacent community-owned land in 2001. Plans are being developed to remove biosolids from Pond 1B. In 2008, solar panels were installed near the ponds system laboratory in a shade-free area facing southeast to southwest. The installation generates power to run the spray field pumps and the aerator in Pond 1B. It has been generating more energy than it uses and is expected to be paid off in 16 years.

Figure 5-13. Bolinas, California AWIPS® with adjacent spray fields, including seasonal wetlands, depressions worked into the surrounding area providing habitat for birds during the rainy season. Operation rotates the area being sprayed over the dry season. Maintenance includes monitoring condition of pipes, pumps, valves and sprinklers and harvesting emergent vegetation.

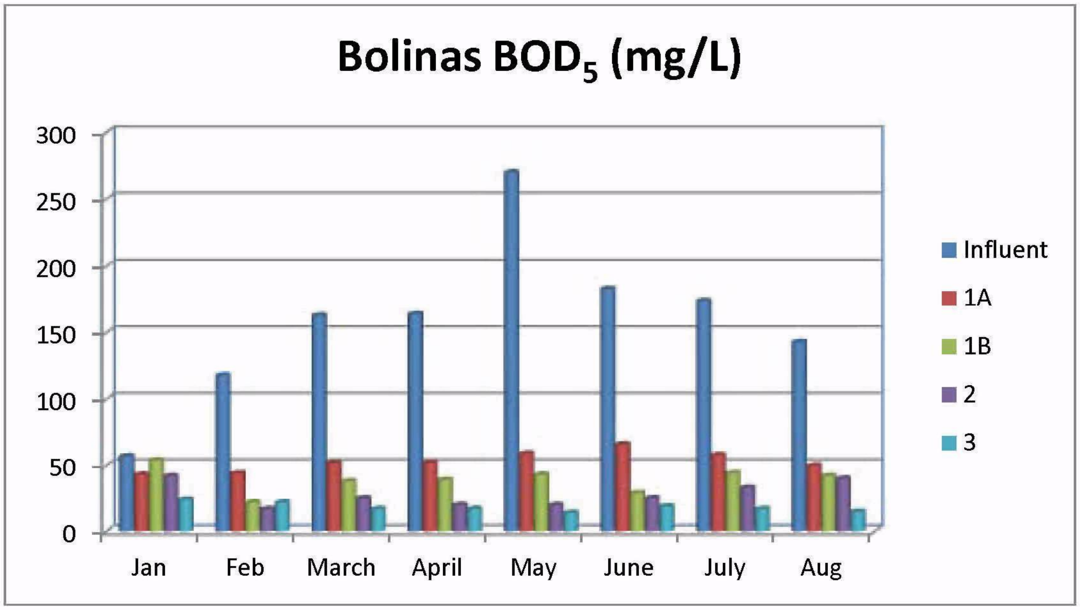


Figure 5-14. Bolinas AIWPS® BOD_5 through the system, 2010. (1A, 1B = primary; 2 = maturation pond; 3 = storage pond.)

5.2.7 Limitations

Limited data are available for cold climates.

5.2.8 Operation and Maintenance

Operation and maintenance for the AIWPS® are basically the same as that for other types of ponds. The system is not maintenance intensive and energy costs are comparable to those of a partial mix pond. Analytical work is essential to ensure proper operation, but extensive sampling and monitoring is usually not necessary. Inspection of earthen dikes is necessary to control rodent damage. See Chapter 9 for details on operation and maintenance of pond systems.

5.2.9 Costs

Costs to construct and operate the AIWPS® are lower than conventional wastewater treatment processes. Green, et al. (1995a) reported, "The overall energy savings of photosynthetic oxygenation in paddle wheel mixed HRPs are significant when compared with the energy requirements of mechanical aeration in activated sludge and extended aeration systems." Construction costs include cost of land, excavation, grading, berm construction, sealing, and inlet and outlet structures. Construction methods used are the same as with other pond systems.

5.3 SYSTEMS WITH DEEP SLUDGE CELLS

The wastewater treatment facilities presented in this section are similar to a Type II AIWPS® system, that is, they do not include the high rate algal growth and settling ponds in the system.

5.3.1 Dove Creek, Colorado

5.3.1.1 Description

Dove Creek is located approximately 48.1 km north of Cortez, CO, at an elevation of 2.1 km above sea level. Air temperatures range from -18 °C to greater than 32 °C. The wastewater treatment plant serves approximately 700 people with an average design flow rate of 173.4 L/min. The system is permitted for a design flow of 302.3 L/m and 130.6 kg of BOD_5/day.

The system has an anaerobic pond preceding the aerated cells followed by a free water surface wetland. The fermentation pit is shown in plan and cross sectional view in Figures 5-15 and 5-16, respectively. The fermentation pit has a total volume of 905 m^3.

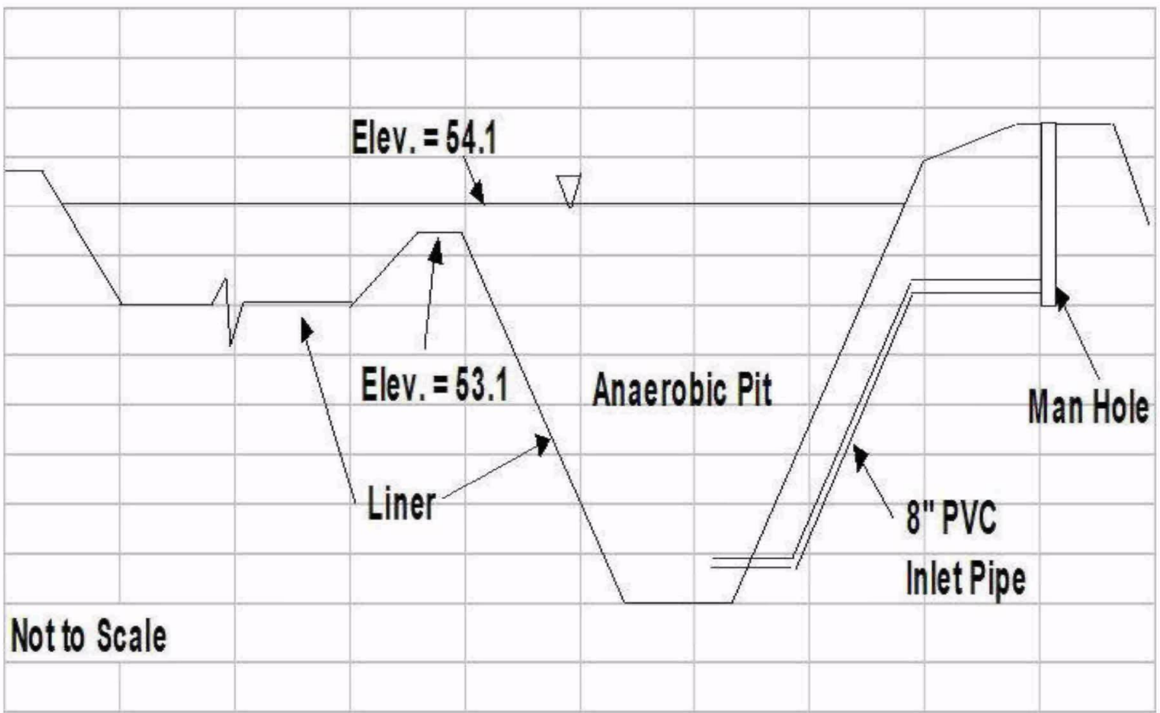


Figure 5-15. Cross-sectional view of the facultative cell at the Dove Creek, Colorado WWTP.

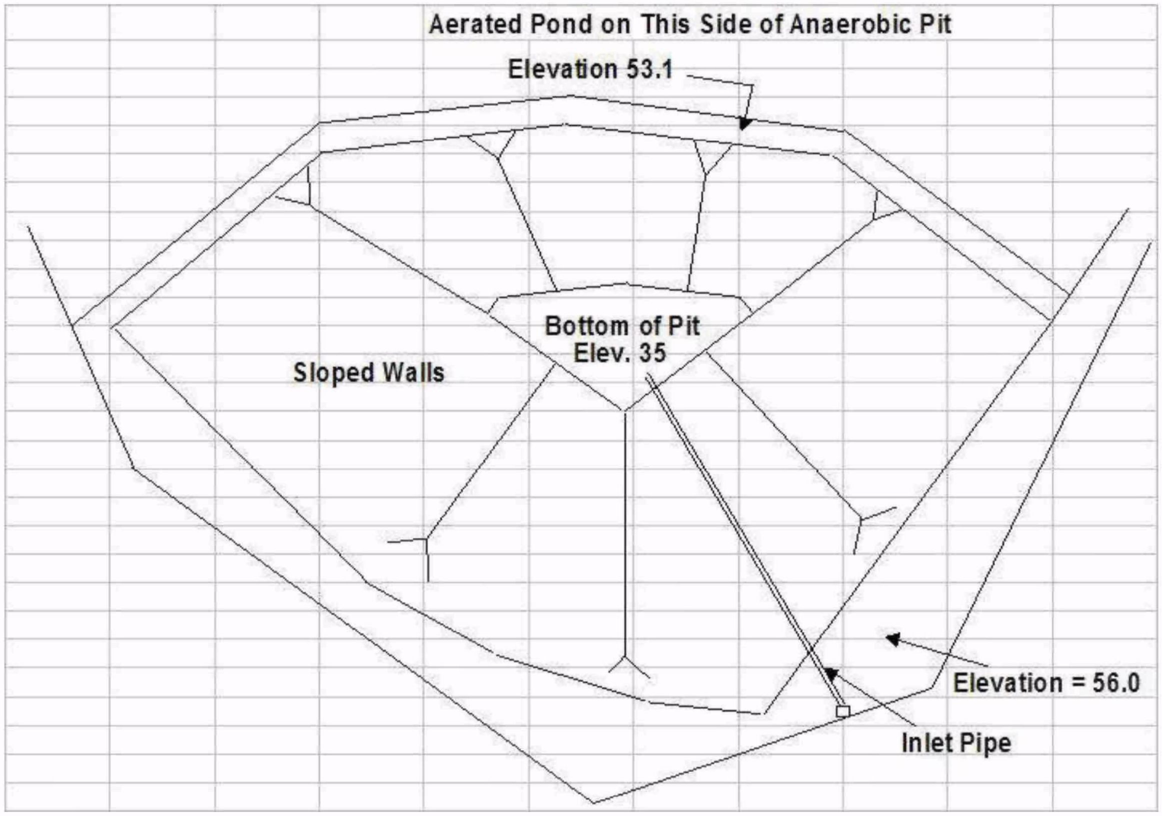


Figure 5-16. Plan view of Dove Creek Fermentation Pit (Fagan, pers. comm.).

5.3.1.2 Common Modifications

The only modification that has been made is that a wetland has been constructed after the pond system to polish the effluent.

5.3.1.3 Applicability

The system is well suited for small communities located in practically any climate. The system is effective in removing TSS, BOD_5, fecal coliform and nitrification of NH_3 during the warmer months of the year.

5.3.1.4 Advantages and Disadvantages

The primary advantage of the system is that it combines the benefits of both anaerobic and aerobic processes. By preceding the aerobic cell with an anaerobic cell, solids production is reduced and less-frequent cleaning of the settling cell is required.

One disadvantage of the system is that biological activity slows down during cold weather. Mosquito and similar insect vectors can be a problem if vegetation on the dikes and berms is not properly managed. Sludge accumulation rates will be higher in cold climates because low temperatures inhibit anaerobic reactions. Energy input is required.

5.3.1.5 Design Criteria

Systems with deep sludge cells are designed with the same criteria as those for anaerobic and aerated cells. The design criteria shown in Table 5-5 are acceptable for most environmental conditions.

Table 5-5. Design Criteria for Pond Systems with Deep Sludge Cells (from Hotchkiss, Colorado Wastewater Ponds Treatment System).

Unit Processes	Unit Process Features Description	Capacity, Hydraulic/Organic
Lagoon #1	Anaerobic portion: Vol. = 1.75 MG, Depth=18.5-21.5 ft.	315 lbs BOD_5/day
	t=3.5 days; Aerobic portion: Vol.=2.9 MG, Depth=13 ft.	
	t=5.9 days.	
Aeration	2-5 hp and 1-10 hp surface aerators, FTR=1.40 lbs O_2/hp-hr	
Lagoon #2	Vol.=5.0 MG, Depth=13 ft., t=10.0 days	
Aeration	1-5 hp and 1-10 hp surface aerators, 1-5 hp aspirating aerator. FTR=1.46 lbs O_2/hp-hr	
Polishing Pond	Vol=1.74 MG, Depth=12 ft., t=3.5 days	0.494 mgd
Recirculation	0.5 hp pump rates at 100 gpm	
Chlorination	2-150 lb. Gas cylinders, 0 to 4 lbs/day and 0 to 10 lbs/day.	
	Regulators: 2.5 mg/L maximum dosage	
Chlorine contact chamber	Serpentine Basin, Length=190 ft, Width=3.5 ft, 54:1 length to width ratio. Vol.=34,800 gallons, t=30 minutes	1.67 mgd
Effluent Flow Measuring	450 V-notch weir, h=15 inches	
Irrigation Pumping	A pump of undetermined size will be used to pump a portion of the effluent to an irrigation ditch supplying 70 acres of farm land	
Dechlorination	50_2 gas, same equipment as gas chlorination equipment	0.494 mgd

See p. xiv for conversion factors.

5.3.1.6 Performance

Performance data from January 31, 2000 to October 31, 2006 are shown in Figures 5-17 through 5-20. At the time, the system was receiving only 44 percent of the permitted flow rate. After an initial excursion, the BOD_5 removal tended to stabilize, but on occasion still exceeded the effluent standard of 30 mg/L. The effluent TSS concentrations varied rather widely. Both BOD_5 and TSS were measured in the constructed wetland following the pond system. Data were not available for the intermediate or pond system effluent.

Developing a sustainable plant crop proved to be more complicated than was expected. Therefore, it is difficult to access the performance of the various components of the system. It is reasonable to assume that the water quality exceedances were attributable to the variability encountered in the management of the constructed wetland.

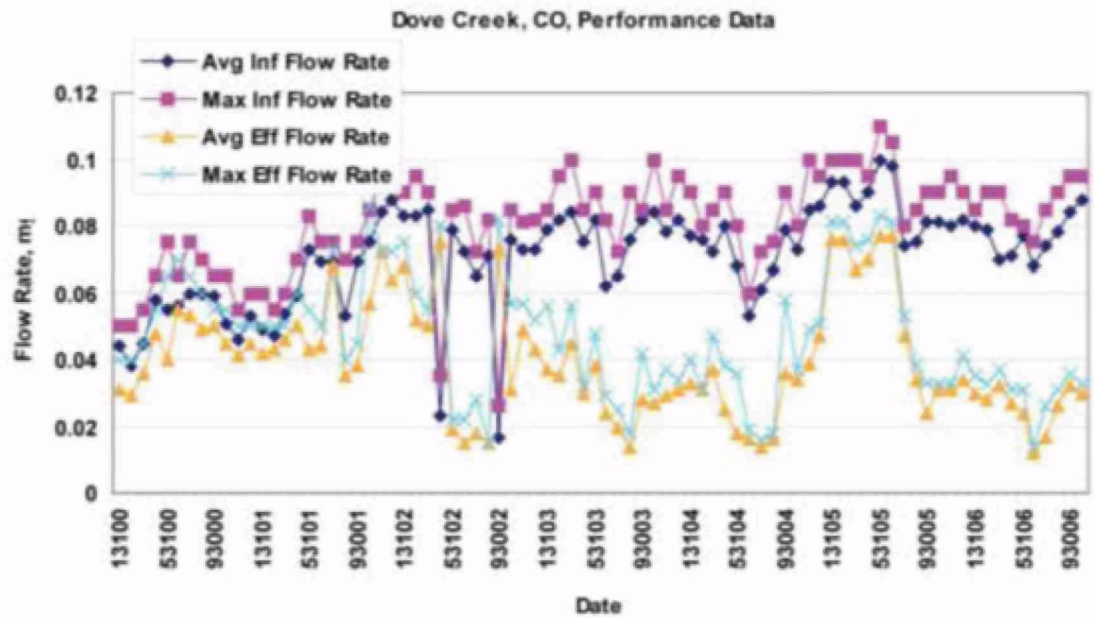


Figure 5-17. Flow rate performance data for Dove Creek, January 31, 2000 to October 31, 2006 (Colorado Department of Public Health and Environment (CDPHE), 2006).

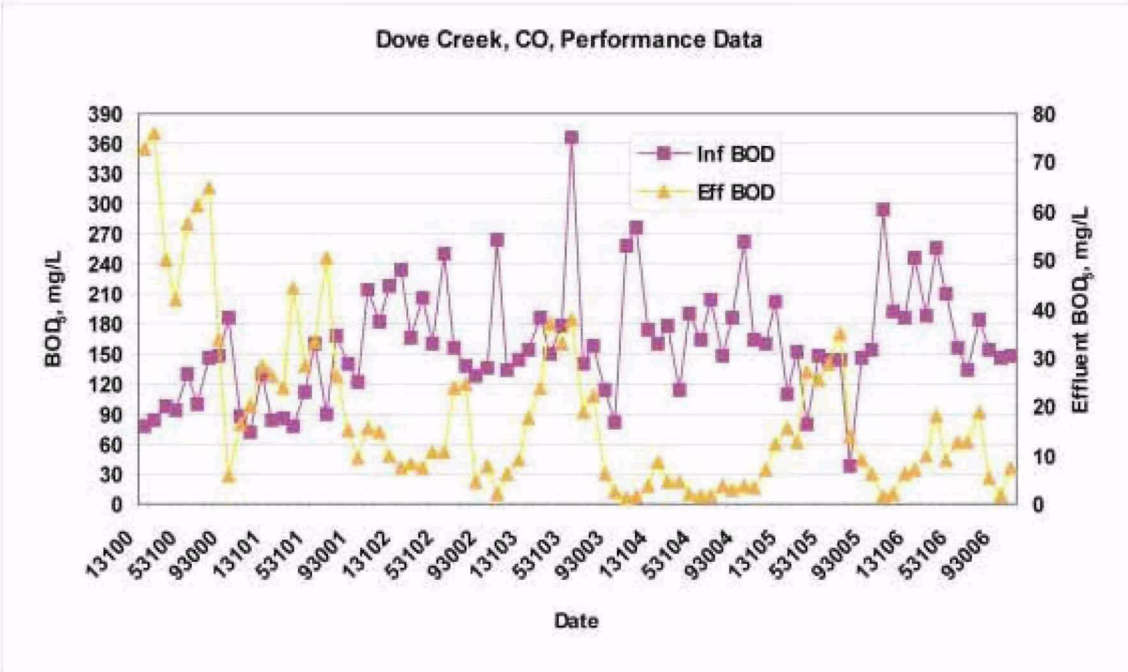


Figure 5-18. BOD_5 performance data for Dove Creek, January 31, 2000 to October 31, 2006 (CDPHE, 2006).

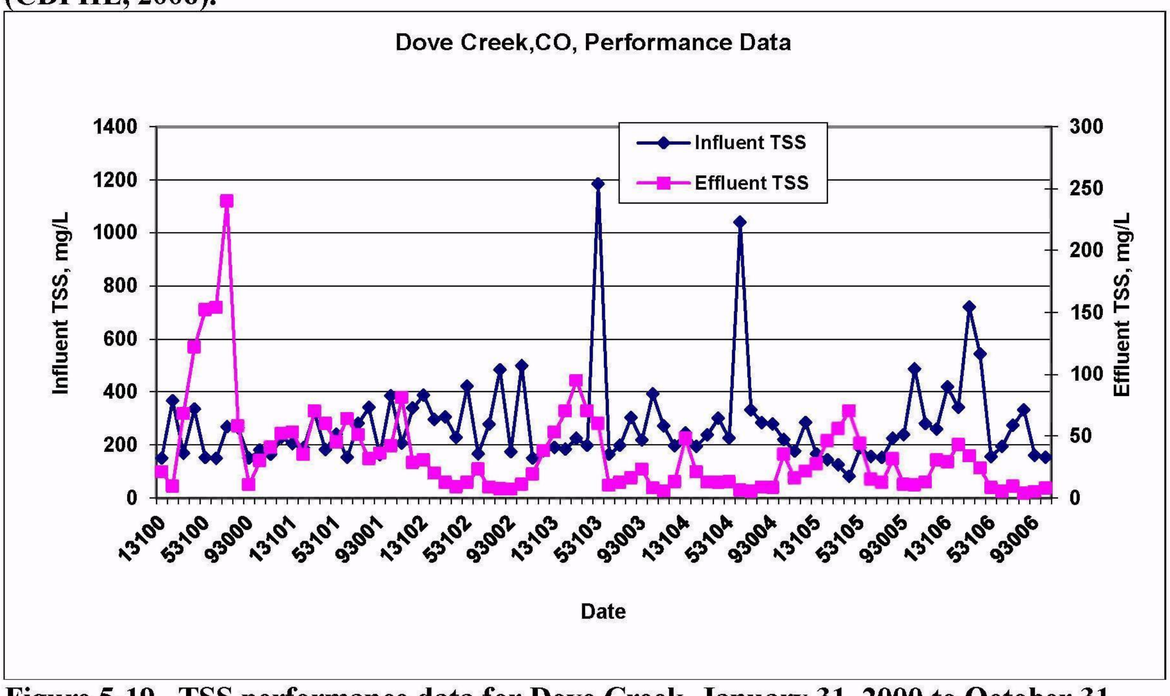


Figure 5-19. TSS performance data for Dove Creek, January 31, 2000 to October 31, 2006 (CDPHE, 2006).

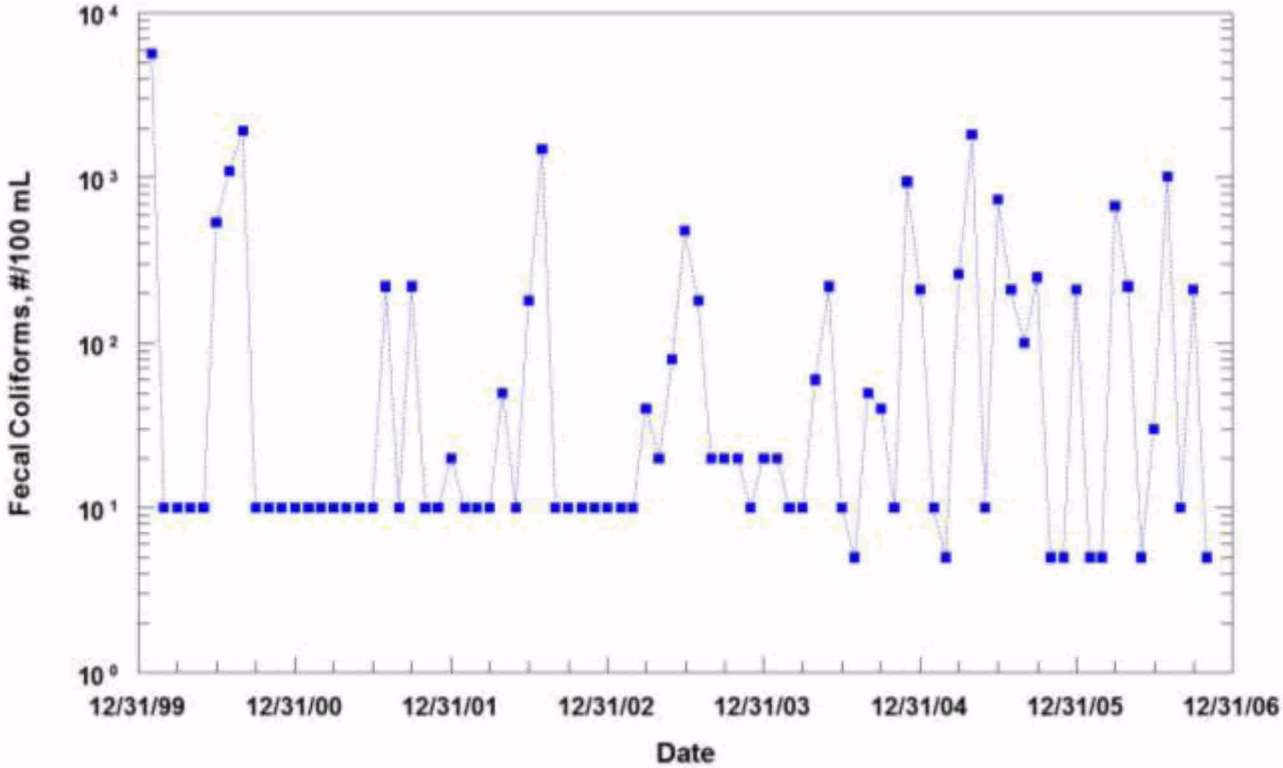


Figure 5-20. Fecal coliform performance data for Dove Creek, January 31, 2000 to October 31, 2006 (CDPHE, 2006).

5.3.1.7 Limitations

Depending upon the rate of aeration and the local climate, aerated ponds may experience ice formation on the water surface during cold weather periods. Reduced rates of biological activity also occur during cold weather. If properly designed, a system will continue to function and produce acceptable effluents under these conditions. The potential for ice formation on floating aerators may encourage the use of submerged diffused aeration in very cold climates (see Chapter 3 for a discussion of maintenance of submerged aeration devices). Any earthen structures used as impoundments must be periodically inspected as rodent damage can cause severe weakening of pond embankments.

5.3.1.8 Energy

Typically, systems are designed to flow by gravity from one pond to the next, otherwise energy will be required to keep the system flowing. Energy is also required for the aeration devices, the amount depending on the intensity of the mixing. Partial mix systems require between 1 - 2 W/m^3 per MG of capacity, depending on the depth and configuration of the system. See the design example 3-7 (#9) in Appendix C for a method of calculating the energy requirements for partial mix systems.

5.3.1.9 Costs

Construction costs associated with deep sludge ponds are similar to those with partially or completely mixed aerated ponds and include cost of the land, excavation, and inlet and outlet structures. Costs are similar to building an anaerobic pond. If the soil where the pond is constructed is permeable, an additional cost for lining should be included. Excavation costs vary, depending on whether soil must be added or removed. Compacting and synthetic lining material should be included in cost estimates. Operating costs of partially aerated ponds include electrical surface or diffused aeration equipment and maintenance of these units.

5.3.2 Fisherman Bay, Washington

5.3.2.1 Description

Fisherman Bay is located on Lopez Island in San Juan County, Washington, approximately 161 km northwest of Seattle at an elevation slightly above sea level. Influent water temperatures range from 7 $^{\circ}$C in the winter to 22 $^{\circ}$C in the summer.

The community has a septic tank effluent pumping (STEP) system that serves residences and small community commercial sites near an enclosed salt water bay with generally poor soil conditions. The system is currently rated for about 139.3 L/m.

The wastewater treatment plant, built in 1979 with a single 946.4 m^3 aerated pond was upgraded in 1995 with a second 1703.4 m^3 aerated pond operated in series with the original basin. The upgrade did not provide consistent treatment at the level needed for compliance. Since there is considerable treatment of wastewater in local septic tanks, the influent coming to the plant is low in BOD_5 and TSS, but has a high NH_3 concentration, averaging 57 mg/L (Li et al., 2006).

In 2000 Sear Brown Engineering (now Stantec) performed a plant evaluation that recommended upgrading the pond system to an AIWPS-like based on research and process theory published by Dr. Oswald (see Section 5.2) and Dr. Michael Richards. In 2003, an anaerobic cell was built to pretreat the STEP system influent with an anaerobic methane bacteria process. This reduced the carbon load to the larger aerated pond, which was partitioned into three cells to provide a settling cell. The fermentation pit has a total volume of 314 m^3 and a depth of 4.57 m. Within the 4.57 m depth is a manhole pit at the bottom 1.5 m deep and 1.8 m in diameter from which the influent enters the pond (Figure 5-21). The original, smaller, shallower pond was taken off line, re-excavated and relined as a storage and surge pond for future water reuse.

Since these modifications were made, the district has not had violations but there were still problems dealing with the seasonal algae and biomass blooms that made disinfection difficult.

The last part of the system, the constructed wetland, was built to use the nutrients in the pond effluent. The effluent quality is now consistently high year round, including low bacteria residuals. Future plans include reusing the plant effluent (Geoffrey Holmes, pers. comm., 2010).

5.3.2.2 Modifications

The modification to this system was to add the anaerobic pond with the deep sludge pit to an existing system of two ponds. Other modifications to the existing ponds were included in the 2003 up-grade of the system. In 2006, a wetland was constructed through which the effluent from the last pond flows.

5.3.2.3 Applicability

The system is well suited for small communities located in practically any climate. The system is effective in removing TSS, BOD_5, fecal coliform and nitrification of NH_3 during the warmer months of the year.

5.3.2.4 Advantages and Disadvantages

See Section 5.3.1.4.

5.3.2.5 Design Criteria

Systems with deep sludge cells are designed using the same criteria for anaerobic and aerated cells. The design criteria shown in Table 5-5 are valid for most environmental conditions.

5 .3.2.6 Performance

The system has functioned well, as shown in Figures 5-22 through 5-26. The average flow rate entering the plant in the years 2003 - 2006 was 45.5 L/m (0.0173 mgd), approximately 50 percent of the design flow of 89.3 L/m (0.034 mgd). In this period, BOD_5 removal has averaged 86.5 percent, $CBOD_5$ removal 88.9 percent, TSS removal 34.8 percent (Max = 69.3, Min = 6.3), and NH_3 removal 56.0 percent (Max = 76.0, Min = 25.2). Ammonia removal is positively correlated with the effluent water temperature. The plant $CBOD_5$ removal failed to achieve 85 percent removal six times out of the 86 samples analyzed; whereas BOD_5 removal failed 32 out of 113 samples. This satisfied the standard of 85 percent removal 93.0 percent of the time, while the BOD_5 removal satisfied the standard only 71.6 percent of the time. The failure to meet the 85 percent removal standard can be attributed to the weak sewage entering the facility from on-site septic systems. The addition of a constructed wetland using a substrate of tire crumbs and gravel in 2007 has reduced $CBOD_5$ by 79 percent, TSS by 88 percent, fecal coliform by 97 percent and total residual chlorine below 0.75 mg/L consistently and below 0.25 mg/L the majority of the time (Li and Holmes, 2010).

Figure 5-21. Fisherman Bay, Washington. Anaerobic cell with recirculation manifold using an aerated cap of polishing cell effluent to provide odor control.

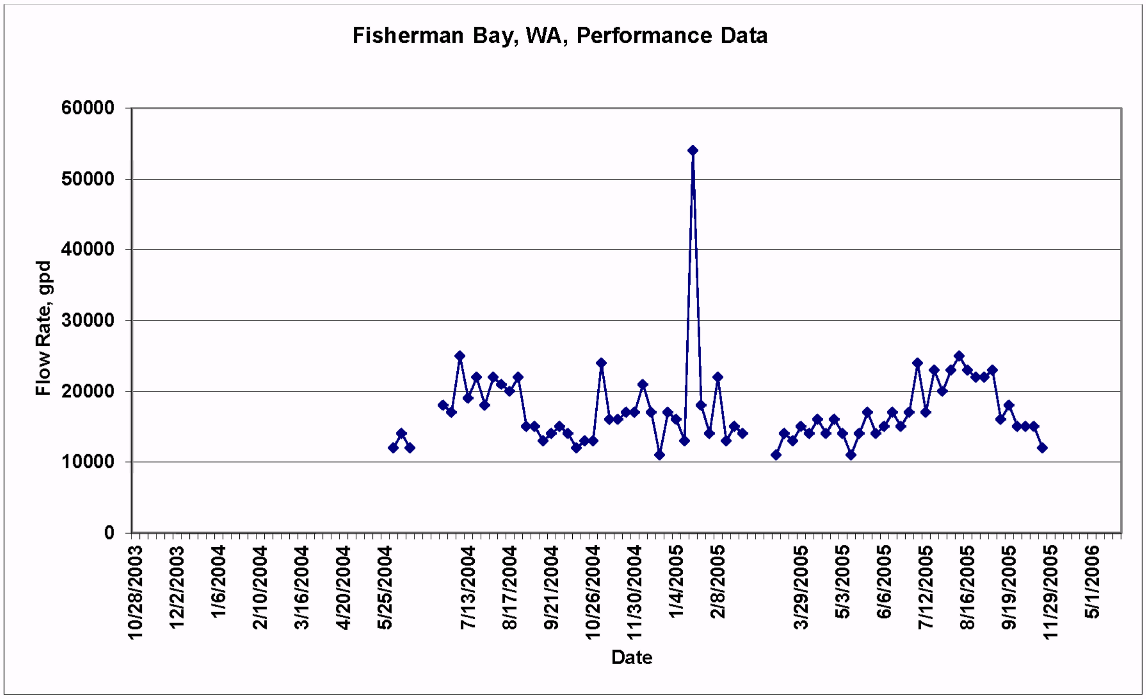


Figure 5-22. Fisherman Bay, Washington flow rate data for October 28, 2003 through August 29, 2006 (Li et al., 2006).

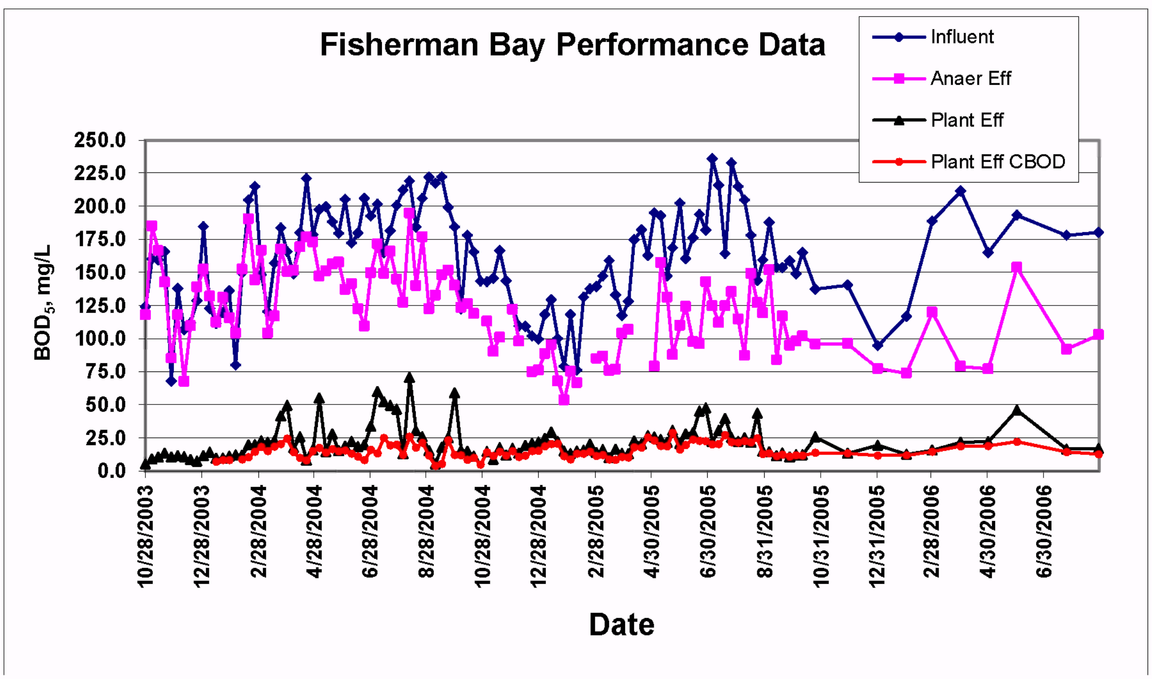


Figure 5-23. Fisherman Bay. BOD_5 performance data for October 28, 2003 through August 29, 2006 (Li et al., 2006).

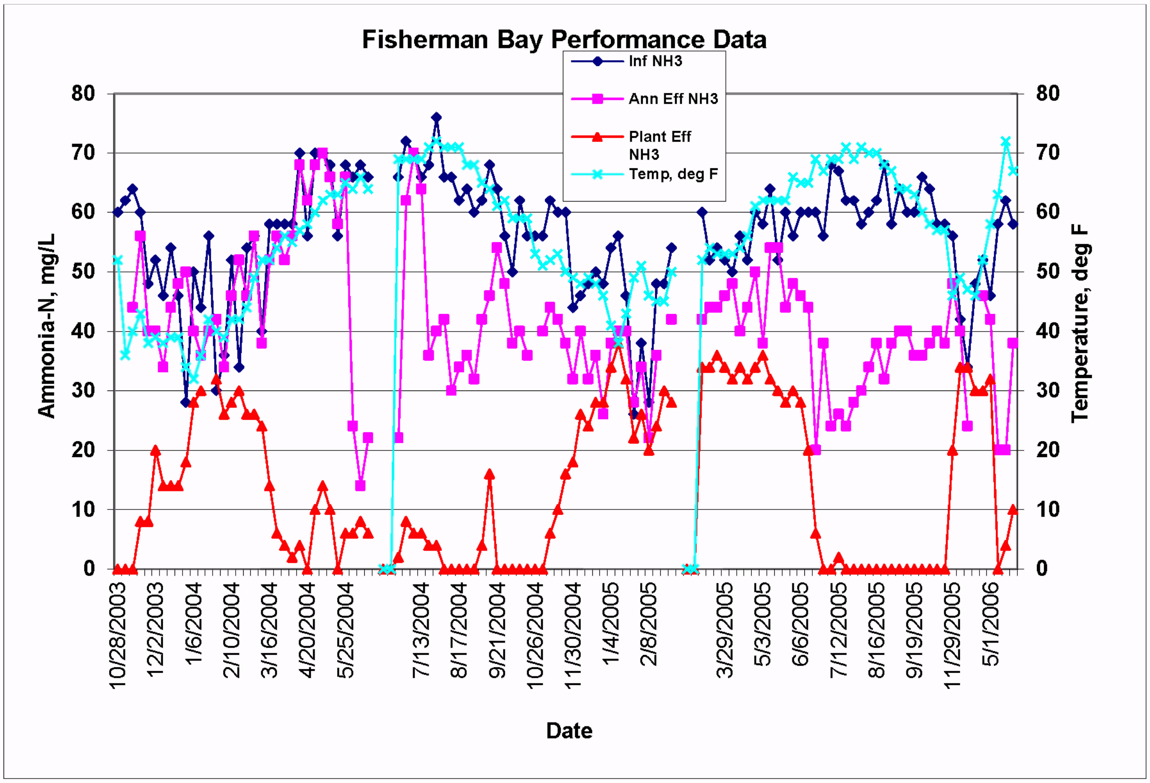


Figure 5-24. Fisherman Bay. ***NH_3*** **performance data for October 28, 2003 through August 29, 2006 (Li et al., 2006).**

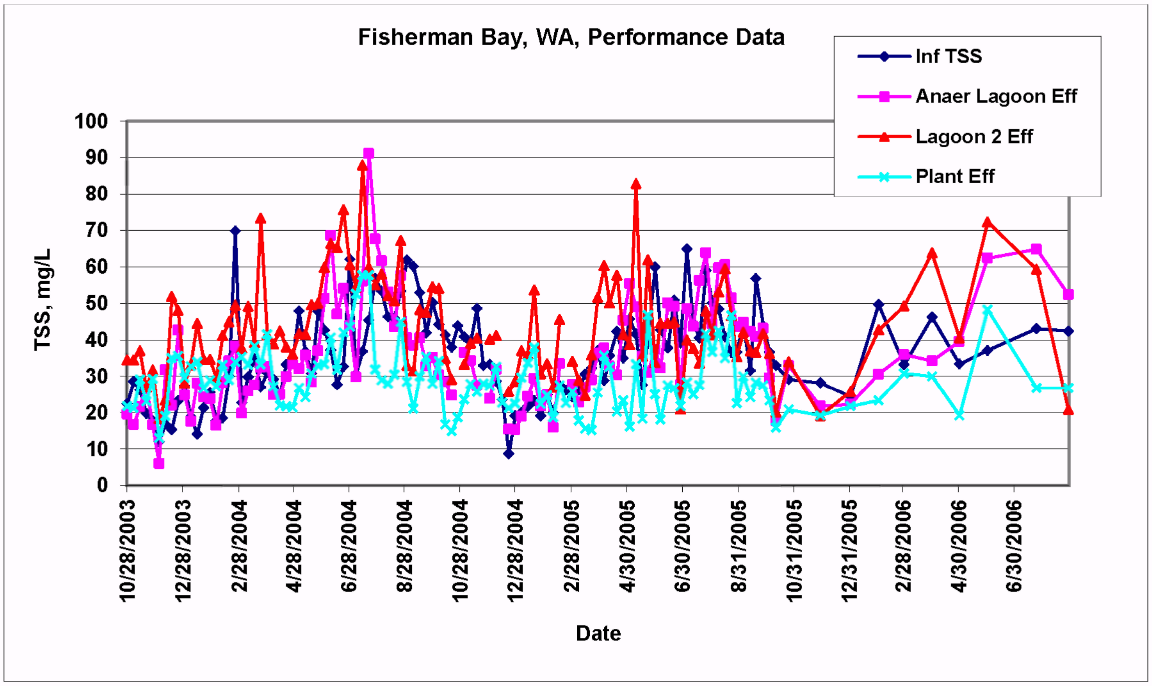


Figure 5-25. Fisherman Bay. TSS performance data for October 28, 2003 through August 29, 2006 (Li et al., 2006).

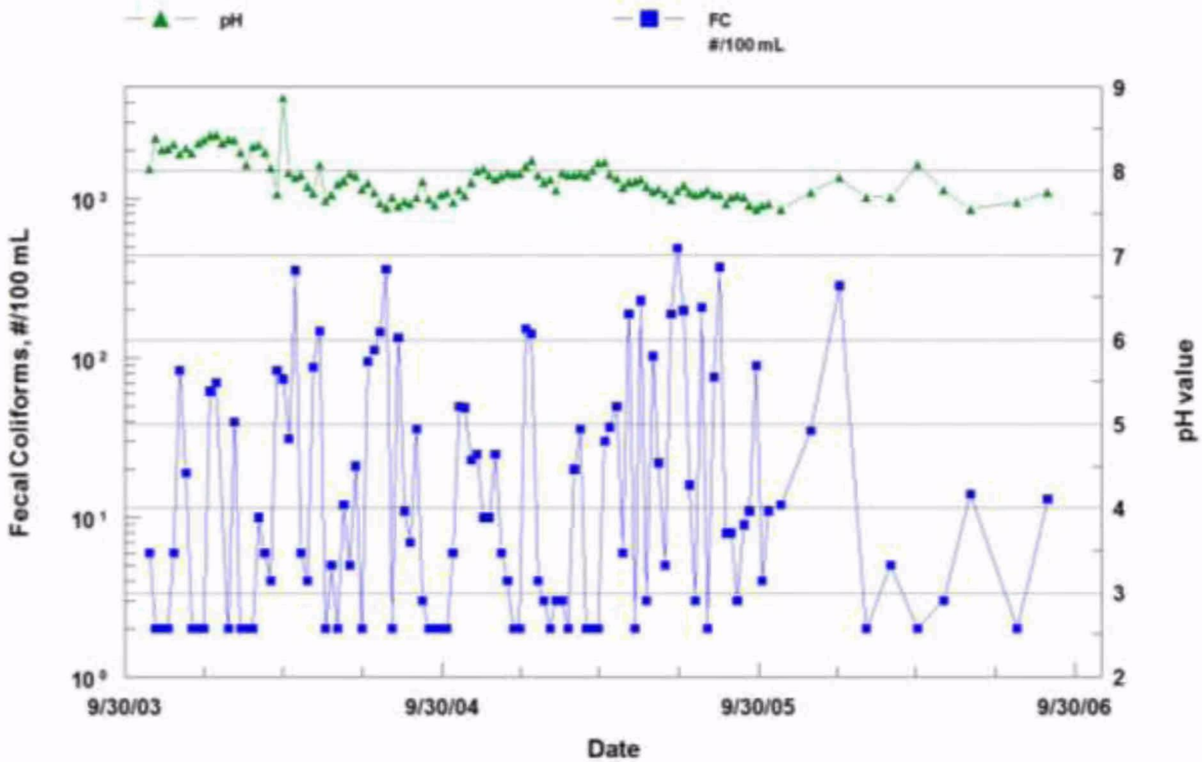


Figure 5-26. Fisherman Bay. Fecal coliform and *p*H performance data for October 8, 2003 through August 29, 2006 (Li et al., 2006).

5.3.2.7 Limitations

See "Limitations," Section 5.3.1.7.

5.3.2.8 Energy

See "Energy," Section 5.3.1.8.

5.3.2.9 Costs

See "Costs," Section 5.3.1.9.

5.4 HIGH-PERFORMANCE AERATED POND SYSTEM (RICH DESIGN)

The high-performance aerated pond system (HPAPS) described by Rich (1999) has been referred to in the literature as dual-power, multi-cell systems (DPMC). The system consists of two aerated basin in series. Screens to remove large solids precede the system. A reactor basin for bioconversion and flocculation is followed by a settling basin dedicated to sedimentation, solids stabilization and sludge storage. Algal growth is controlled by limited hydraulic retention time and division of the settling basin into cells in series. Disinfection facilities follow the settling basin (Figure 5-27).

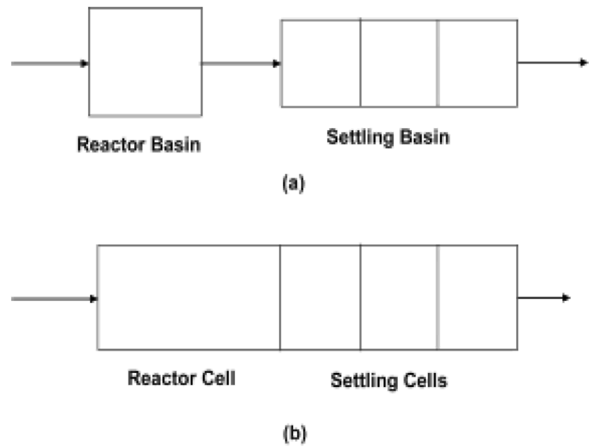


Figure 5-27A and B. Flow diagrams of DPMC Aerated Pond System. A. Two basins in series utilizing floating baffles in settling cells. B. A single basin using floating baffles to divide various unit processes. (Rich, 1999)

Design procedures are available for the HPAPS system and are illustrated in Example 3-6 (Appendix C).

5.4.1 Performance

Several sets of performance data for the HPAPS systems are available; all are for locations in mild climates, such as South Carolina and Georgia.

Performance data for the DPMC system in Berkeley County, South Carolina are presented in Figure 5-28. Data are for six years of operation; the system appears to be functioning as designed. Sludge removal data were not available.

Continuous operation of the aeration system is essential to obtain maximum efficiency as illustrated by Figure 5-29. The Berkeley County performance data were taken from a system using continuous aeration, while the data for the performance of a similar system in South Carolina from a system using aeration 50 percent of the time. Results in the second case were improved by about 50 percent by changing to a continuous aeration operation.

A DPMC system (design flow = 12,870 m^3/d) followed by an intermittent sand filter located at the Ocean Drive plant in North Myrtle Beach, South Carolina has been in service for over twelve years and has performed very well, (Figure 5-30). A flow diagram for the system is shown in Figure 5-31. Final effluent TSS concentrations have not exceeded 15 mg/L. In October 1997, EPA, Region 4 collected two 24-hr composite samples from the DPMC aerated ponds. The data from this evaluation are presented in Table 5-6 (Rich, 1999). A similar plant, the Crescent Beach plant at Myrtle Beach, also performed well as shown in Figure 5-32.

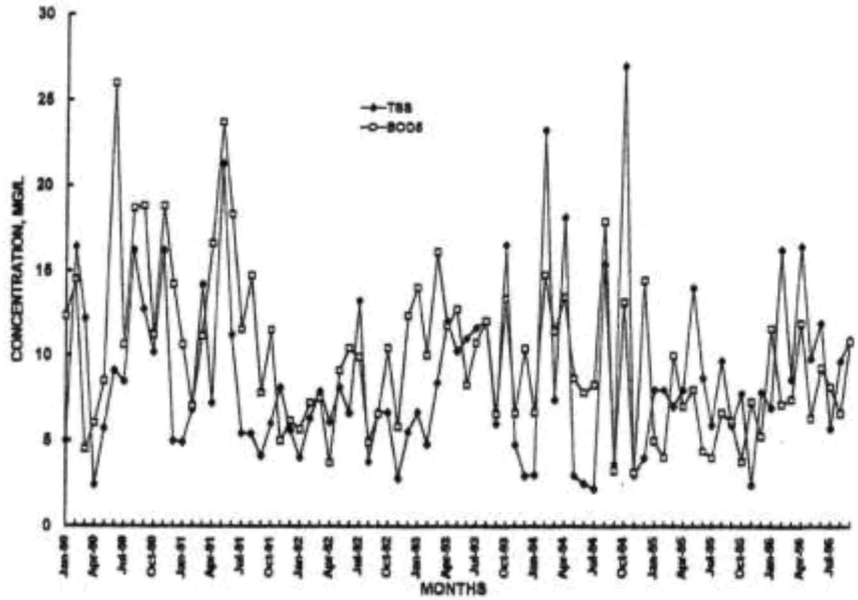


Figure 5-28. Performance of DPMC Aerated Pond System in Berkley County, South Carolina, aerators operating continuously (Rich, 1999).

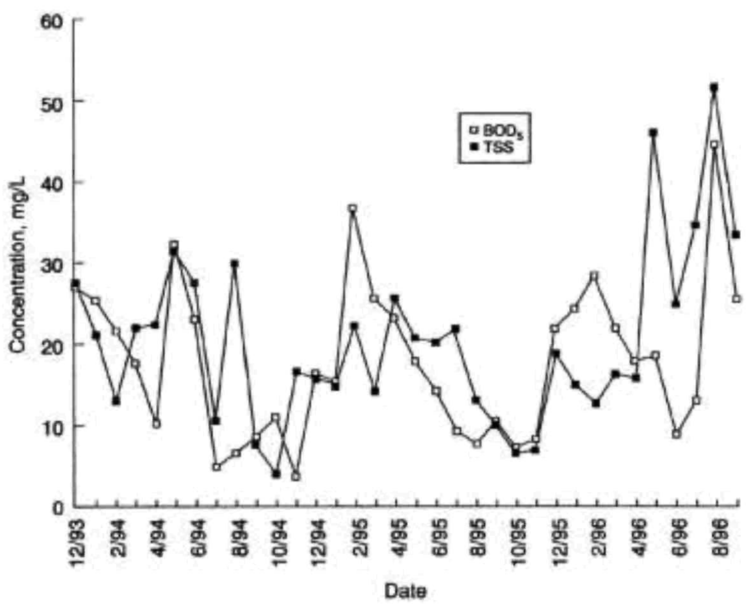


Figure 5-29. Effluent TSS and BOD_5 from a DPMC Aerated Pond System, aerators operating intermittently (Rich, 1999).

Table 5-6. Performance of DPMC Aerated Pond at North Myrtle Beach (Rich, Bowden, and Henry unpublished data, personal comm.).

	Influent	Effluent				
		Aerated Reactor A1	Aerated Reactor A2	Settling Pond A4	Settling Pond B4	Intermittent Sand Filter
Parameter (mg/L)						
BOD_5	160	21	23	10	12	2
$CBOD_5$	165	16	20	8	6	1
$SCBOD_5$	62	5	5	4	4	1
TSS	185	79	77	8	4	4
Alkalinity	195	190	190	210	220	17
NH_3	25	25	28	31	30	1
NO_3	0.07	0.05	0.05	0.09	0.44	32
TK*N*	37	35	40	34	33	2
T*P*	5.9	2.8	3.3	0.6	1.2	0.8
Chl α	-	-	-	0.036	0.043	-

$CBOD_5$ = Carbonaceous BOD_5
$SCBOD_5$ = Soluble $CBOD_5$

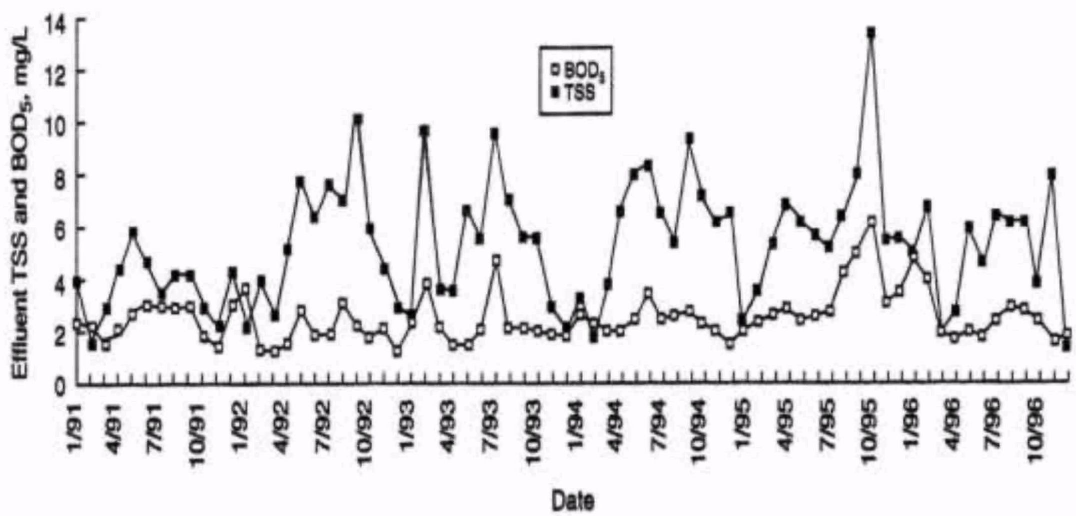


Figure 5-30. Monthly Average BOD_5 and TSS from Ocean Drive, North Myrtle Beach (Rich 1999).

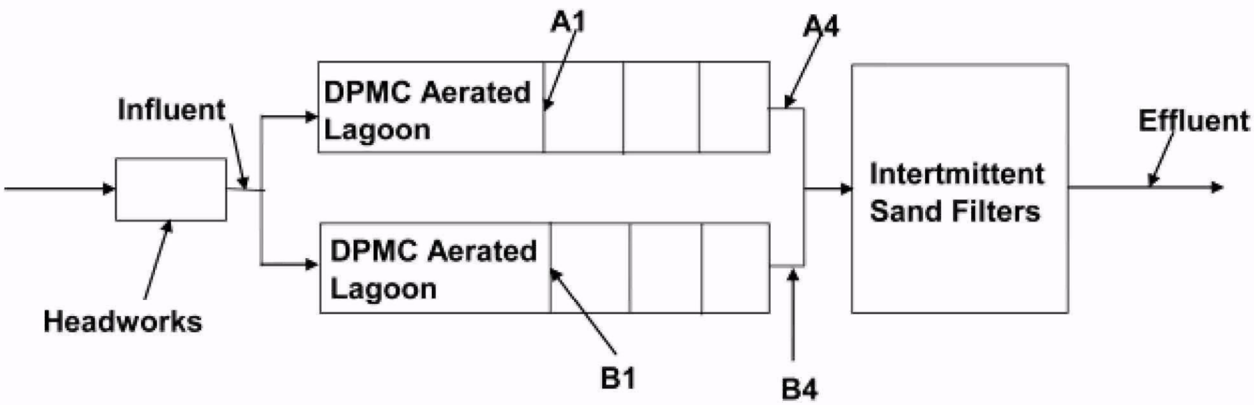


Figure 5-31. Sketch of a DPMC Aerated Pond-Intermittent Sand Filter System at North Myrtle Beach (Rich et al., pers. comm.).

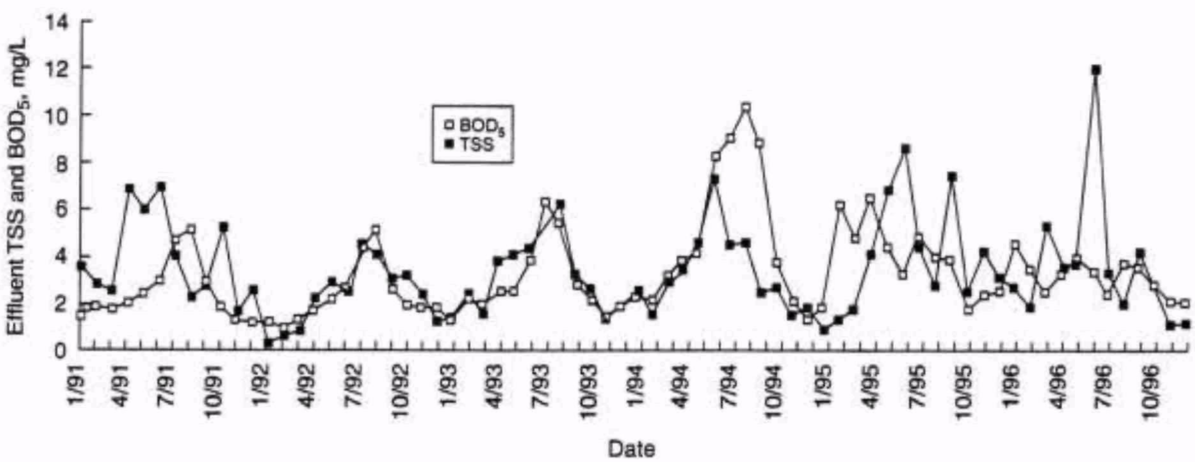


Figure 5-32. Monthly average effluent BOD_5 and TSS at Crescent Beach, North Myrtle Beach (Rich, 1999).

Having collected 36 months of data, Rich and Rich (2005) compared the performance of three DPMC-Intermittent Sand Filter systems with three carousel-extended aeration type systems (Table 5-7). The performance of the DPMC-intermittent sand filter (-IS) systems approached that of the carousel-extended aeration type systems with tertiary rapid sand filtration. Ammonia removal permit requirements were met in the intermittent sand filter facility in winter and summer, but the carousel systems reduced the concentration to less than 1 mg/L at the 90 percentile value. The DPMC-IS systems effluent 90 percentile value ranged from 1.66 to 1.81 mg/L. Capital and operational costs of the DPMC-IS systems were approximately 74 and 61 percent, respectively, less than the costs of the carousel-type systems.

Table 5-7. Comparison of Pond-Intermittent Sand Filter Systems with Carousel-Extended Aeration Systems (Rich and Rich, 2005).

	DPMC-Intermittent Sand Filter Systems			Carousel-Extended Aeration Systems			
	Ocean Drive	Crescent Beach	Hampton	Page Creek	Yellow River	Jackson Creek	Beaver Run
	2001-2004	2001-2004	2002-2005	2001-2004	2001-2004	2001-2004	2001-2004
Flow, mgd							
Mode	1.4	1.1	0.8	0.4	8.6	2.9	4.1
Max	3.1	1.8	1.3	0.6	11	3	4.5
Permit	3.4	2.1	2.0	1.0	12.0	3.0	4.5
Max/Permit	0.91	0.86	0.65	0.60	0.92	1.00	1.00
TSS, mg/L							
50%[2]	1.66	1.35	1	8.81	1.69	1.84	2.19
90%[3]	5.84	3.20	1	13.82	2.56	2.55	3.90
Permit	30	30	30	30	10	20	20
90%/Permit	0.19	0.11	0.03	0.46	0.26	0.13	0.20
BOD_5, mg/L							
50%	2.06	2.34	2.6	3.31	1.27	2.05	1.54
90%	3.33	3.98	3.85	4.86	1.95	3.34	2.64
Permit	10	10	10	30	30	10	10
90%/Permit	0.33	0.40	0.38	0.16	0.06	0.33	0.26
NH_3-N, mg/L							
50%	0.67	0.44	1.70	0.02	0.15	0.15	0.13
90%	1.81	1.66	1.88	0.11	0.28	0.31	0.25
Permit	W10, S2[4]	6	W4.2, S1	20	1	2	2
Violations	0	0	0	0	0	0	0

[1]1mgd=3,785 m^3d
[2]50 percentile value
[3]90 percentile value
[4]W-winter value, S=summer value

5.4.2 Limitations

The only performance data for the HPAPS systems available are for locations in mild climates.

5.4.3 Operation and Maintenance

As with most ponds systems, the HPAPS is not maintenance intensive, but more maintenance is required than for facultative ponds. Operation and maintenance is similar to that required for other complete mix systems. See Chapter 9 for operation and maintenance requirements.

5.4.4 Costs
Cost information for all pond systems varies widely (see Chapter 8).

5.5 BIOLAC® PROCESS (ACTIVATED SLUDGE IN EARTHEN PONDS)

5.5.1 Description
EPA published an excellent summary of the status of the BIOLAC® processes in the U.S. (U.S. EPA, 1990). Information from that report is presented in this section. Additional information was provided by the Parkson Corporation (2004) (http://www.parkson.com/main.aspx).

There are several variations of the BIOLAC® process. Basically, the processes are extended aeration activated sludge systems with and without recirculation of solids. There are three basic systems: (1) BIOLAC-R, an extended aeration process with recycle of solids; (2) BIOLAC®-L, an aerated pond system without recycle of solids; and (3) BIOLAC® Wave Oxidation Modification (WOM) used to nitrify and denitrify wastewater. In addition to these systems, floating aeration chains have been installed to upgrade existing pond systems.

The BIOLAC®-R system is shown in Figure 5.33. It is an extended aeration process operating within earthen embankments or other types of structures.

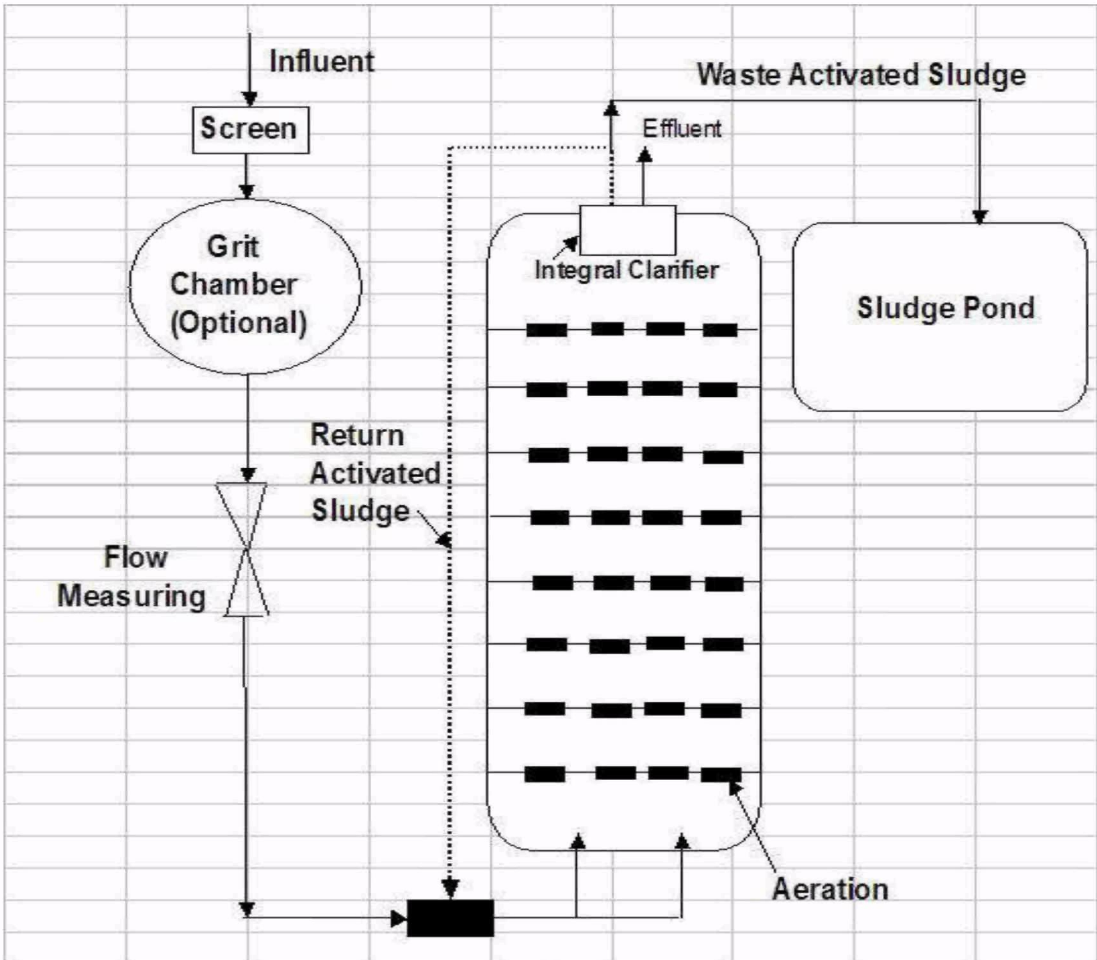


Figure 5-33. Flow diagram of BIOLAC®-R System (Parkson Corp., 2004).

5.5.2 Common Modifications

5.5.2.1 BIOLAC®-L System

The BIOLAC®-L system is a typical flow-through aerated pond without recycle of solids and a waste sludge pond. The flow diagram is the same as that shown in Figure 5-33 minus the clarifier and sludge pond. Sludge storage and decomposition occur in the polishing pond.

5.5.2.2 Wave Oxidation Modification

Carbon oxidation and nitrification-denitrification occur in the Wave Oxidation Modification system (Figure 5-34). This is a BIOLAC®-R system operated at low DO concentrations and automatic control of the airflow rate in each aeration chain. Airflow is alternated such that several moving oxic and anoxic zones are created in the aeration basin. This modification has been used successfully for *N* removal.

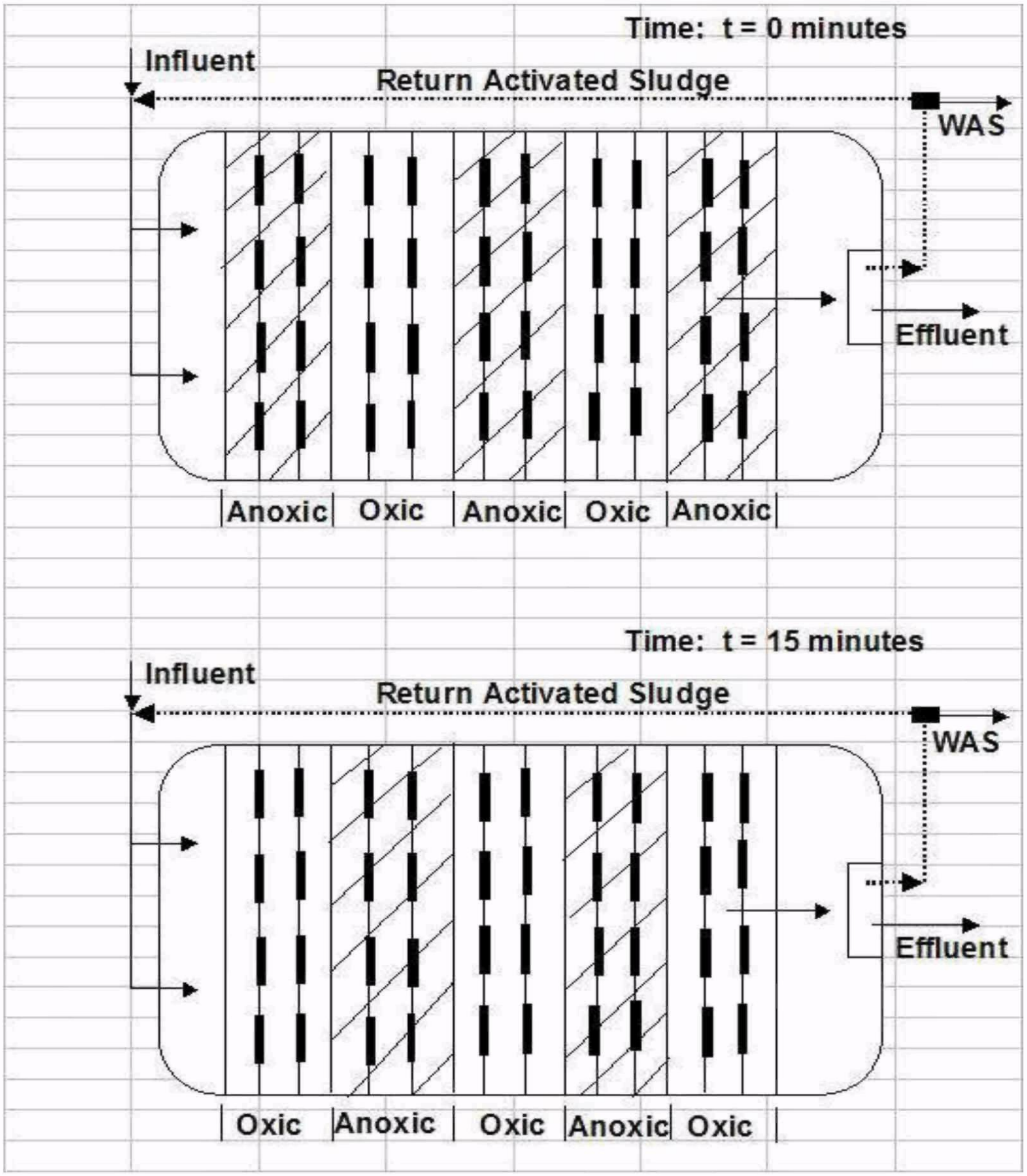


Figure 5-34. Wave-Oxidation modification of the BIOLAC®-R System (Parkson Corp., 2004).

5.5.2.3 Other Applications

BIOLAC® floating aeration chains are used as retrofits for existing ponds and installed as original aeration equipment. Several operations around the country are using BIOLAC® aeration equipment.

5.5.2.4 Applicability

The concepts are suited to any size of system, depending upon the requirements for the municipality or industry. When used as a conventional partial mix or complete mix pond system, the concept is generally limited by the same constraints imposed on other aerated pond systems. The processes are very flexible and are applicable at any location where activated sludge, partial mix or complete mix ponds are acceptable.

5.5.3. Advantages and Disadvantages

The advantage of the BIOLAC® systems is that they are very flexible and can be designed as a typical partial mix or complete mix pond system to achieve BOD_5 removal using the patented aeration system. The system can also be designed to operate as an extended aeration activated sludge system to nitrify NH_3 or to achieve nitrification and denitrification.

The disadvantages of the BIOLAC® systems are that operation and maintenance are more complex, but no more than that required for conventional extended aeration activated sludge entails when nitrification or denitrification are required. Sludge bulking occurred, but was corrected by inserting a selector section (anoxic section) at the head of the aeration tank**.**

5.5.4 Design Criteria

Recommended design criteria are shown in Table 5-8. Conservative design parameters are used, and loadings typically are 0.11 - 0.16 kg/d/m^3 BOD_5 in the aeration pond with food to microorganism ratios of 0.014 - 0.045 kg/kg of mixed liquor suspended solids (MLSS). The average loading rate for 25 BIOLAC®-R plants reported (U.S. EPA, 1990) was 0.12 kg BOD_5/m^3. The average relationship between the aeration basin volume and the number of diffusers used for the 25 BIOLAC®-R plants was 385 diffusers/mg with an airflow rate of 0.01 scm/min/m^3. The actual operating hp at the 25 BIOLAC®-R plants averaged 34 kW/MG for fully nitrified effluent. The average kW usage is not significantly different from other complete mix systems.

HRTs range from 24 - 48 hours with solids retention times of 30 - 70 days. Preliminary and primary treatment is normally not provided, but screening of the influent is desirable. Depths in the aeration ponds range from 2.5 - 6.1 m. Because sludge production is expected to be low, a relatively small waste sludge tank is provided.

Design of the BIOLAC®-L system is based on HRT, with values ranging from 6 - 20 days. Equivalent loadings of 0.008 to 0.029 kg $BOD_5/m^3/d$ are used. A polishing pond is required for the BIOLAC®-L system and has a HRT of 2 - 4 days. Sludge storage and decomposition occur in the polishing pond.

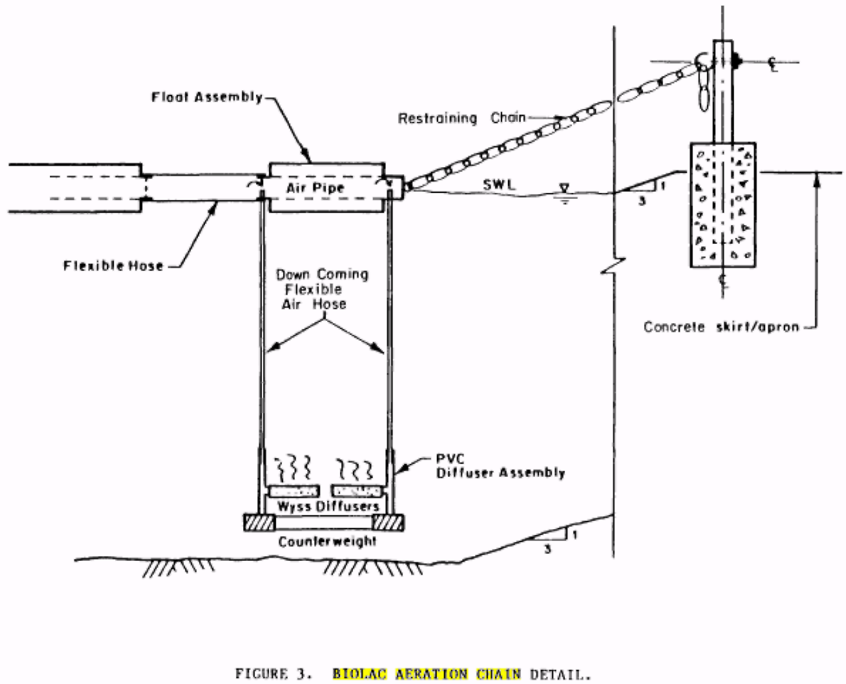


Figure 5-35. Detail of the BIOLAC® aeration chain element.

Table 5-8. Typical Manufacturer's Design Criteria for BIOLAC®-R Systems versus Conventional Extended Aeration Systems (Parkson Corp., 2004).

Parameter	Extended Aeration	BIOLAC®-R
HRT, h	18 - 36	24 - 48
SRT, d	20 - 30	30 - 70
F/M lbs BOD_5/d-lb MLVSS	0.05 – 0.15	0.03 – 0.1
Volumetric Loading, Lbs BOD_5/d-1000 ft^3	10 – 25	7 - 12
MLSS, mg/L	3000 – 6000	1500 - 5000
Basin Mixing, hp/mg	80 – 150	12 - 15

HRT - hydraulic retention time, hrs
SRT – solids retention time, days
F/M – food/ microorganism
MLVSS – mixed liquor volatile suspended solids
MLSS – mixed liquor suspended solids
See p.xiv for conversion factors.

Sludge storage for up to one to two decades is provided in the quiescent zone of the polishing or settling basin. Further sludge degradation of 40 - 60 percent occurs under anaerobic conditions in the settling basin.

The unique feature of the BIOLAC® systems is the floating aeration chain system (Figure 5-35). Fine bubble diffusers are suspended from a floating aeration chain that carries air to the diffusers. The floating aeration chain is attached to an anchor on the embankment, and allowed to move in a controlled way to create the oxic and anoxic zones. Each diffuser assembly can support from 2- 5 diffusers. Each diffuser is rated at 2 - 10 scfm and normally operates at an airflow rate of 6 scfm. Diffuser membranes are expected to last about 5 - 8 years.

Continuous service positive displacement rotary blowers are generally used. In larger systems, multistage centrifugal blowers may be more economical. Most systems use three blowers, each capable of providing 50 percent of the required airflow, one unit being a spare.

An integral clarifier is used with the BIOLAC®-R system although conventional clarifiers are used on occasion. BIOLAC®-L systems require that a polishing basin be installed for solids separation and storage. A cross-sectional view of the integral clarifier is shown in Figure 5-36. The integral clarifier is constructed in the aeration basin but is separated from the aeration zone by a concrete partition wall. Flow enters the clarifier along the bottom over the entire length of the partition wall to minimize short-circuiting. A flocculating rake moves the length of the clarifier sludge trough to concentrate and distribute the sludge. Sludge return and waste is removed with an air-lift pump.

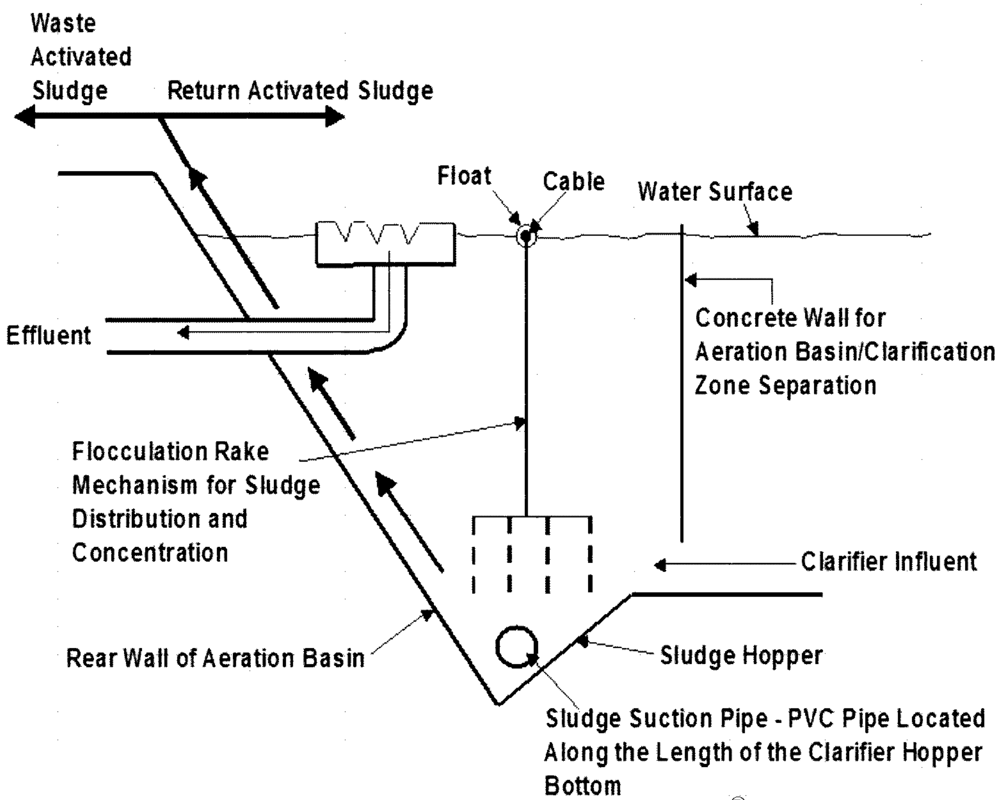


Figure 5-36. Cross-section view of the Integral BIOLAC®-R Clarifier (Parkson Corp., 2004).

5.5.5 Performance

Over 600 BIOLAC® systems have been installed in the United States and throughout the world. The results from two installations in Colorado are presented in this section.

5.5.5.1 Colorado BIOLAC® Facilities

The Alamosa and Tri-Lakes, Colorado BIOLAC® Systems have similar flow diagrams. A description of the Alamosa facility is shown in Table 5-9; the effluent standards are presented in Table 5-10. The systems have met the effluent standards established by the State of Colorado (Figures 5-37 through 5-41).

Table 5-9. Description of Alamosa BIOLAC® Facility (CDPHE 2007).

Unit Process	Unit Process Features/Description	Capacity Hydraulic/Organic
Influent Flow	Alamosa – 12" Parshall flume with recorder.	0.23 to 7.37 MGD
Measuring	East Alamosa – 8" Electromagnetic meter with recorder.	Recommended Range
Lifts Pumps	3-48" diameter screw pumps each rated at 2,900 gpm @ 24' head. Return sludge is by gravity to bottom of screw pumps.	8.3 MGD (Peak)
Bar Screen	One mechanical unit, 2.5' wide, 3/16" bars, ¾" spacing.	9.25 MGD (Peak)
Grit Removal	One aerated vortex grit chamber, 10' diameter at top, volume = 3,000 gallons, Minimum td = 30 seconds at peak flow .	8.3 MGD (Peak)
Aeration Basins	2 Basins, 150' x 196' x 12.75' deep with 2:1 inside sloops, Volume= 1,730,000 gallons each, td = 32 hours.	2.571 MGD
Aeration	3 – 100 hp blowers each rated @ 1965 scfm at 6.6 psi, 10 air laterals per basin, hanging diffuser assemblies per lateral.	3,110 lbs BOD_5/day
Clarifier	2 center feed octagon basins 58' inside diameter, 14' diameter center well, SWD = 12', Volume = 213,000 gallons, SOR = 515 gpd/ft^2 , td = 4 hours.	2.571 MGD
UV Light	2 UV channels, 2 UV modules per channel, lamps per module, 20,000 uWs/cm^2/module.	2.571 MGD
UV Contact Chamber	2 channels, 25' long, 3' wide.	2.571 MGD
Measuring	12" Parshall flume recorder.	0.23 to 7.27 MGD Recommended Range

Table 5-10. Effluent Requirements for the Alamosa BIOLAC® Facility (CDPHE, 2007).

Effluent Parameter (maximum)	**Discharge Limitations (maximum)**		
	30-D Average	7-D Average	Daily
Flow, mgd (reported)	2.57	NA	NA
BOD_5 mg/L	30	45	NA
TSS mg/L	30	45	NA
DO mg/L	NA	NA	5
Fecal coliform (MPN/100mL	288	576	NA
*p*H units	NA	NA	6.5-9.0
NH_3 mg/L			
Jan	6.0	NA	13
Feb	4.2	NA	10
Mar	2.4	NA	7.9
Apr	2.7	NA	11
May	2.3	NA	12
June	1.4	NA	9
July	1.6	NA	12
Aug	2.5	NA	15
Sept	1.8	NA	10
Oct	2.5	NA	11
Nov	2.1	NA	7
Dec	4.3	NA	10

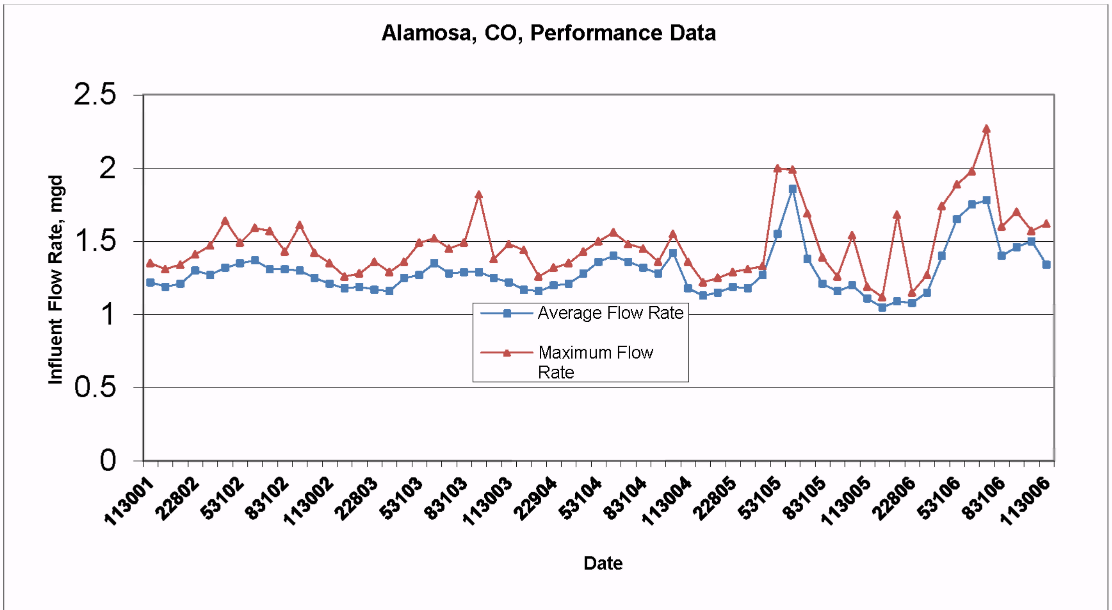


Figure 5-37. Flow rate for Alamosa BIOLAC® facility.

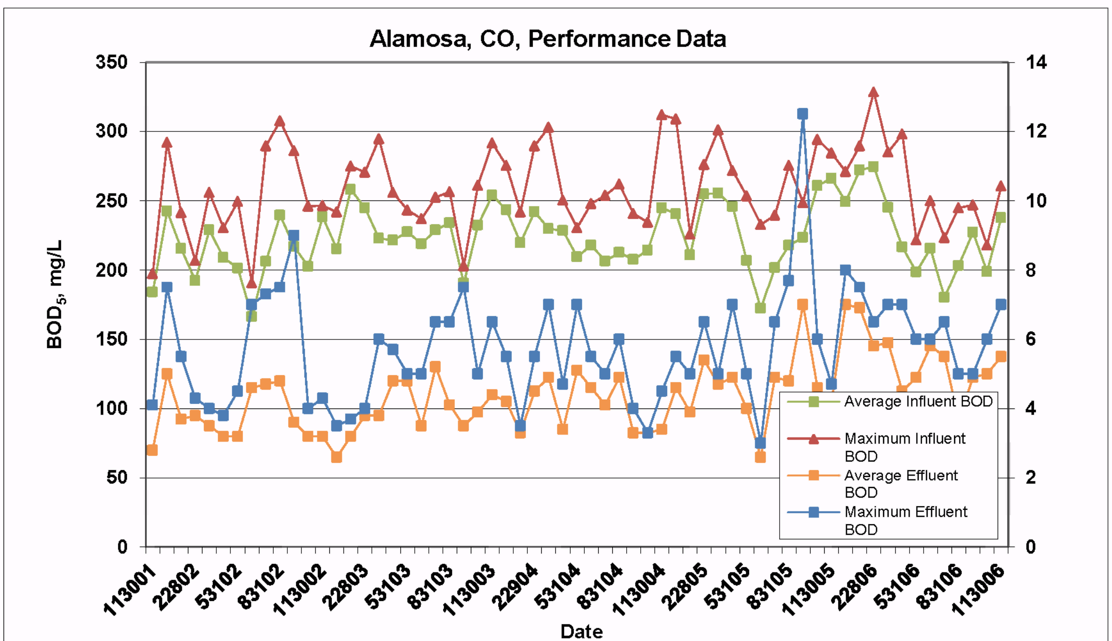


Figure 5-38. BOD_5 for Alamosa BIOLAC® facility. (upper lines =influent; lower lines = effluent).

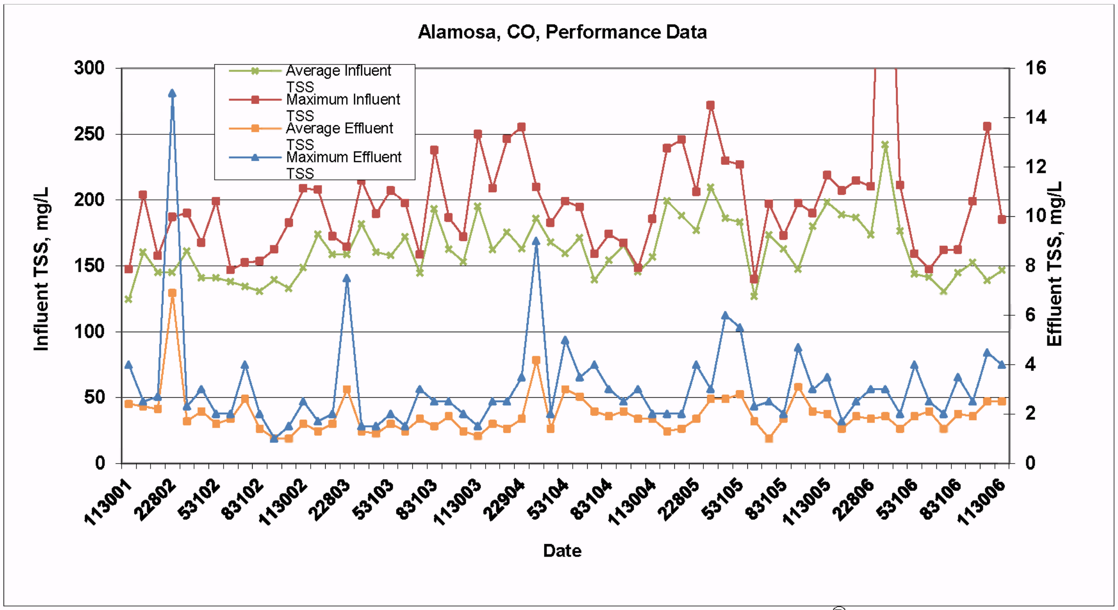


Figure 5-39. TSS for Alamosa BIOLAC® facility.

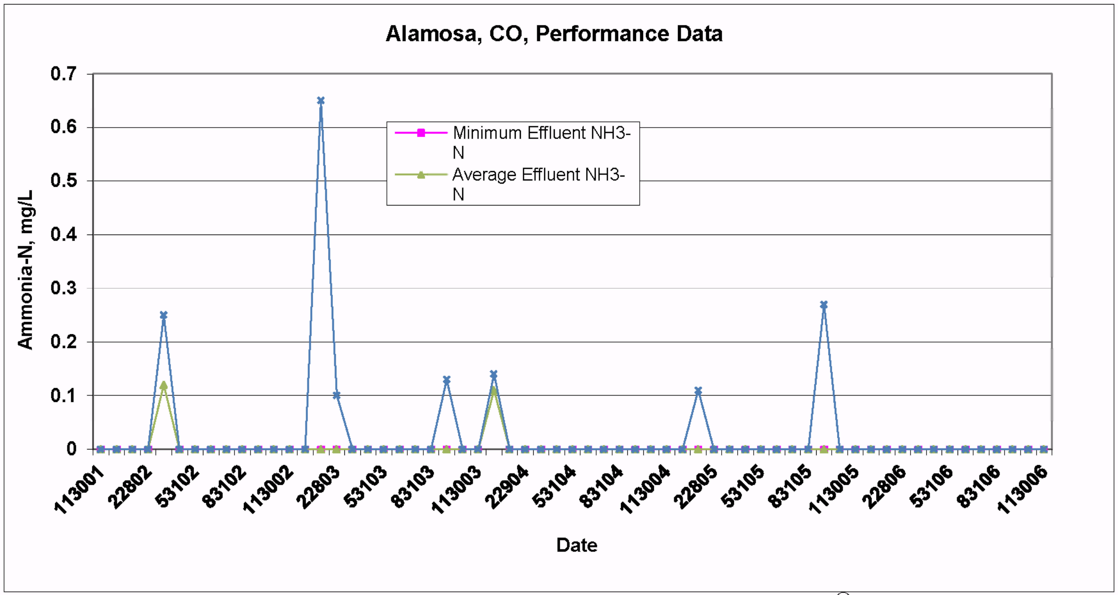


Figure 5-40. *NH_3* for Alamosa BIOLAC® facility.

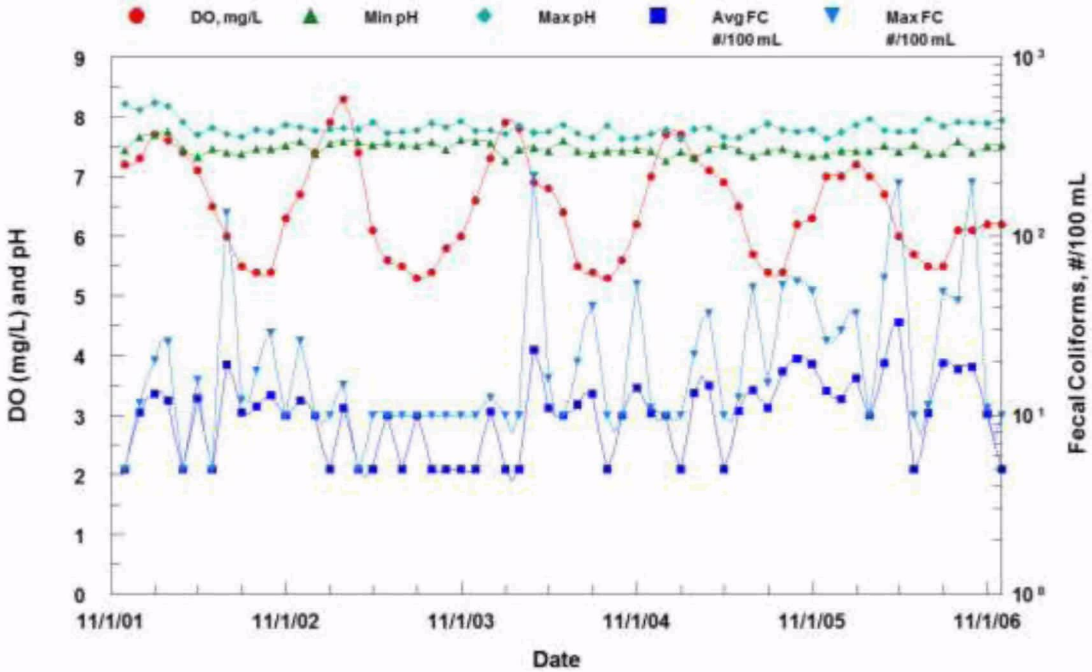


Figure 5-41. DO, *p*H and fecal coliform for Alamosa BIOLAC® facility.

The Tri-Lakes, Colorado facility is similar to the typical BIOLAC®-R system shown in Figure 5-33 except that there are a modified entrance to the aeration tank and a separate clarifier in place of the intra-pond settling section. The system has performed well for over five years as shown in Figures 5-42 through 5-46. Influent BOD_5 averaged 271 mg/L and the effluent averaged 3.4 mg/L, and the TSS influent averaged 288 mg/L with an effluent concentration of 4.4 mg/L. Effluent NH_3 was measured during 2005 and 2006, and the average for the two years was 1.73 mg/L, with values ranging from 0.4 to 7.1 mg/L. The maximum effluent concentration reported was less than 5 mg/L throughout the two years with only two exceptions. In those instances, the concentrations were 5.12 and 7.1 mg/L.

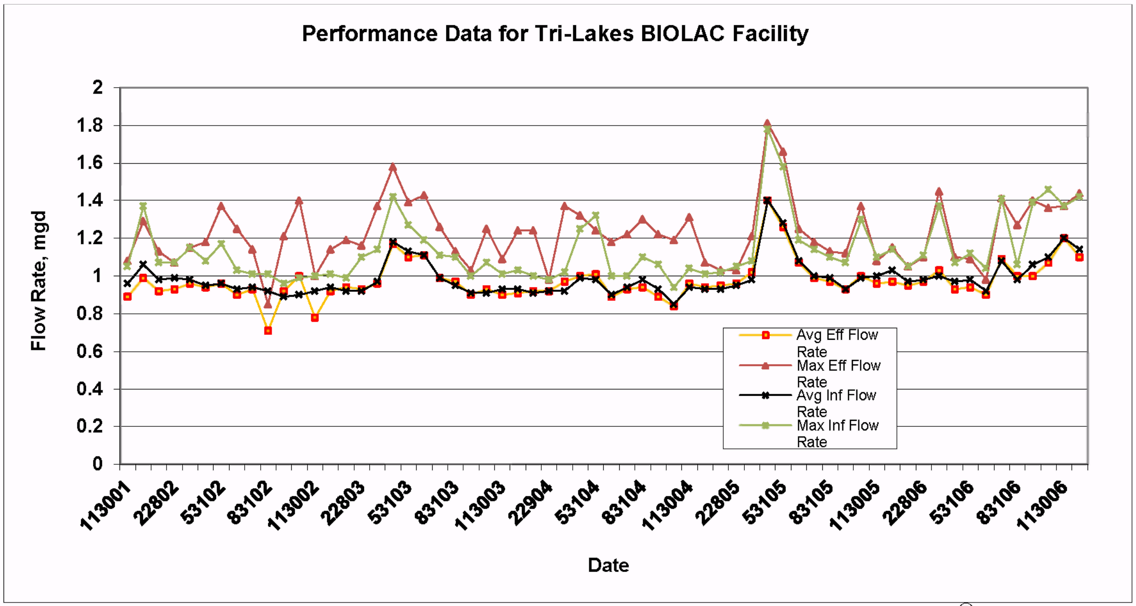


Figure 5-42. Flow rate for Tri-Lakes, Colorado BIOLAC® facility.

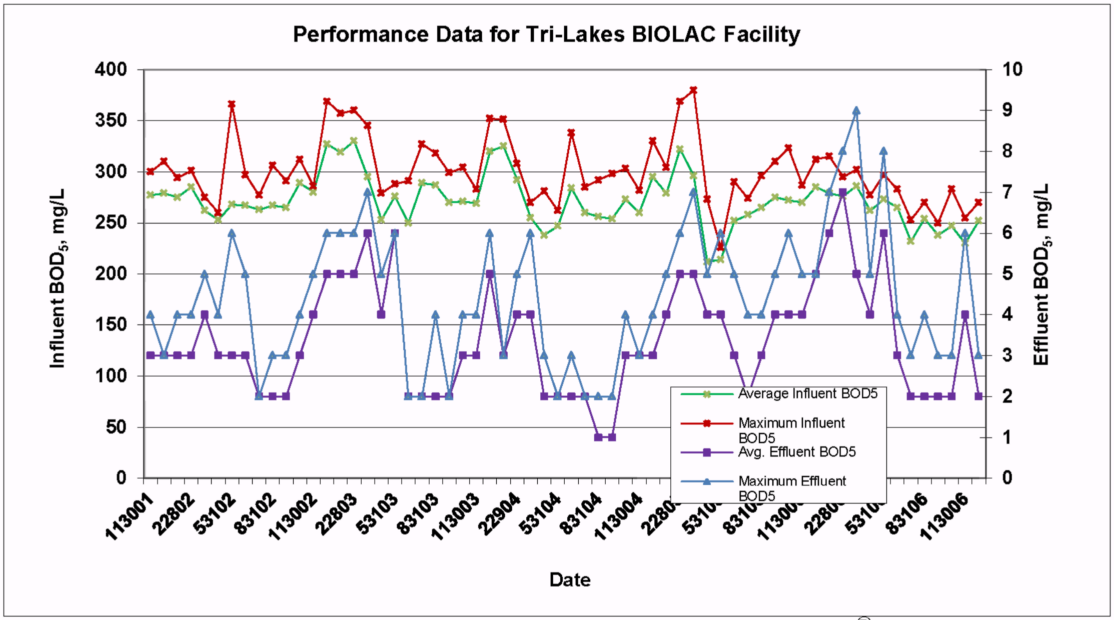


Figure 5-43. BOD_5 for Tri-Lakes BIOLAC® facility.

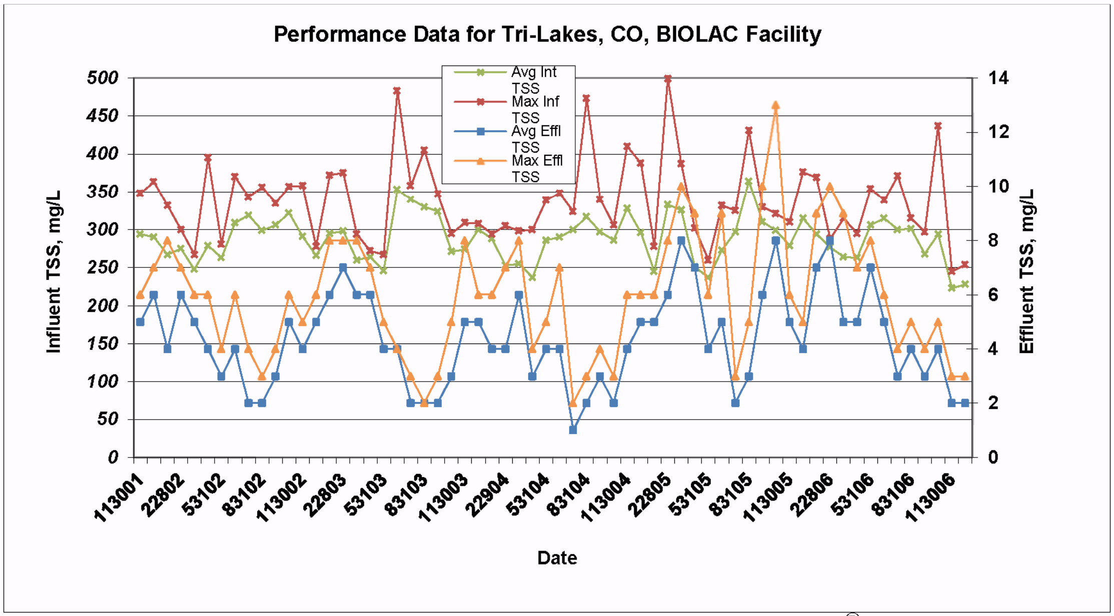


Figure 5-44. TSS for Tri-Lakes BIOLAC® facility.

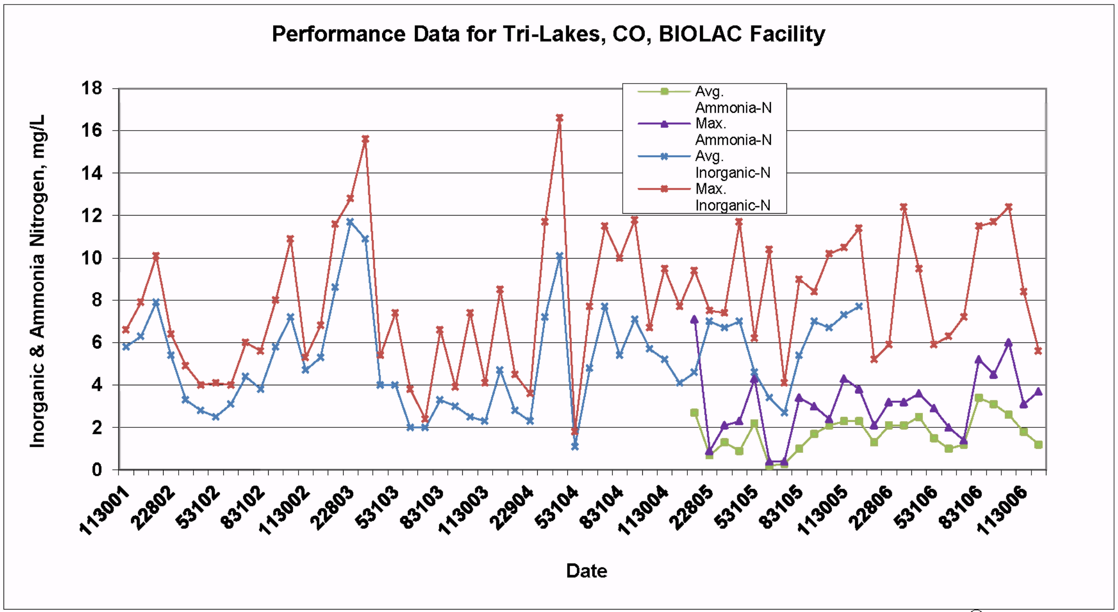


Figure 5-45. NH_3 and inorganic N (NO_2^-, NO_3^-) for Tri-Lakes BIOLAC® facility.

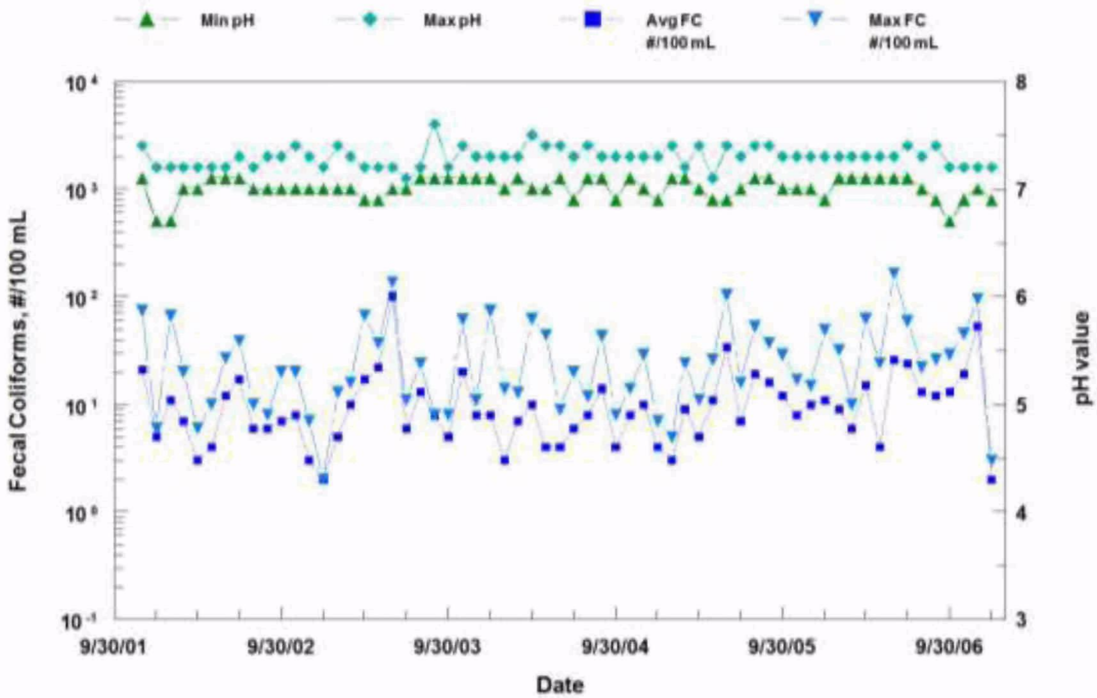


Figure 5-46. Fecal coliform for Tri-Lakes BIOLAC® facility.

5.5.5.2 Limitations

Depending upon the rate of aeration and the environment, BIOLAC® systems may not be cost-competitive when compared with conventional aerated pond systems.

5.5.5.3 Operation and Maintenance

In U.S. EPA (1990) there is a summary of the problems encountered at various BIOLAC® facilities. The difficulties were typical mechanical failures and excessive debris and floating sludges, with excessive oil and grease in the clarifier. Most of the problems appear to be correctable with routine maintenance.

5.5.5.4 Energy

See Section 5.3.1.8.

5.5.5.5 Costs. See Section 5.3.1.9. The operating costs of BIOLAC®-R and aerated ponds include aeration equipment and maintenance of these units. Solids production, treatment and handling in the BIOLAC®-R system will be higher than those with conventional aerated ponds depending upon the system used.

5.6 LEMNA SYSTEMS

5.6.1 Description

There are numerous references to the use of duckweed (*Lemna* spp.) in pond wastewater treatment systems dating back to the early 1970's. The present discussion, however, is limited to the application of proprietary processes produced by Lemna Technologies, Inc. (Culley and Epps, 1973; Wolverton and McDonald, 1979; Zirschky and Reed, 1988; Reed et al., 1995; Crites et al., 2006).

Lemna Technologies, Inc. offers two basic duckweed-based systems for wastewater treatment: the Lemna Duckweed System with floating partitions used to keep the plants evenly distributed over the surface of the pond, and the LemTec™ Biological Treatment Process. In addition to these two basic units, the company produces LemTec™ Modular Cover Systems, Lemna Polishing Reactor™, LemTec™ C-4 Chlorine Contact Chamber-Cleaner, LemTec™ Anaerobic Pond System, and the LemTec™ Gas Collection Cover.

Lemna Technologies, Inc. has installed approximately 370 projects worldwide. While the majority of the covered systems are located primarily in the United States they have been installed in Afghanistan, Mexico, Chile, and Poland. The typical client for the covered wastewater treatment system is small municipality. Gas collection cover systems have been placed exclusively in industrial treatment systems (http://www.lemnatechnologies.com/index.htm).

5.6.2 Modifications and Processes Available

5.6.2.1 Lemna Duckweed System

The Lemna System can be used to retrofit an existing facultative or aerated pond system or can be an original design. An original design consists of a regular facultative or aerated pond followed in series by Lemna System components including the floating barrier grid to prevent clumping of the duckweed and baffles to improve the hydraulics of the system. The treatment processes are followed by disinfection, if required, and reaeration of the effluent beneath the duckweed cover that is anaerobic. A diagram of the flow scheme for a typical Lemna System design is shown in Figure 5-47 (Lemna Technologies, Inc., 1999a and b). The Lemna System in Rayne, Louisiana is shown in Figures 5-48 and 5-49.

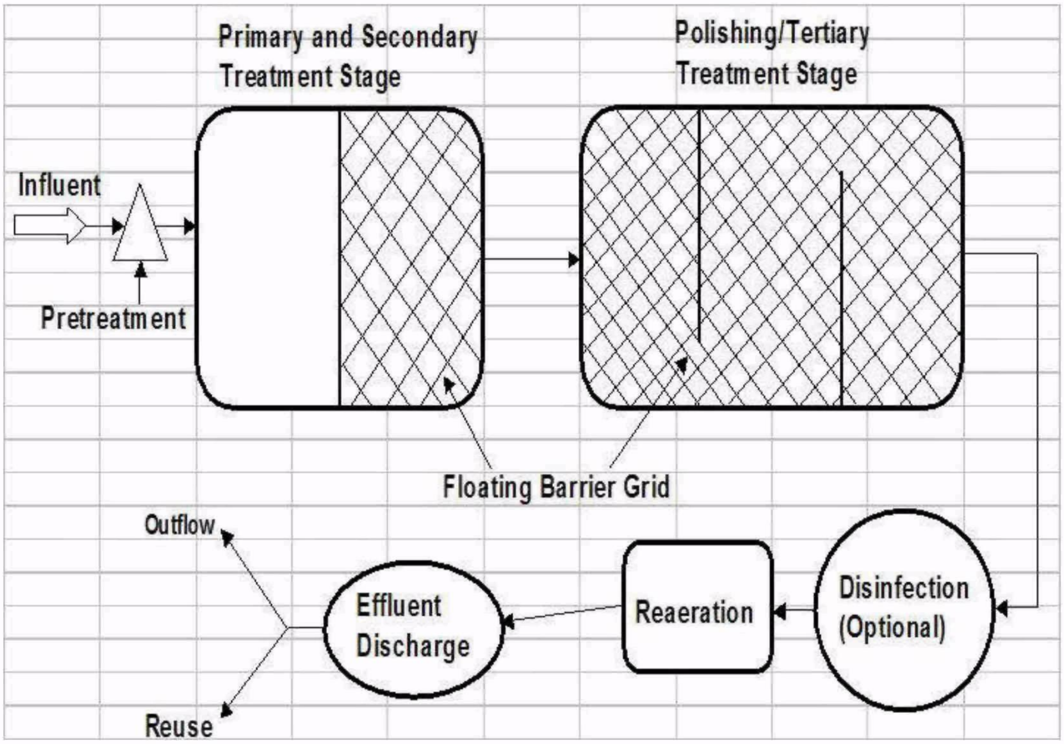


Figure 5-47. Flow diagram for typical Lemna System (Lemna Technologies, Inc., 1999a and 1999b, www.lemtechtechnologies.com).

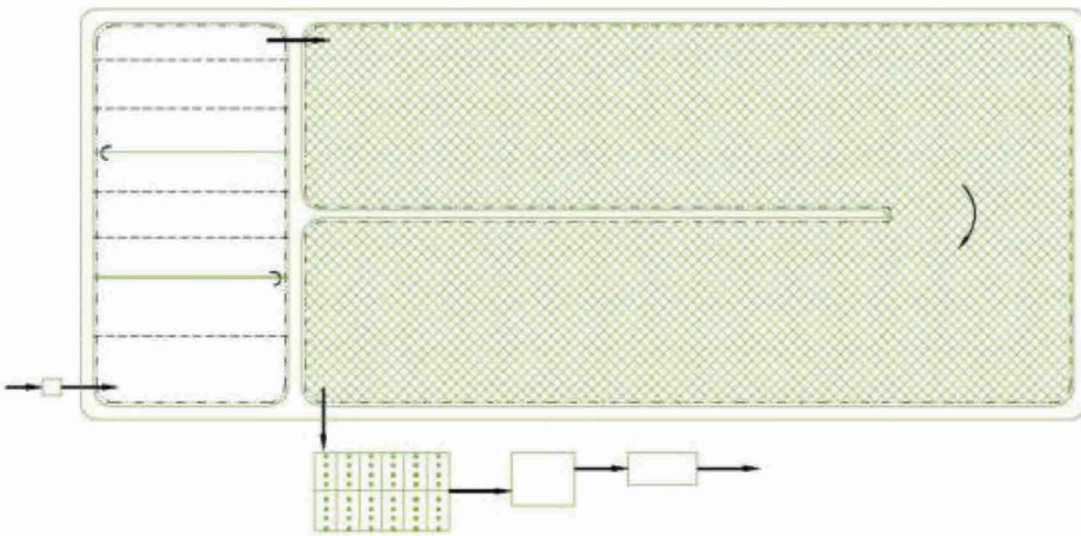


Figure 5-48. Diagram of Rayne, Louisiana wastewater treatment ponds. Green-hatched area is the Lemna System. Influent comes into the aeration pond to the left.

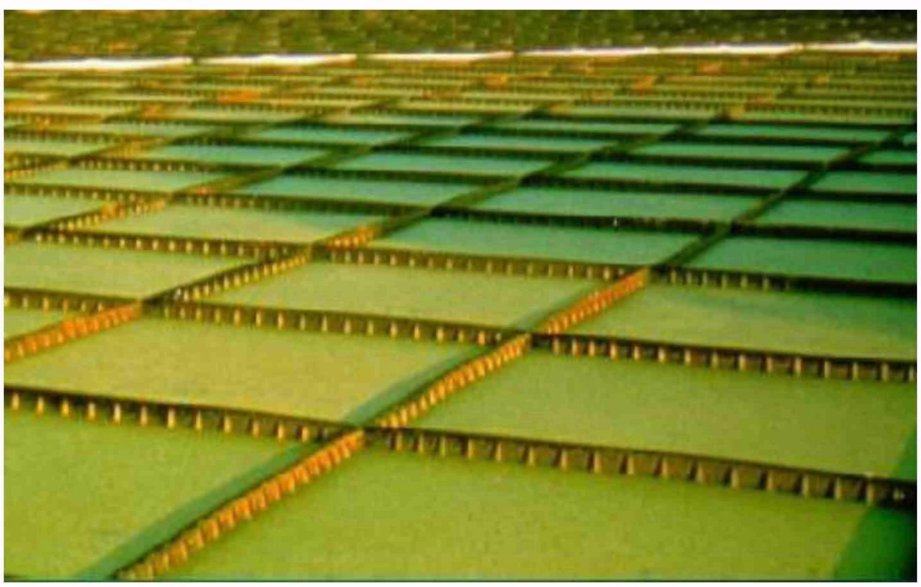


Figure 5-49. Lemna System, Rayne.

5.6.2.2 LemTec™ Biological Treatment Process

The Lemtec™ Biological Treatment Process uses the Lemtec™ Modular Cover to completely cover the system rather than a mat to retain duckweed (Figure 5-50). The process is still a pond-based treatment composed of a series of aerobic cells followed by an anaerobic settling pond. Cells in series consist of a complete mix aerated reactor, a partial mix aerated reactor, a covered anaerobic settling pond, and a Lemma polishing reactor. The polishing reactor is aerated and has submerged, attached-growth media modules to supplement BOD_5 and NH_3 reduction. Sludge removal from the settling pond is expected to be required about every 5 to 12 years. Frequency of cleaning will vary with climate and strength of the wastewater. The Lemtec™ System in Jackson Indiana is shown in Figures 5-51 and 5-52 .

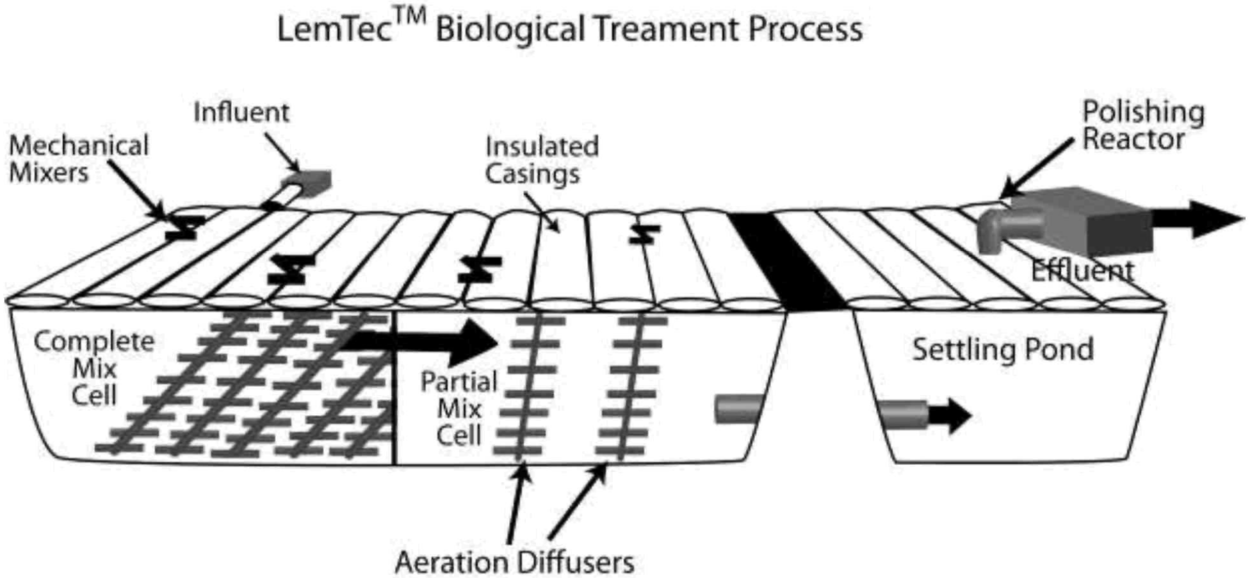


Figure 5-50. LemTec™ Biological treatment process (Lemna Technologies, 1999a and b, www.lemtechtechnologies.com).

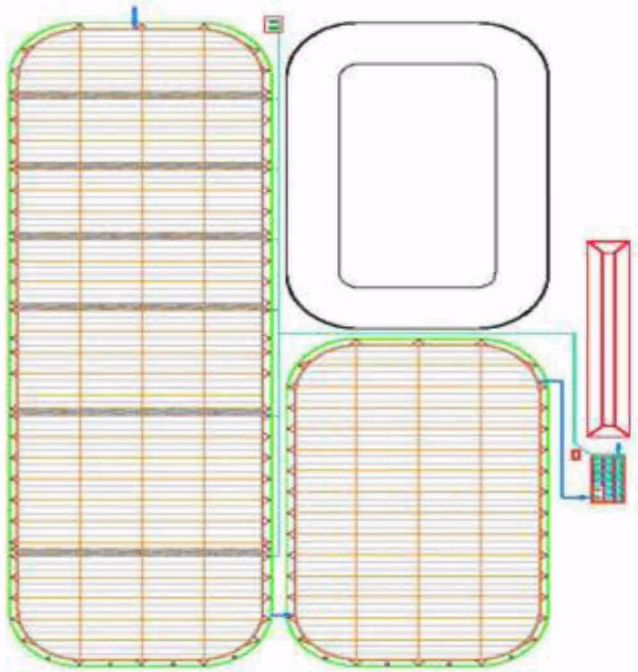


Figure 5-51. Site plan for LemTec™ System in Jackson, Indiana. The covered pond with crosswalks is on the left.

Figure 5-52. LemTec™ System, Jackson.

5.6.3 Applicability
The treatment process is suitable for areas where land costs are not a limiting factor. The systems can be used to treat raw, screened, or primary settled municipal or biodegradable industrial wastewaters. The process has been used to remove BOD_5, TSS, NH_3 and total N.

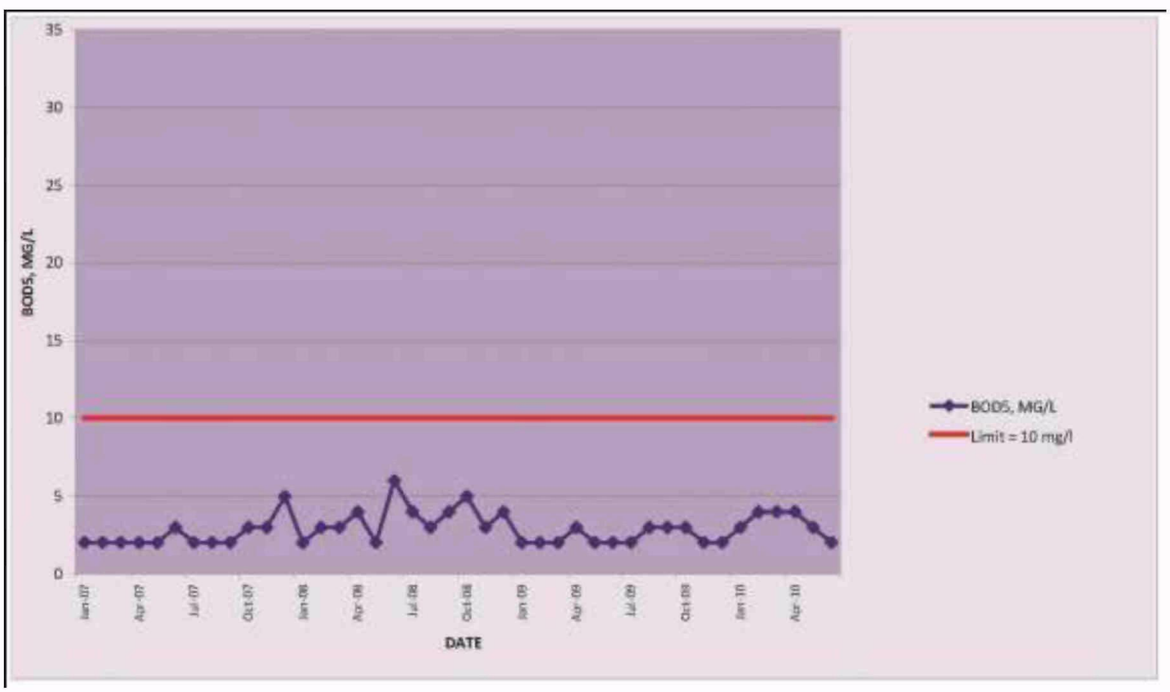
5.6.4 Advantages and Disadvantages
Both the Lemna Duckweed System and the LemTec™ Biological Treatment Process are effective in removing TSS, BOD_5, fecal coliform and NH_3.

On the other hand, both are susceptible to system impacts from toxic substance inputs, as are most biological systems. Harvesting, treatment and disposal of the duckweed can be time consuming and expensive (see Appendix D). Both systems may require increased maintenance, and aeration, therefore, increased operating costs.

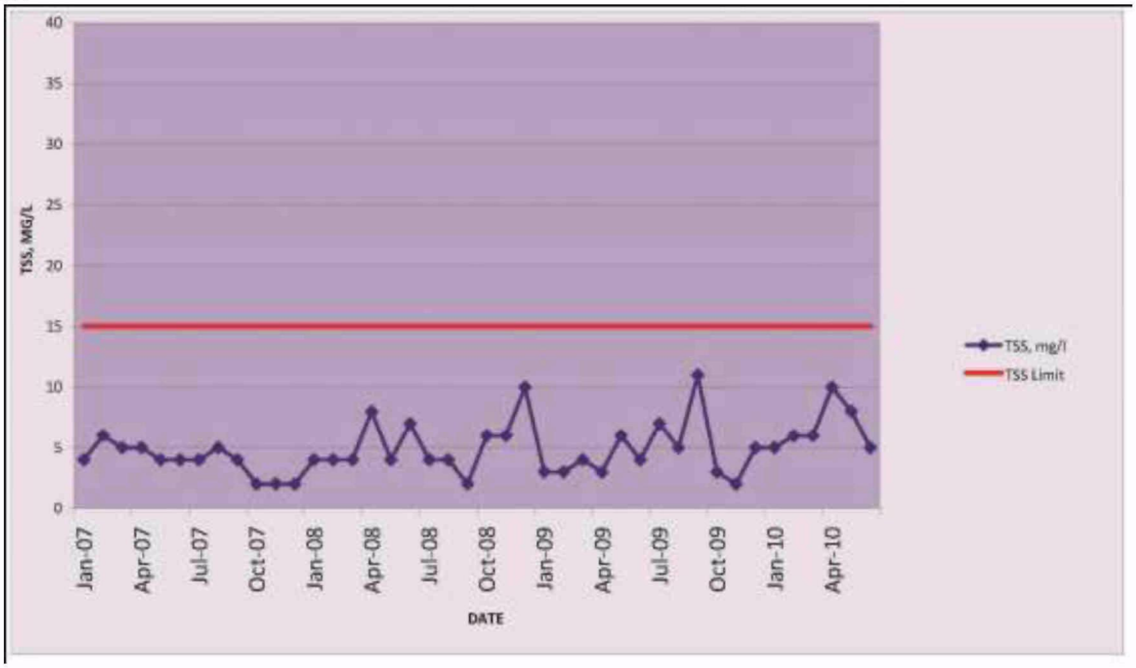
5.6.5 Design Criteria
Design criteria for Lemna Duckweed and the LemTec™ Biological Treatment processes were not provided by the manufacturer.

5.6.6 Performance Data for Lemna Systems
Performance data summary reported by Lemna are shown in Figures 5-53 A, B, and C (Rayne, Louisiana , the Lemna System) and Figures 5-54 A, B and C (Jackson, Indiana, the LemTec™ System).

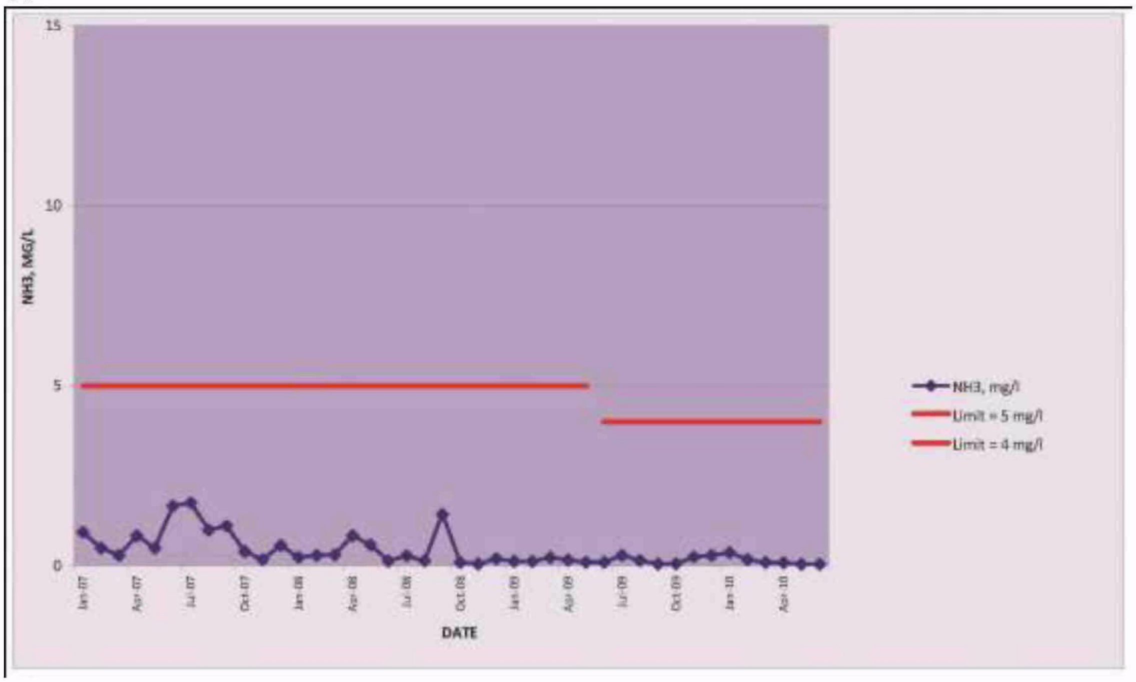
A.



B.



C.

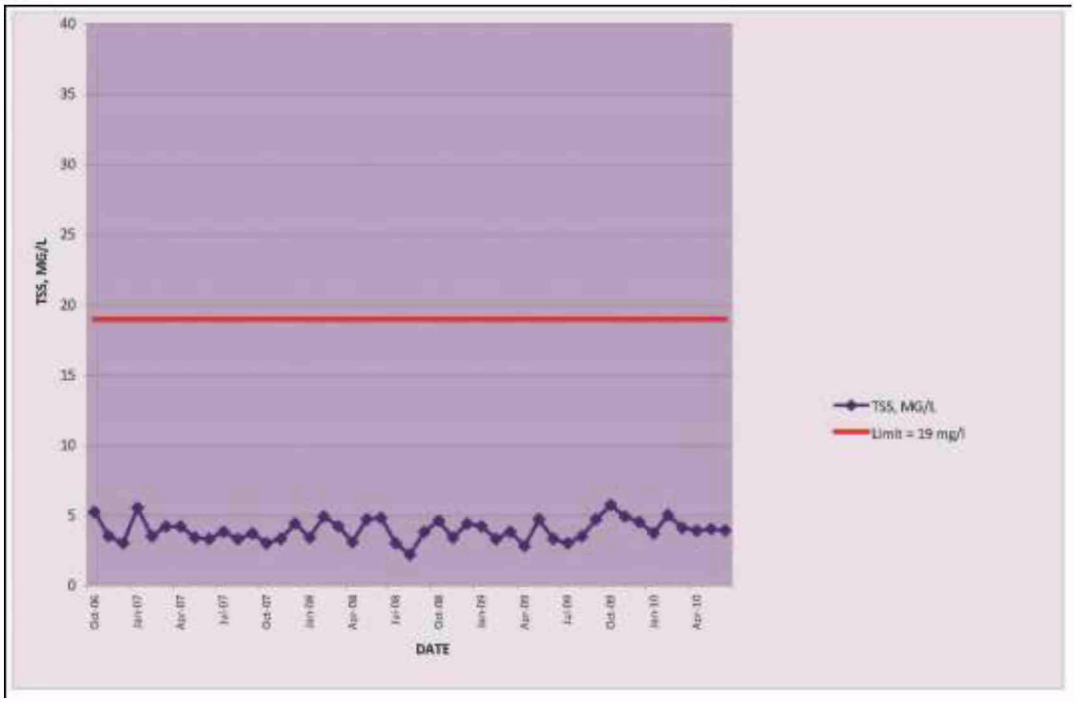


Figures 5-53 A. (BOD$_5$), B. (TSS) and C. (*NH_3*). Compliance data for Rayne (U.S. EPA Enforcement and Compliance History Online [ECHO] Database).

A.

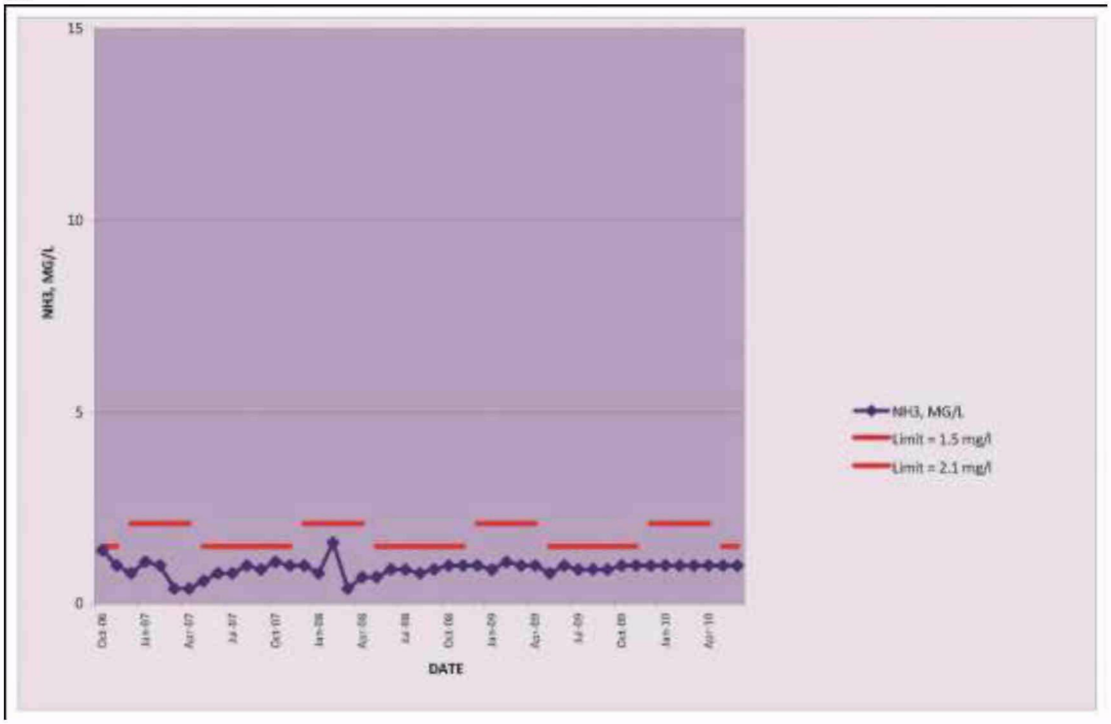
CBOD5, MG/L
DATE
CBOD5, MG/L
Limit = 10 mg/l

B.



TSS, MG/L
DATE
TSS, MG/L
Limit = 19 mg/l

C.



Figures 5-54 A ($CBOD_5$) B. (TSS) and C. (*NH_3*). Compliance data for Jackson (U.S. EPA ECHO Database).

5.6.7 Limitations
The Lemna systems have been used in both warm and cold climates. It is necessary to aerate the anoxic/anaerobic effluent from the *Lemna* covered cell to meet discharge requirements of 5 to 6 mg/L of DO.

5.6.8 Operation and Maintenance
For the Lemna System to function properly, it may be necessary to harvest the duckweed on a regular basis. Lemna Technologies markets a harvester for use in ponds with the floating barrier grid used to ensure distribution of the duckweed (Figure 5-55). The harvesters operate by depressing the floating barrier and removing the duckweed from the water surface. Biomass harvested from the Lemna System can be managed via land application, composting the duckweed or producing pelletized feedstuff. Other than land application, these management methods can be expensive, and data are needed to evaluate the economic feasibility of these two options. Other operation and maintenance procedures are the same as with other pond systems.

Figure 5-55. Photograph of Lemna harvesting equipment and floating barrier grid (Lemna Technologies, www.lemtechtechnologies.com).

5.6.9 Costs
Cost information was not available for either of the two Lemna Technologies processes, but costs are probably higher for these processes given the need for special equipment, such as the floating barrier grid for the Lemna systems and the cover for the LemTec™ Biological Treatment Process. Costs other than the special equipment would be the same as for facultative ponds or aerated ponds.

5.7 LAS International, Ltd., Accel-o-Fac® and Aero-Fac® Systems

5.7.1 Description
Accel-o-Fac® and Aero-Fac® systems are offered as upgrades and original installations. The Accel-o-Fac® is a facultative pond with wind-driven aerators, and the Aero-Fac® is a partial mix aerated pond with an Aero-Fac® diffused air bridge and LAS Mark 3 wind and electric aerators. Systems have been installed in several countries including the United Kingdom, Canada and the United States, but the only information that could be found on the web was related to installations in LaPine, Oregon and Holkham, Norfolk, United Kingdom (http://www.water-technology.net/projects/accelofac/accelofac8.html).

5.7.2 Applicability
The systems are appropriate for any aerated pond system.

5.7.3 Advantages and Disadvantages
The primary advantage of the processes is a savings in energy costs if adequate wind velocity is available.

A disadvantage is the lack of control of the aeration process.

5.7.4 Design Criteria
The systems are designed using the same approach as that used in the partial mix and complete mix design examples (see Chapter 3, Section 3.4 and Appendix C).

5.7.5 Performance
Performance data are limited and are mostly drawn from operation during warm months of the year. Winter performance data are needed to evaluate the processes fully. It is expected that the systems will perform essentially as other partial mix pond systems with equivalent aeration.

5.7.6 Limitations
Depending upon the rate of aeration and the environment, aerated ponds may experience ice formation on the water surface during cold weather.

5.7.7 Operation and Maintenance
Operation and maintenance are essentially the same as for aerated ponds.

5.7.8 Energy
See Section 5.3.1.8. The use of wind power should reduce the cost of energy, depending on climatic conditions.

5.7.9 Costs
See Section 5.3.1.9. Operating costs for the LAS aerated ponds include stand-by aeration equipment and maintenance of these units in addition to the wind driven aerators.

5.8 OXYGEN ADDITION SYSTEMS

5.8.1 Pure Oxygen Aeration System for Wastewater Treatment
While there are no ponds with these systems, they may have application in the future as efficiency and costs data become available (Agent: Holme Roberts & Owen LLP - Denver, CO, US, Inventor**:** Jai-Hun Lee, US Patent and Trademark Office [USPTO] Applicaton #**:** 20090008311 - Class: 210194 (1/8/2009).

5.8.2 PRAXAIR, INC., I-SO SYSTEM™
The Praxair® In-Situ Oxygenation (I-SO)™ Systems have been installed in over 100 locations throughout the world (Figure 5-56). The company states that the units are capable of transferring 109 kg/hr of O_2 per unit. Praxair® has reported that the total energy required to operate the I-SO™ System, including the generation of O_2, is as much as 60 percent less than the air systems the system replaced. Plants located near an oxygen pipeline can decrease power costs by as much as 90 percent (www.praxair.com).

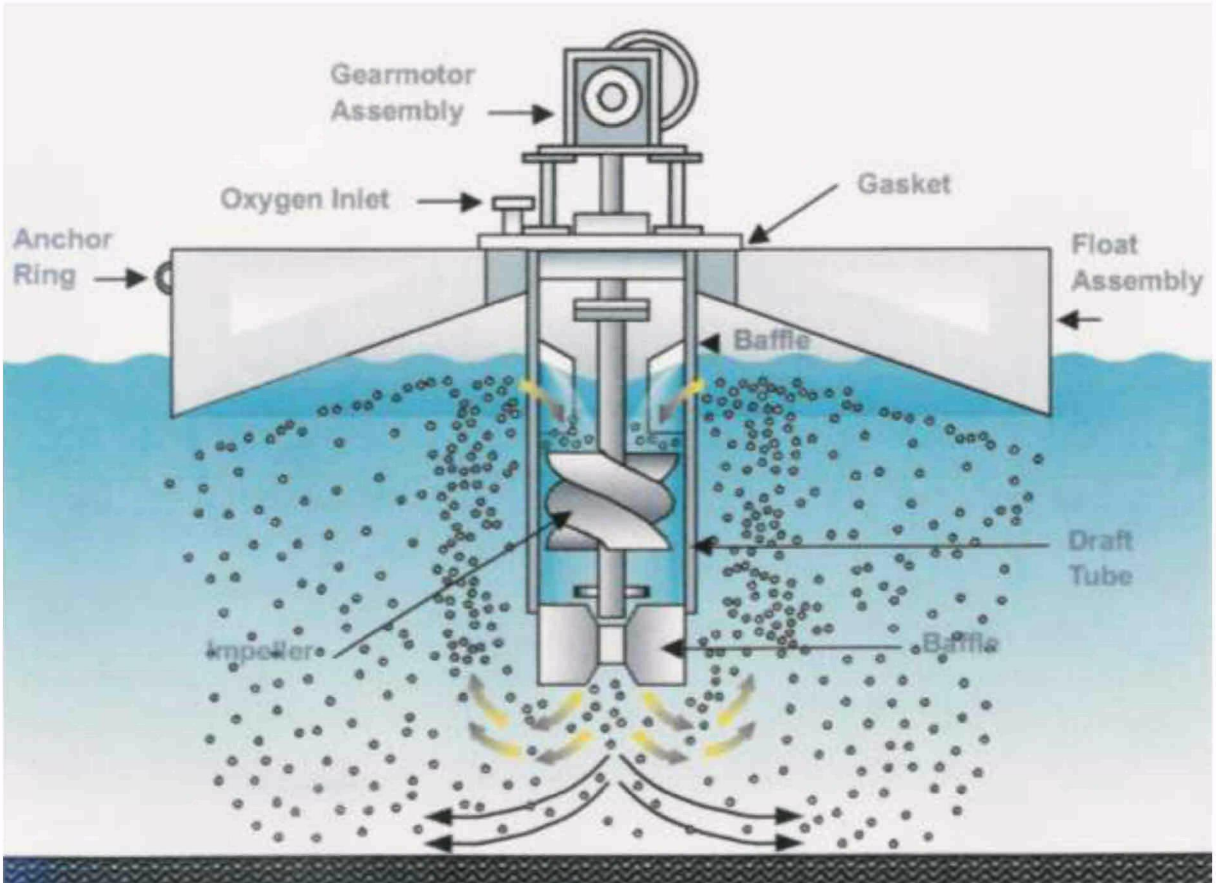


Figure 5-56. Praxair® In-Situ Oxygenation (I-SO™) System (Yoon et al., 2010).

5.8.3 ECO_2 SUPER-OXYGENATION SYSTEM

ECO_2 SuperOxygenation systems for water and wastewater treatment are designed and produced by Eco-Oxygen Technologies, LLC, based on the work of Dr. Richard Speece, who invented the Speece Cone, a device originally used to add oxygen to the bottom of lakes to enhance downstream fisheries (Speece, 2007). Photographs of the laboratory experiment used to develop the concept are shown in Figure 5-57. The ECO_2 SuperOxygenation method is a simple process based upon the scientific principle of Henry's Law. No chemicals, other than O_2, and no moving parts other than standard municipal wastewater pumps are used.

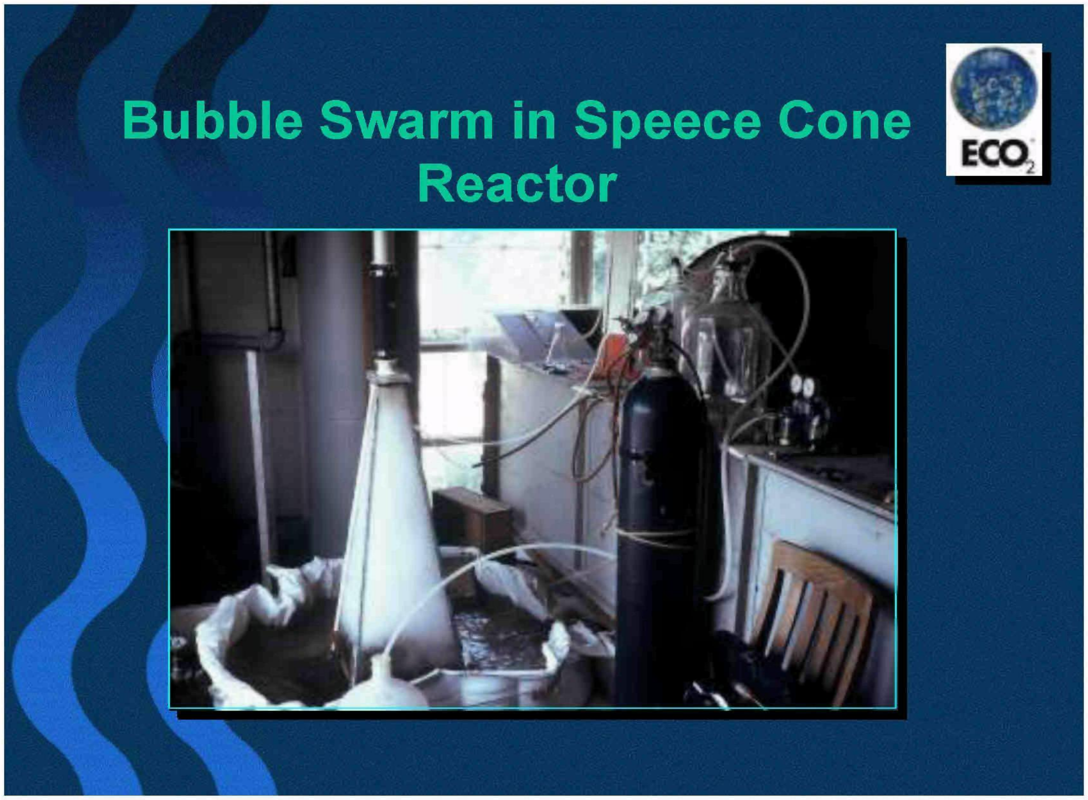
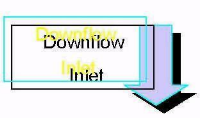


Figure 5-57. Photograph of the laboratory experiment used to develop the concept of the Speece Cone (Dominick and DeNatale, 2009).



CHAPTER 6

NUTRIENT REMOVAL

6.1 INTRODUCTION

While the reliability of pond systems to remove BOD_5 and suspended solids is well-documented, the *N* and *P* removal capability of wastewater ponds has been given little consideration in any type of system design until recently. As more stringent nutrient standards are adopted, nutrient removal processes must be included in design for new systems and added to existing systems. Nitrogen removal can be critical in many situations since NH_3, even at low concentrations, can adversely affect aquatic life in receiving waters, and the addition of NO_3 to surface waters is a major contributor to eutrophication. $Nitrate^-$is often the controlling parameter for design of land treatment systems. Any *N* removal in the primary pond units can result in very significant savings in acreage required for final land treatment. Phosphorus, which is limiting for algal growth, is present at concentrations in municipal wastewater that stimulate that growth and must be reduced to control eutrophication.

The dominant forms of *N* coming into a conventional facultative wastewater treatment pond system are referred to as the Total Kjeldahl Nitrogen (TKN), which is the sum of the organic *N*, NH_3 and ammonium ions (NH_4^+). In biological systems, such as aerobic ponds, where the *p*H is usually less than 8.0, the majority of the NH_3 is in the ionic form. TKN can be reduced through several processes, including gaseous NH_3 stripping to the atmosphere, NH_3 assimilation into the biomass, biological nitrification/denitrification and sedimentation of insoluble organic *N*. These processes are affected by temperature, DO concentration, *p*H value, retention time and wastewater characteristics. Within bottom sediments under anoxic conditions in facultative ponds, denitrification can take place. Temperature, redox potential and sediment characteristics affect the rate of denitrification. In well-designed aerated ponds with good mixing conditions and distribution of DO, however, the effect on the rate of denitrification will be negligible.

The capacity of conventional facultative and aerated ponds to convert NH_3 is discussed in the following sections. Several commercial processes that have been developed for *N* removal are also described. There is, however, little operational data available to demonstrate the effectiveness of the commercial units.

6.2 FACULTATIVE PONDS

6.2.1 Removal Mechanisms

Nitrogen loss from streams, lakes, impoundments, and wastewater ponds has been observed for many years. Data on *N* losses in pond systems were not sufficient to conduct a comprehensive analysis until the early 1980's, and even then there was no agreement as to the mechanisms of removal. Investigators have suggested algal uptake, sludge deposition, adsorption by bottom soils, nitrification, denitrification, and loss of NH_3 as a gas to the atmosphere (volatilization). Evaluations by Pano and Middlebrooks (1982), Reed (1984b) and Reed et al. (1995) indicate that a combination of factors may be responsible. The dominant mechanism, under favorable conditions, is thought to be loss by volatilization to the atmosphere. The several mechanisms are depicted in Figure 6-1.

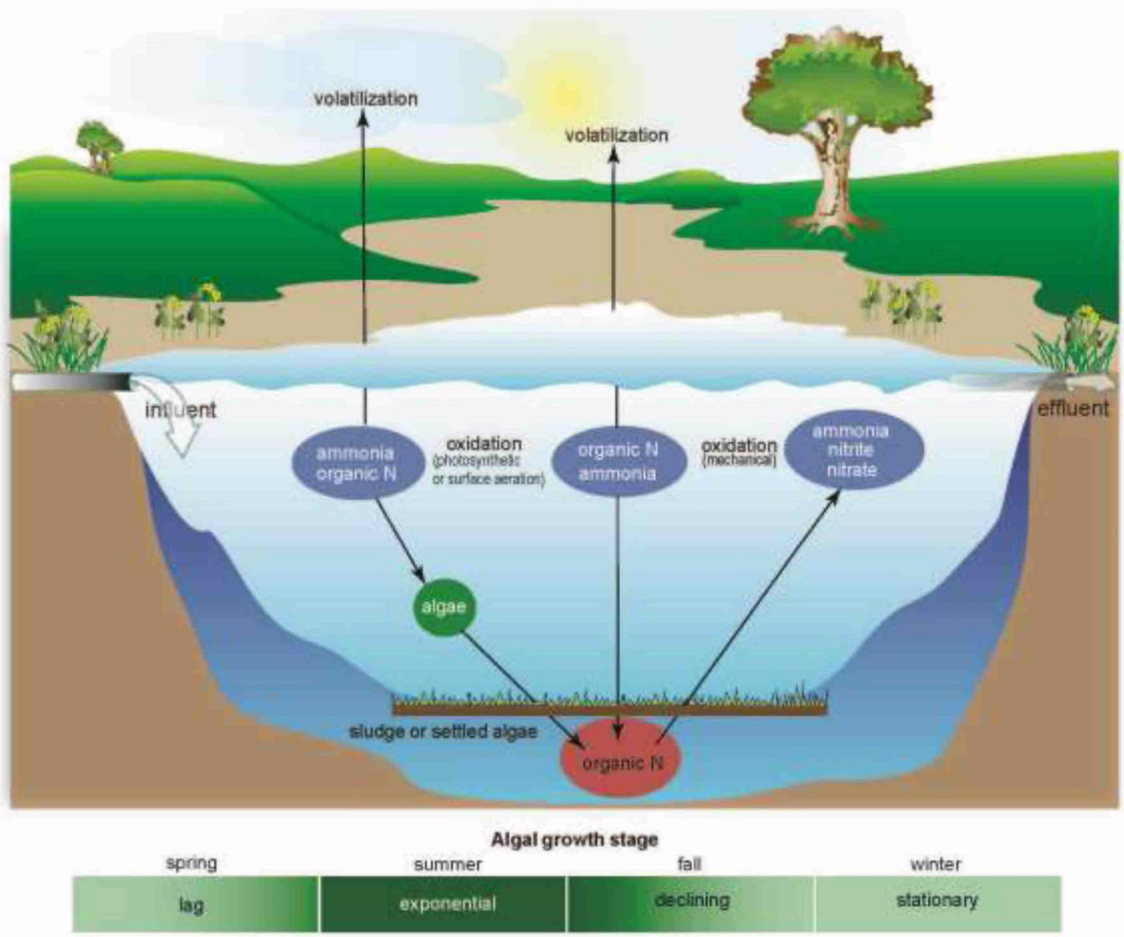


Figure 6-1. Generalized *N* pathways in wastewater ponds.

6. 2.2 Performance

EPA undertook a number of studies of facultative wastewater pond systems in the late 1970's (Bowen, 1977; Hill and Shindala, 1977; McKinney, 1977 and Reynolds et al., 1977). The results verified the hypothesis that significant *N* removal occurs in pond systems. Their findings from those studies are summarized in Table 6-1. Nitrogen removal is related to *p*H, detention time, and temperature. *p*H fluctuation resulting from the interaction of algae and HCO_3^- changes the alkalinity and is an important parameter to monitor. Under ideal conditions, up to 90 percent NH_3 removal can be achieved in facultative wastewater treatment ponds.

Table 6-1. Annual Values from EPA Facultative Wastewater Pond Studies

Location	HRT (d)	Temp (°C) Ave. range	*p*H (median)	Alkalinity (mg/L)	*N* Removal (percent)
Peterborough 3 cells	107	-7 – 20	7.1	85	43
Kilmichael 3 cells	214	4.5 – 26	8.2	116	80
Eudora 3 cells	231	1.1 – 27	8.4	284	82
Corinne First 3 cells	42	-3.9 – 23	9.4	555	46

Several studies of *N* removal have been completed more recently, but the quantity of data is still limited. A study of 178 facultative ponds in France showed an average *N* removal of 60 to 70 percent (Racault et al., 1995). Wrigley and Toerien (1990) studied four small-scale facultative ponds in series for 21 months and observed an 82 percent reduction in NH_3.

Shilton (1995) quantified the removal of NH_3 from a facultative pond treating piggery wastewater, and found that the rate of volatilization varied from 355 to 1534 mg/m^2/d (0.07 - 0.314 lb/1000 ft^2/d). The rate of volatilization increased at higher concentrations of NH_3 and TKN.

Soares et al., (1995) monitored NH_3 removal in a wastewater treatment pond complex of different geometries and depths in Brazil, and found that the NH_3 concentrations were lowered to 5 mg/L in the maturation ponds so that the effluent could be discharged to surface waters. The NH_3 removal in the facultative and maturation ponds could be modeled by the equations based on the volatilization mechanism proposed by Pano and Middlebrooks (1982).

Using ^{15}N-labelled NH_3, Camargo Valero and Mara (2007) demonstrated the uptake of NH_3 by the algal biomass in the pond, followed by assimilation into the suspended organic fraction (85 percent in the effluent), and movement into the pond benthos by sedimentation. A study of ponds in Kansas (Tate et al., 2002) showed increased NH_3 in August, which would tend to confirm that it is taken up by algae, and its movement to the benthos, from which it is released under late summer conditions.

6.2.3 Theoretical Considerations

It is hypothesized that NH_3 removal in facultative wastewater treatment ponds occurs via three mechanisms: gaseous NH_3 stripping to the atmosphere, NH_3 assimilation in algal biomass and biological nitrification.

The low concentrations of NO_3^- and $NO_2^=$ measured in pond effluents indicate that nitrification generally does not account for a significant portion of NH_3 removal. Ammonia assimilation in algal biomass depends on the biological activity in the system and is affected by temperature, organic load, detention time, and wastewater characteristics. The rate of gaseous NH_3 losses to the atmosphere depends mainly on the pH value, temperature, and the mixing conditions in the pond. Alkaline pH shifts the equilibrium equation $NH_3\uparrow + H_2O \leftrightarrow NH_4^+ + OH^-$ toward gaseous NH_3, whereas the mixing conditions affect the magnitude of the mass transfer coefficient. Temperature affects both the equilibrium constant and mass transfer coefficient.

At low temperatures, when biological activity decreases and the pond contents are generally well mixed due to wind effects, stripping will be the major process for NH_3 removal in facultative wastewater treatment ponds. The NH_3 stripping process in ponds may be expressed by assuming a first-order reaction (Stratton, 1968; 1969). The mass balance equation will be:

$$VdC/dt = Q(C_o - C_e) - kA(NH_3) \qquad (6\text{-}1)$$

where:

Q = flow rate, m^3/d
C_o = influent concentration of ($NH_4^+ + NH_3$), mg/L as N
C_e = effluent concentration of ($NH_4^+ + NH_3$), mg/L as N
C = average pond contents concentration of ($NH_4^+ + NH_3$), mg/L as N
V = volume of the pond, m^3
k = mass transfer coefficient, m/d
A = surface area of the pond, m^3
t = time, d

The equilibrium equation for NH_3 dissociation may be expressed as

$$K_b = \frac{[NH_4^+][OH^-]}{[NH_3]} \qquad (6\text{-}2)$$

where:

K_b = NH_3 dissociation constant.

By modifying Equation 6-2, gaseous NH_3 concentration may be expressed as a function of the pH value and total NH_3 concentration ($NH_4^+ + NH_3$) as follows:

$$[H^+] = \frac{K_W}{[OH^-]} \qquad (6-3)$$

$$C = NH_4^+ + NH_3 \qquad (6-4)$$

$$NH_3 + \frac{C}{1 + 10^{pKw\ -pKb\ -\ pH}} \qquad (6-5)$$

where:

$pK_W = -log\ K_W$
$pK_b = -log\ K$

Assuming steady-state conditions and a completely mixed pond where $C_e = C$, Equations 6-4 and 6-5 will yield the following relationship:

$$\frac{C_e}{C_o} = \frac{1}{1 + \frac{AK}{Q}\left[\frac{1}{1 + 10^{pKw - pKb - pH}}\right]} \qquad (6 - 6)$$

This relationship emphasizes the effect of *p*H, temperature (pK_W and pK_b are functions of temperature) and hydraulic loading rate on NH_3 removal.

Experiments on NH_3 stripping conducted by Stratton (1968, 1969) showed that the NH_3 loss-rate constant was dependent on the *p*H value and temperature (T = °C) as shown in the following relationships:

$$NH_3 \text{ loss rate constant} \propto e^{1.57(pH-8.5)} \qquad (6 - 7)$$
$$NH_3 \text{ loss rate constant} \propto e^{0.13(T-20)} \qquad (6 - 8)$$

King (1978) reported that only four percent N removal was achieved by harvesting floating *Cladophora fracta* from the first pond in a series of four receiving secondary effluents. The major N removal in the ponds was attributable to NH_3 gas stripping. The removal of total N was described by first-order kinetics, using a plug flow model ($N_t = N_0\, e^{-0.03t}$ where N_t = total N concentration, mg/L, N_0 = initial total N concentration, mg/L and t = time, d).

During windy seasons, or large-scale facultative steady-state conditions, will well-designed ponds approach completely mixed conditions. Moreover, when NH_3 removal through biological activity becomes significant, or NH_3 is released from anaerobic activity at the bottom of the pond, the expressions for NH_3 removal in the system must include these factors along with the theoretical consideration of NH_3 stripping. A mathematical relationship for total N removal based on the performance of three full-scale facultative wastewater treatment ponds is developed here that considers the theoretical approach and incorporates temperature, *p*H value, and hydraulic loading rate as variables. In this case, Equation 6-9 for TKN removal in facultative ponds is substituted for the theoretical expression for NH_3 stripping:

$$\frac{C_e}{C_o} = \frac{1}{1 + \frac{AK}{Q} \times f(pH)} \qquad (6 - 9)$$

where:

K = removal rate coefficient (L/t)
$f(pH)$ = function of *p*H.

The K values are considered to be a function of temperature and mixing conditions. For a similar pond configuration and climatic region, the K values may be expressed as a function of temperature only. The function of *p*H, which is considered to be dependent on temperature, affects the pK, and pK_b values, as well as the biological activity. Based on a statistical analysis of the data when incorporated into the equation, the *p*H function was found to describe an exponential relationship. As many reaction rate and temperature relationships are described by exponential functions, such as the Van't Hoff-Arrhenius equation, it is not unreasonable to assume that such a relationship would apply to the application of the theoretical equation to a practical problem.

6.2.4 Design Models

Data were collected on a frequent schedule from every cell in the pond system listed in Table 6-1 for at least a full annual cycle. A quantitative analysis of all major variables was performed and several design models were developed. Both plug flow and complete mixing models were useful in predicting *N* removal in facultative pond systems (Tables 6-2 and 6-3). They are first-order models that depend on *p*H, temperature and HRT. They have been validated using data from other sources. Further validation of the models can be found in Crites et al. (2006), Reed et al. (1995), Reed (1985), and Reed (1984b).

Both models assume that volatilization of NH_3 is the major pathway of *N* removal from wastewater treatment ponds. The application of the models is shown in Figure 6-2, and the predicted total *N* in the effluent is compared to the actual monthly average values measured at the Peterborough, New Hampshire ponds. The models are expressed in terms of total *N*, and should not be confused with the equations reported by Pano and Middlebrooks (1982) that are limited to the NH_3 fraction.

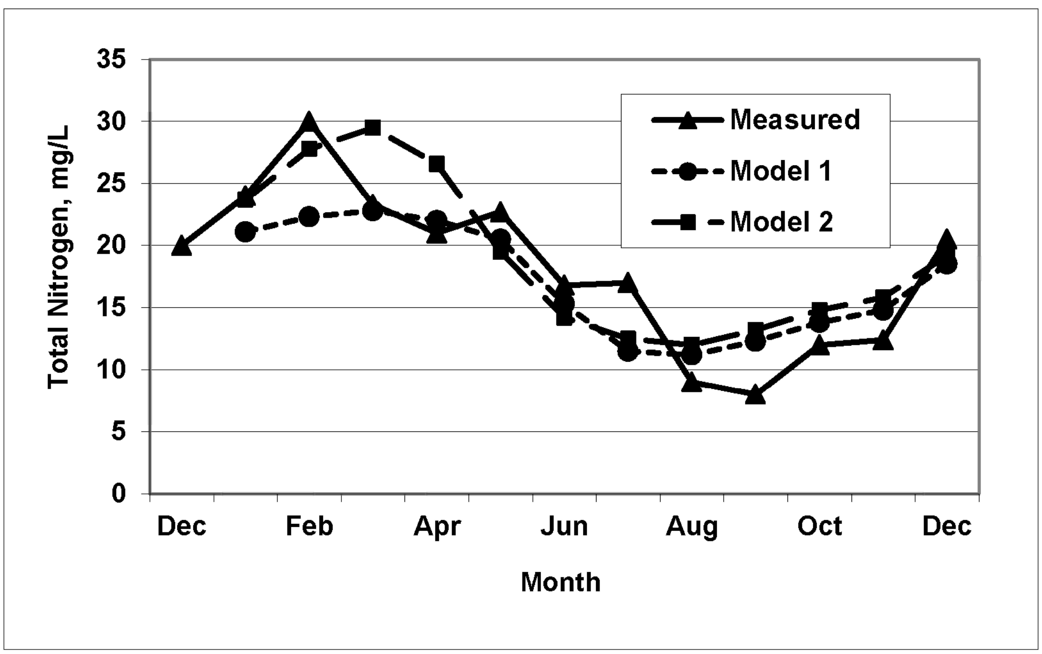


Figure 6-2. Predicted versus actual effluent *N*, Peterborough, New Hampshire.

Table 6-2. Model 1, N Removal in Facultative Ponds – Plug Flow Model (Reed, 1985)

$$N_e = N_o e^{-K_T [t + 60.6 (pH - 6.6)]} \qquad (6-10)$$

where:

N_e = effluent total nitrogen, mg/L
N_0 = influent total nitrogen, mg/L
K_T = temperature dependent rate constant
$K_T = K_{20} (\theta)^{(T-20)}$
K_{20} = rate constant at 20°C = 0.0064
θ = 1.039
t = detention time in system, d
pH = pH of near surface bulk liquid

See Reed (1984b) for typical pH values or estimate with:

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

Use the Mancini and Barnhart Equation (1976) for pond water temperature determination.
where:

ALK = expected influent alkalinity, mg/L [derived from data in U.S. EPA (1983a) and Reed (1984b)]

$$T = \frac{0.5AT_a + QT_i}{0.5A + Q}$$

where:

A = surface area of pond, m^2
T_a = ambient air temperature,°C
T_i = influent temperature,°C
Q = influent flow rate, m^3 / d

A high rate of NH_3 removal by air stripping in advanced wastewater treatment depends on a high (> 10) chemically adjusted pH. The algae-carbonate interactions in wastewater ponds can elevate the pH to similar levels for brief periods during the day. At other times, at lower pH levels, the rate of removal may be slower, but the long detention time results in lower N concentrations in the effluent.

Table 6-3. Model 2, N Removal in Facultative Ponds – Complete Mix Model (Middlebrooks, 1985).

$$N_e = \frac{N_0}{1 + t(0.000576T - 0.00028)e^{(1.080-0.042T)(pH-6.6)}} \qquad (6-11)$$

where:

N_e = effluent total nitrogen, mg/L
N_o = influent total nitrogen, mg/L
t = detention time, d
T = temperature of pond water, °C
pH = pH of near surface bulk liquid

Use the Mancini and Barnhart (1976) Equation (Table 6-2) to determine pond water temperature.

Figure 6-3 (Crites, 2006) illustrates the validation of both models using data from pond systems from other sources. The diagonal line represents the best fit of predicted versus actual values. The close fit and consistent trend demonstrate that either model can be used to estimate N removal. In addition, the models have been used in the design of several ponds systems and have been found to work as predicted. It is nevertheless likely that other removal mechanisms are at work (Camargo Valero and Mara, 2007) and therefore this model should be used only as a first step in designing for NH_3 removal in ponds.

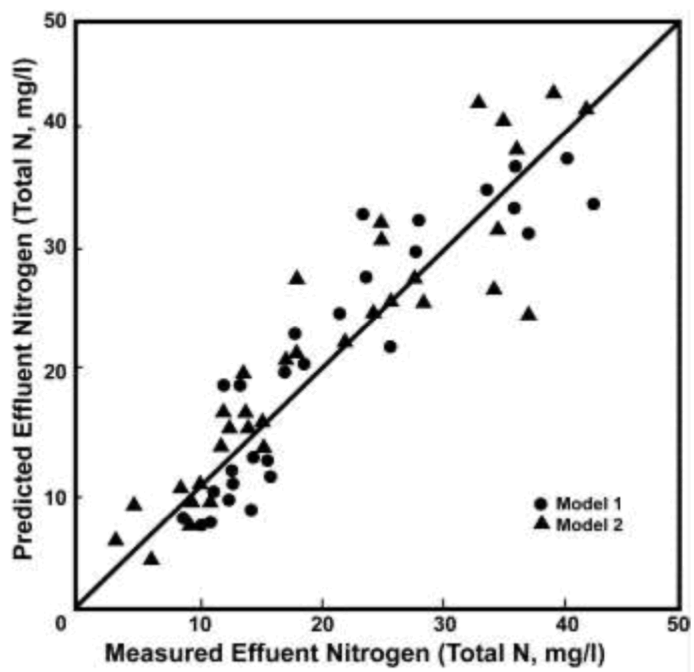


Figure 6-3. Verification of design models.

6.2.5 Applicability

Nitrogen removal occurs in facultative wastewater treatment ponds, and may be reasonably predicted for design purposes with either of the two models. Nitrogen removal in ponds may be more cost-effective than other alternatives for removal and/or NH_3 conversion. These models should be useful for new or existing wastewater ponds when N removal and/or NH_3 conversion is required. The design of new systems would typically base detention time on the BOD_5 removal requirements. The N removal that will occur during that time can then be calculated with either model. It is prudent to assume that the remaining N in the effluent will be NH_3 and to design any further removal/conversion for that amount. A final step would be to compare the cost of additional detention time in an expanded pond system for *N* removal with other removal alternatives.

Use of these models is particularly important when ponds are used as a component in land treatment systems, since total *N* is often the controlling design parameter. A reduction in pond effluent *N* will often permit a significant reduction in the area needed for land treatment.

6.2.6 Limitations

Other than the requirement for sufficient pond acreage, the facultative pond system can be expected to provide maximum NH_3 conversion during the summer months, occasionally exceeding 90 percent. This level of treatment cannot be expected during winter months.

6.2.7 Operation and Maintenance

Operation and maintenance are the same as for facultative ponds designed to remove BOD_5.

6.3 AEROBIC PONDS

6.3.1 Introduction

The same conditions that apply to push the equilibrium of NH_4^+ to NH_3 in conventional facultative pond systems applies to aerobic ponds.

6.3.2 Performance

EPA sponsored comprehensive studies of aerated pond systems between 1978 and 1980 that provided information about *N* removal (Tables 6-4 and 6-5). The results verify the consensus of previous investigators that *N* removal is related to *p*H, detention time and temperature in the pond system.

Table 6-4. Wastewater Characteristics and Operating Conditions for Five Aerated Ponds (Earnest et al., 1978; Englande, 1980; Gurnham et al., 1979; Polkowski, 1979; Reid and Streebin, 1979; Russell et al., 1980)

See p.xiv for conversion table.

Parameter	Pawnee[a]	Bixby[b]	Koshkonong[c]	Windber[d]	North Gulfport[e]
BOD, mg/L	473	368	85	173	178
COD, mg/L	1026	635	196	424	338
TKN mg/L	51.41	45.04	15.3	24.33	26.5
NH_3-N mg/L	26.32	29.58	10.04	22.85	15.7
Alkalinity mg/L	242	154	397	67	144
*p*H	6.8-7.4	6.1-7.1	7.2-7.4	5.6-6.9	6.7- 7.5
Hydraulic loading rate mgd	0.0213	0.0285	0.0423	0.0663	0.0873

Parameter	Pawnee[a]	Bixby[b]	Koshkonong[c]	Windber[d]	North Gulfport[e]
Organic loading rate kg BOD_5/ha/d	151	161	87	285	486
Detention time, d	143	107	72	46	22

[a]Illinois; [b]Oklahoma; [c]Wisconsin; [d]Pennsylvania; [e]Mississippi

Table 6-5. *N* Removal in Aerated Ponds (adapted from U.S. EPA 1983a)

Location	Pawnee, Illinois		Bixby, Oklahoma		Koshkonong, Wisconsin	
Parameter, mg/L	Influent	Effluent	Influent	Effluent	Influent	Effluent
TKN	51.4	5.0	45.0	8.4	15.3	7.6
NH_3	26.3	1.3	29.6	3.5	10.0	5.3
NO_3^-	-	0.8	-	-	1.7	4.4
$NO_2^=$	-	0.1	-	-	0.1	-
Alkalinity	242	161	154	70	397	382
*p*H	6.8-7.4	7.8-9.3	6.1-7.1	6.7-9.2	7.2-7.4-7.9	
Temp °C	-	3-22	-	5-29	-	1-25
DO	-	1.9-16.0	-	3.9-13.5	-	7.6-15.3
Location	**Windber, Pennsylvania**		**N. Gulfport, Mississippi**		**Mt. Shasta, California**	
Parameter, mg/L	Influent	Effluent	Influent	Effluent	Influent	Effluent
TKN	24.3	23.6	26.5	10.8	15.7	11.1
Range	13.2-46.0	14.4-34.1	20.6-30.9	7.2-13.3	10.1-20.9	6.8-14.2
NH_3	22.9	22.9	15.7	5.1	10.3	5.4
NO_3^-	-	0.72	-	2.36	0.3	0.7
$NO_2^=$	-	0.2	-	0.6	0.2	0.5
Alkalinity	67	82	144	102	93	74
*p*H	5.6-6.9	6.8-8.5	6.7-7.5	6.8-7.5	6.5-7.6	7.4-9.7
Temp, °C	-	2 – 24		11 – 29	-	2 - 27
DO	-	5.7-15.0	-	0.8-9.3	-	10.9-14.0

6.3.3 Empirical Design Equations

Table 6-6 contains a summary of selected equations developed to predict NH_3 and TKN removal in diffused air aerated ponds (Middlebrooks and Pano, 1983). All of the equations draw values from the same database. Different combinations of data were chosen and combined to develop several of the equations. The "system" column in Table 6-6 indicates which ponds or series of ponds were used to develop each equation. These data were analyzed statistically and the equations were selected based upon the best statistical fit for the various combinations. It should be noted that the combinations of data sets are not directly comparable.

An examination of the HRT calculated using the various formulas for TKN removal show that the greatest difference between the maximum and minimum detention times calculated from the equation is 14 percent. In view of the variation in methods used, this variability is reasonable. All of the relationships are statistically significant at levels higher than one percent. As a result, any of them may be applied to estimate TKN removal in an aerated pond design. Given the simplicity of the plug flow model and the fraction removed model, it is recommended that one or both be used along with the theoretical models to verify that there will be adequate removal in the event that unusual BOD_5 loading rates are encountered.

Table 6-6. Comparisons of Various Equations to Predict NH_3 and TKN Removal in Diffused-Air Aerated Ponds (Middlebrooks and Pano, 1983).

Equation Used to Estimate HRTor Effluent Concentration	Correlation Coefficient	HRT, d	Compared w Max RT % Difference	System
TKN Removal				
ln C_e/C_0 = 0.0129(Detention Time)	0.911	125	5.3	Ponds 1, 2 and 3
				Mean Monthly Data
TKN Removal Rate				
TKNrr = 0.809(TKN Loading Rate)	0.983	132	0	Total System
				Mean Monthly Data
TKN Removal Rate				
TKNrr = 0.0946(BOD_5 Loading Rate)	0.967	113	14.4	Total System
TKN Fraction Removed				
TKNfr =0.0062(Detention Time)	0.959	129	2.3	Ponds 1, 2 and 3
				Mean Monthly Data
NH_3 Removal				
lnCe/C0=-0.0205(Detention Time)	0.798	79	40.2	All Data
				Mean Monthly Data
NH_3 Removal Rate				
NH_3rr = 0.869(NH_3-N Loading Rate)	0.968	92	30.3	Total System
				Mean Monthly Data
NH_3 Removal Rate				
NH_3rr = 0.0606(BOD_5 Loading Rate)	0.932	132	0	Total System
				Mean Monthly Data
NH_3 Fraction Removed				
NH_3fr = 0.0066(Detention Time)	0.936	121	8.3	Ponds 1, 2 and 3
				Mean Monthly Data

Using any of the above expressions will result in an estimate of the TKN removal that is likely to occur in diffused-air aerated ponds. Unfortunately, data are not available to develop similar relationships for surface aerated ponds.

While the relationships developed to predict NH_3 removal are significant for all of the equations presented in Table 6-6, the agreement between the calculated HRT for NH_3 removal differed significantly from that observed for the TKN data. This is not surprising in view of the variety of mechanisms involved in NH_3 production and removal in wastewater ponds, but it does complicate the use of the equations to estimate NH_3 removal in aerated ponds.

Statistically, any of the expressions may be used to calculate the HRT required to achieve a certain percent reduction in NH_3. Perhaps the best equation to use in designing NH_3 removal is the relationship between the fraction removed and the HRT. The correlation coefficient for this relationship is higher than the correlation coefficient for the plug flow model, and both equations are relatively straightforward to use.

6.3.4 Nitrogen Removal in Continuous Feed Intermittent Discharge (CFID) Basins

6.3.4.1 Description

Rich (1996, 1999) has proposed CFID basins for use in aerated pond systems for nitrification and denitrification. The systems are designed to use in-basin sedimentation to uncouple the solids retention time from the HRT. The influent flow is continuous. A single basin has a dividing baffle to prevent short-circuiting.

6.3.4.2 Applicability of CFID

Some CFID systems have experienced major operational problems with short-circuiting and sludge bulking; however, by minimizing these problems with design changes the systems can be made to function properly. CFID design modifications can be made to overcome most difficulties and details are presented by Rich (1999).

6.3.4.3 Advantages and Disadvantages

When designed and built correctly, the system is capable of producing an effluent in a pond system comparable with activated sludge systems designed for nitrification/denitrification. Experience with the system, however, is limited.

6.3.4.4 Design Criteria

The basic CFID system consists of a single reactor basin divided into two cells with a floating baffle. The two cells are referred to as the influent (Cell 1) and effluent cell (Cell 2). Mixed liquor is recycled from Cell 2 to the head-works to provide a high ratio of soluble biodegradable organics to organisms and the O_2 source is primarily NO_3^-. This approach is used to control bulking. Although some nitrification will occur in the influent cell, the system is designed for nitrification to occur in the effluent cell. Further details of the operation of the CFID systems, see Rich (1999).

6.3.4.5 Performance

Performance data were not available.

6.3.4.6 Limitations
There is little proven design information and operational difficulties have been encountered.

6.3.4.7 Operation and Maintenance
It is expected that maintenance would be the same as that required for other aerobic ponds.

6.3.4.8 Costs
Construction costs would be the same as those for conventional aerobic pond systems. Considerable savings would accrue when comparing the cost to produce an effluent quality with a CFID system with an equivalent effluent produced by an activated sludge system designed for nitrification/denitrification.

6.3.4.9 General Applicability
The Rich (1999) method is a design for nitrification in an aerobic pond. The equations in Table 6-6 are empirical and may or may not apply to a general design. That said, they show what might be expected in terms of N removal. Designing a pond system to nitrify wastewater is not difficult if the water temperature and detention time are adequate to support nitrifiers and sufficient DO is supplied. Recycling of the mixed liquor is a significant benefit. As with all treatment methods, an economic analysis should be performed to determine whether this system is cost effective.

6.3.5 Nitrification Using Fixed Film Media
In addition to the proprietary systems described later in this chapter, Reynolds et al., (1975), Polprasert and Agarwalla, (1995), and Ripple (2002) have conducted studies using baffles and suspended materials as media for attached growth for nitrifying organisms. Nitrification is a function of temperature, and where temperature was a factor in the studies, there was a significant decline in nitrification. It is not clear that the impact of winter temperatures can be overcome when the water temperature drops below 10 °C.

6.3.6 Pump Systems, Inc. Batch Study
6.3.6.1 Description
In 1998, a solar-powered circulator (equivalent to the SolarBee Model SB2500) was installed in an 11.7 ha pond with a depth of 4.5 m at Dickinson, North Dakota with no incoming wastewater. The circulator flow rate was 9463 L (2500 G) per minute. The NH_3 concentration at the beginning of the experiment was approximately 20 mg/L. Dissolved oxygen, *p*H, BOD_5, TSS, NH_3, water temperature and other parameters were measured over a 90-day period at various locations and depths. Over 1500 samples were collected over the test period. Average data for the various locations and depths are shown in Table 6-7. The average water temperature was 20.5 °C. DO was present throughout the pond at all depths, but on occasion dropped to 0.4 mg/L at the bottom. These occasional low DO concentrations may have had an adverse effect on the results presented below, but they do provide some guidance as to how to estimate the expected conversion of NH_3 in a partial mix aerobic pond system (see Equation 6.9).

Table 6-7. Average Values for Batch Test in Pond 4 at Dickinson, North Dakota Area = 11.7 Ha (29 Ac), No Inflow. (Pump Systems, Inc., 2004).

Days	Ln *Ce/Co*	*p*H
0	0	7.7
1	0.15	7.7
7	0.21	7.7
12	0.16	8
13	0.11	8.1
19	-0.06	8.4
26	-0.36	8.8
29	-0.36	8.7
36	-0.6	8.8
42	-0.67	8.6
48	-0.79	8.5
50	-0.82	8.5
57	-0.76	7.7
62	-0.75	8
70	-0.48	8.1
76	-0.62	8
84	-0.74	7.8
90	-0.91	8.1

C_o = influent concentration of (NH_4^+ + NH_3), mg/L as N
C_e = effluent concentration of (NH_4^+ + NH_3), mg/L as N

The reduction in NH_4^+ with time was directly related to the variation in *p*H value (Table 6-7). When the *p*H exceeded 8.0, the reduction in NH_4^+ increased, implying a greater loss of the NH_3 to the atmosphere.

The results of this experiment show the low reaction rate for nitrification that occurs in partial mix aerobic ponds. The reaction rate of 0.0107/d obtained at an average temperature of 20.5 °C in the Dickinson experiments agrees with results obtained with data collected in an aerobic pond located in Wisconsin (Middlebrooks et al., 1982). At 1 °C, the NH_3 conversion reaction rate for the Wisconsin partial mix pond ranged between 0.0035 and 0.0070/ d. Using the average value of 0.005/d at 1 °C and the value of 0.0107/d obtained at Dickinson at 20.5 °C, an approximate value of 1.04 can be inserted into the θ in the classic temperature correction equation: $k_T = k_{20}(1.04)^{(T-20)}$. Example C-6-1 in Appendix C illustrates the effects of reaction rates and temperature on the performance of partial mix pond systems.

6.3.7 Nitrogen Removal in Ponds with Wetlands and Gravel Nitrification Filters

The nitrification filter bed (NFB) was developed by Sherwood C. Reed, and the following material was extracted from Reed et al. (1995). The NFB has been installed at three locations in the United States. The system was developed as a retrofit for existing free water surface (FWS) and subsurface flow (SF) wetland systems having trouble meeting NH_3 effluent standards. Schematic diagrams of both FWS and SF wetlands fitted with NFBs are shown in Figure 6-5. The NFB is a vertical-flow gravel filter bed located on top of existing wetlands. When applied to the FWS wetland, the fine gravel bed is supported on a coarse gravel layer to ensure aerobic conditions in the NFB.

NFB units can be located at the front or near the end of the wetland where wetland effluent is pumped to the top of the NFB and distributed evenly over the surface. Introducing the wetland effluent to the NFB at the head of the system has the advantage of mixing the influent wastewater with the highly nitrified NFB, effluent which results in denitrification and removal of nitrogen from the system. In addition the BOD_5 is reduced, and some of the alkalinity lost during nitrification will be recovered. If the NFB is placed at the end of the wetland, nitrification will occur, but denitrification will be limited and the NO_3^- will pass out of the system. This will require less pumping capacity, but the advantages of denitrification could easily offset the cost of pumping.

Although similar to a recirculating sand filter, the NFB uses gravel rather than sand and can process a much higher hydraulic loading rate than the sand filter. Hydraulic loading rates, including a 3:1 recycle ratio, for a NFB located in Kentucky is 4 $m^3/m^2/d$ (100 $G/ft^2/d$), in contrast to loadings on recirculating sand filters of 0.2 $m^3/m^2/d$ (5 $G/ft^2/d$).

Trickling filter and rotating biological contactor attached-growth concepts were used to develop a design relationship for the NFB (Equation 6-12). The relationship in Equation 6-12 was derived from curve fitting performance data and should be used with caution. The equation should give reasonable estimates of the specific surface area to produce effluent NH_3 concentrations between 0 and 6 mg/L. Equation 6-12 has been verified at a 2 MGD system in Mandeville, Louisiana (Reed et al., 2003).

$$A_v = \frac{2713 - 1115(C_e) + 204(C_e)^2 - 12(C_e)^3}{k_r} \qquad (6\text{-}12)$$

where:

A_v = specific surface area, $m^2/kg\ NH_3/d$
C_e = desired NFB effluent NH_3, mg/L
k_T = temperature-dependent coefficient
$\theta\ (NH_3)$ = 1.048
$\theta\ (NH_4^+)$ = 1.15
At temperatures ≥ 10 ˚ C, $k_T = 1(1.048)^{(T-20)}$

At temperatures 1-10 $^\circ$ C, $k_T = 0.626(1.15)^{(T-10)}$

The following conditions are necessary for good nitrification performance:

- BOD_5:TKN ratio must be less than one
- Sufficient oxygen must be present
- Surface must be moist at all times
- Sufficient alkalinity must be available to support nitrification (approximately 10g of alkalinity per gram of NH_3)

The NFB bed depth ranges from 0.3 - 0.6 m and the bed extends across the entire width of the wetland cell to ensure mixing with the influent wastewater (Fig 6-4). Sprinklers are used to distribute the wetland effluent over the surface of the NFB. In cold climates it may be necessary to enclose the NFB to prevent freezing.

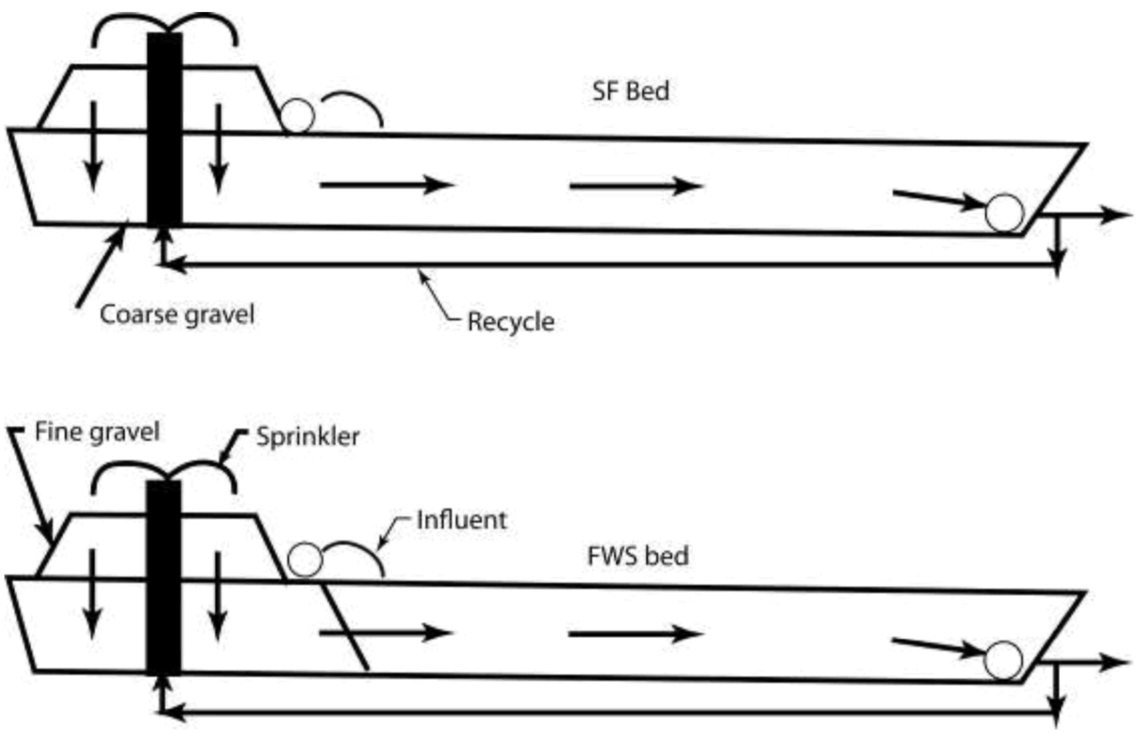


Figure 6-4. Schematic diagram of nitrification filter bed (Reed et al., 1995).

The Benton, Kentucky NFB has been operating successfully at a hydraulic design flow of 3.79 x 10^6 L/d (1 MGD) per day with a wetland input of 20 mg/L NH_3 and an output of 2 mg/L. Performance data for approximately three years for the Benton facility are shown in Table 6-8 (Reed, 2000). Pond influent carbonaceous BOD_5 and NFB effluent carbonaceous BOD_5 are shown in Figure 6-6. The very large concentration values are probably analytical errors because the TSS concentrations for these days were very low. NFB effluent carbonaceous BOD_5, TSS and NH_3 concentration variability are shown in Figure 6-7. Ammonia effluent concentrations were well below 5 mg/L with very few exceptions.

Table 6-8. Benton, Kentucky Recirculating Gravel Filter / Constructed Wetland Operational Data (Reed, 2000).

Date	Carbonaceous BOD_5 (mg/L)		TSS(mg/L)		NH_3 (mg/L)
	Influent	Effluent	Influent	Effluent	Effluent
April 92	137 (3)[a]	4.2 (3)	89 (4)	4.5 (4)	1.5 (4)
May	180 (4)	2.6 (4)	273 (4)	2.8 (4)	2 (4)
June	160 (4)	4.6 (4)	124 (4)	7 (4)	1.5 (4)
July	208 (5)	6.7 (5)	113 (5)	13.8 (5)	1.9 (5)
August	202 (4)	4 (4)	144 (4)	4.5 (4)	1.3 (4)
September	270 (5)	4.9 (5)	164 (5)	5.6 (5)	1.4 (5)
October	153 (3)	5.7 (3)	131 (3)	2.7 (3)	0.7 (3)
November	99 (4)	18 (4)	195 (4)	9 (4)	0.5 (4)
December	216 (3)	54.5 (4)	124 (4)	2.5 (4)	0.9 (4)
January 93	173 (4)	5 (4)	148 (4)	4 (4)	6.8 (4)
February	115 (4)	5.5 (4)	129 (4)	7 (4)	1 (4)
March	73 (5)	3.8 (5)	170 (5)	5 (5)	3.7 (5)
April	178 (4)	6.3 (4)	158 (4)	6.8 (4)	3.8 (4)
May	174 (5)	7.4 (5)	212 (5)	4.8 (5)	1.1 (5)

[a]BOD and TSS averages from April 1992 – May 1993.
Number in parentheses = number of samples.

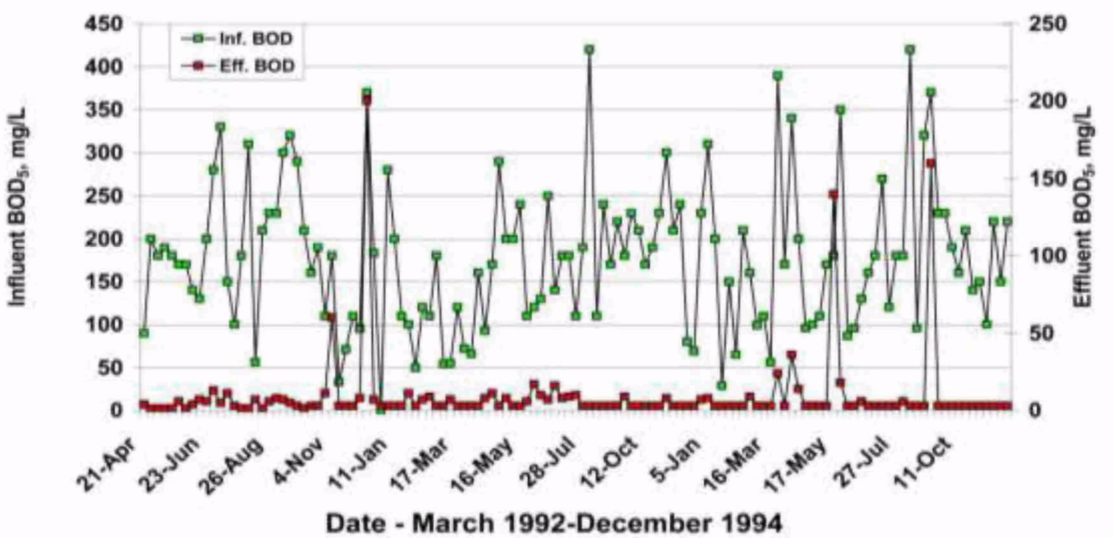


Figure 6-5. Benton performance data for pond + wetland + NFB (Reed, 2000).

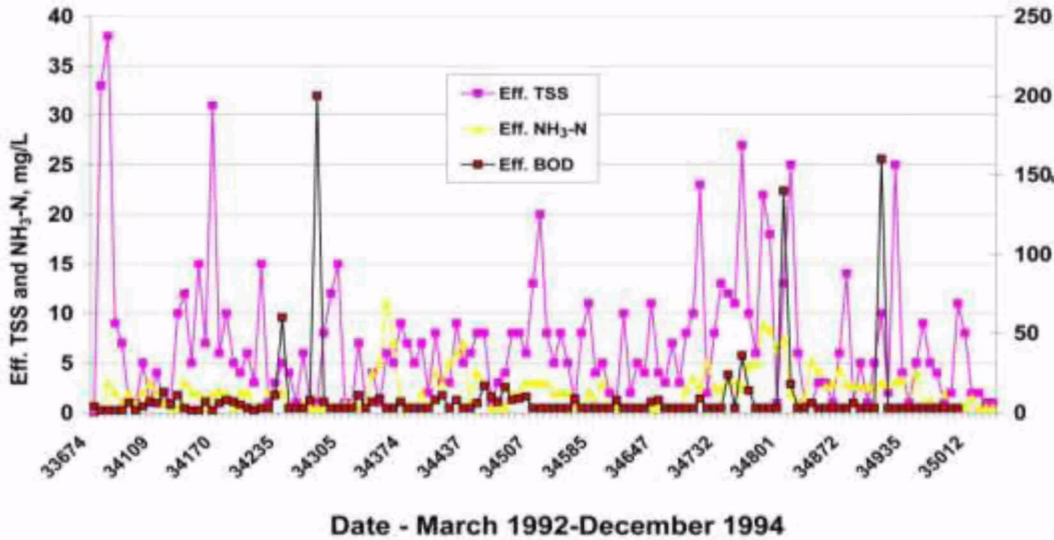


Figure 6-6. Benton performance data for pond + wetland + NFB (Reed, 2000).

When retrofitting an existing pond-wetland system for nitrification and *N* removal, the NFB appears to have economic advantages and simplicity of construction and operation. It also is likely that the NFB would be a viable alternative for *N* removal in the initial design of a pond-wetland system.

6.4 COMMERCIAL PRODUCTS

There are numerous products and processes that are offered as a means to improve pond performance and remove *N*. Several options are presented below (Burnett et al., 2004).

6.4.1 Description of Options

6.4.1.1 Add Solids Recycle

Adding solids recycle can be a reliable method of producing an effluent that can meet stringent NH_3 limits. With the addition of solids recycle, a pond is converted to a low-mixed liquor suspended solids (low-MLSS) activated sludge system. This can be accomplished using an external clarifier and adding a pump to return solids to the headworks. The BIOLAC® process uses an internal clarifier. Effluent from the clarifier is discharged to disinfection or routed through the subsequent cells of the pond system.

Successful operation of low-MLSS activated sludge system requires that the recycled solids be kept in suspension. The aerobic pond must be kept completely mixed. In most cases, a portion of the existing pond is partitioned into a complete mix cell because the energy required to mix the cell is far greater than that required to reduce the BOD_5 or nitrify the NH_3. The remaining portion of the system is used for polishing the effluent or storing the water before discharge.

Because the recycle system is an activated sludge variation, it can be designed and operated with traditional activated sludge design methods. Floating baffle curtains with exit ports are frequently used for cell partitioning. Excess sludge wasting can be accomplished in a separate holding pond, or downstream cells of the existing pond can be used to store and treat sludge for disposal.

A complete mix section can be located anywhere in the flow train of an aerobic pond system. If the complete mix cell is placed near the end of the flow train nitrification occurs after carbonaceous BOD_5 has been removed. With the complete mix zone first in the process, sludge can easily be returned to a manhole or other suitable location upstream of the plant influent. By recycling sludge to the headworks, anoxic conditions and a high food-to-microorganism ratio will help control sludge bulking, provide some denitrification, and recover alkalinity.

6.4.1.2 Converting to a Sequencing Batch Reactor (SBR) Operation

Converting an aerobic pond to an activated sludge system can be accomplished by operating the aerobic pond as an SBR. A portion of the aerobic pond is partitioned into two or more complete mix SBR zones.

SBRs operate in a sequence of fill, react, settle, and decant. In a single-train SBR, flow into the basin will continue through all four cycles. Where parallel systems exist, the SBR can be operated as a typical SBR system; however, the construction costs will be higher. Rich (1999) has referred to this operation as a CFID process, but it is the same as the commercial SBR system marketed by Austgen-Biojet.

In SBR mode, aeration is used intermittently, and a decanting process transports the settled wastewater to downstream facultative cells or to a disinfection chamber before discharge. Decanting is accomplished with pumps, surface weirs, or floating decanting devices. A portion of the low-MLSS must be wasted during the reaction (mixing and aeration) phase to keep the process in balance. Rich (1999) suggests adding a recycle pump station with mixed liquor returned to the influent sewer to provide an anoxic environment for control of sludge bulking.

6.4.1.3 Install Biomass Carrier Elements

The addition of baffles and suspended fabrics for attached growth to accumulate and reduce pollutants has been suggested for many years (Reynolds et al., 1975; Polprasert and Agarwalla, 1995). The availability of commercial fabrics for the removal of NH_3 is a relatively new development. The carriers are plastic ribbons or wheels that are installed in the aerated zone to provide surface area for the growth of microorganisms. Provided that there is adequate surface area, nitrifying microorganisms can grow and multiply on the plastic surfaces and achieve NH_3 removal. The aerated cell does not have to be completely mixed, which is required in the recycle and SBR approaches, but the increased O_2 demand of the attached microorganisms must be met. Solids that drop from the biomass carriers settle or pass to next pond cells. Sludge buildup will increase, but will be reduced by anaerobic digestion, which will minimally affect the frequency at which sludge will need to be removed.

6.4.1.4 Commercial Pond Nitrification Systems

The following is a partial list of pond nitrification systems offered commercially:

1. ATLAS IS™ – Internal clarifier system by Environmental Dynamics, Inc.

2. CLEAR™ Process – SBR variant by Environmental Dynamics, Inc.
3. Ashbrook SBR – SBR system by Ashbrook Corporation
4. AquaMat® Process – Plastic biomass carrier ribbons by Nelson Environmental, Inc.
5. MBBR™ Process – Plastic biomass carrier elements by Kaldnes North America, Inc.
6. Poo-Gloo™ – Wastewater Compliance Systems, Inc.

6.4.1.5 Applicability of Commercial Systems
Experience with all of the systems mentioned above is limited, and it is difficult at this time to predict the applicability and performance.

6.4.1.6 Advantages and Disadvantages
Until more design and operational data become available, it is difficult to delineate differences in the various commercial systems.

6.4.1.7 Design Criteria
Although design criteria were not available, the SBR systems would be designed the same way as a typical SBR. The quantities of plastic biomass required apparently are proprietary information. The design of the MBBR™ process is proprietary, as is the Poo-Gloo™ system.

6.4.1.8 Performance
The limited performance data are presented with the descriptions of the individual processes.

6.4.1.9 Limitations
Because of the limited experience and associated data, it is difficult to assess the effectiveness of the various processes.

6.4.1.10 Operation and Maintenance
With the addition of any of these treatments, it is expected that there will be more operation and maintenance time.

6.4.1.11 Costs
It is expected that the base costs associated with the type of pond into which the commercial equipment was placed in would continue. Additional costs would include the commercial product and any associated operation and maintenance.

6.4.1.12 ATLAS-IS™
Description
The Advanced Technology Pond Aeration System with Internal Separator (ATLAS-IS™) is offered by Environmental Dynamics, Inc. (EDI). It is designed to provide a high level of treatment with minimal operation and maintenance requirements. The process consists of a fine bubble floating lateral aeration system that contains a series of internal clarifiers or settlers. The settlers are constructed of a plastic material and may contain lamellate baffles. The units are installed within a complete mix zone of the aerated pond system. Mixed liquor enters the settling chamber through the bottom. A slight concentration of the low-MLSS takes place in the settler as the mixed liquor rises and spills over a weir into an effluent pipe. No return activated sludge

(RAS) or waste activated sludge (WAS) is required. Over time the low-MLSS will build up to a level adequate to grow nitrifying microorganisms. Some solids are carried downstream so no separate sludge wasting is necessary.

6.4.1.12.1 Performance

The ATLAS-IS™ system has been tested at Ashland, Missouri and has been successful in building up low-MLSS and in achieving nitrification. A schematic of the system is shown on Figure 6-8.

6.4.1.13 CLEAR™ Process

EDI also offers an SBR variant known as the Cyclical Pond Extended Aeration Reactor (CLEAR™). A completely mixed aerated pond cell is partitioned into three zones using floating baffle curtains. Influent is fed to each of the three zones in sequence. Aeration is applied to the zone receiving influent wastewater and, for part of this cycle, one of the other two zones. While the inflowing zone is aerated, the other two zones cycle between settling and decanting. WAS is removed using airlift pumps, either to downstream facultative ponds for storage or for further processing and disposal. A control system is provided to operate the motorized wastewater influent valves and decanters. There are currently no full-scale installations of the CLEAR™ process. A depiction of the process is shown in Figure 6-9.

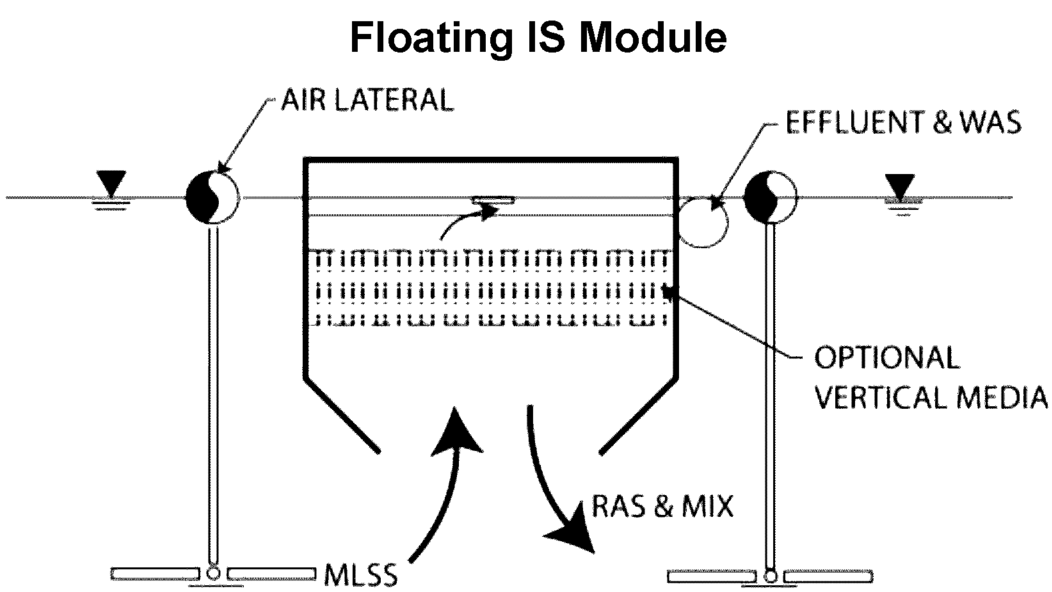


Figure 6-7. EDI ATLAS – IS Internal Pond Settler.

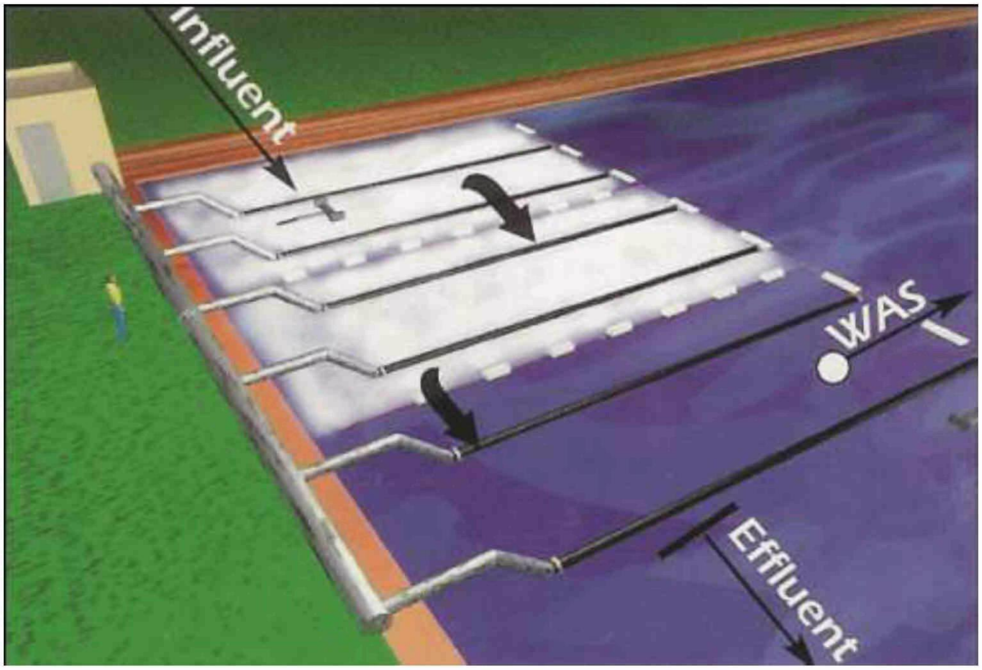


Figure 6.8. CLEAR™ Process.

6.4.1.14 Ashbrook Sequencing Batch Reactor

Description

Ashbrook Corporation (Houston, Texas) Sequencing Batch Reactor (SBR) system consists of decanters, motorized valves, and a control system. A facility has been installed in a pond system in Quincy, Washington. The aerated pond was portioned into sections and air was provided for complete mixing in two or more SBR cells. Operation is similar to a conventional SBR process, and the system in Quincy has been working well. Performance data are presented in Table 6.9. Figure 6-10 is a photograph of the system.

Table 6.9. Data for the Quincy, Washington SBR System (Ashbrook Corporation).

2002-2003 Average Influent/Effluent Data

Flow, MGD = 0.78

	BOD, mg/L	TSS, mg/L	*NH_3, mg/L*
Influent	145	159	19
Effluent	14	6	1.7

Figure 6-9. The Quincy SBR System.

6.4.1.15 AquaMat® Process Description

AquaMat®is a biomass carrier system marketed by Nelson Environmental, Inc., Winnipeg, Manitoba. Plastic ribbons slightly more dense than water are connected to a plastic float, and ribbons extend into the waste stream three feet or more and provide additional surface area for bacteria to grow. When used with pond systems, the application is referred to as the Advanced Microbial Treatment System (AMTS).

6.4.1.15.1 Performance

Year-round nitrification has been achieved in an aerated pond in Laurelville, Ohio, and in Canada. Performance data are shown in Table 6-10 and two views of the AquaMat® are shown in Figure 6-11.

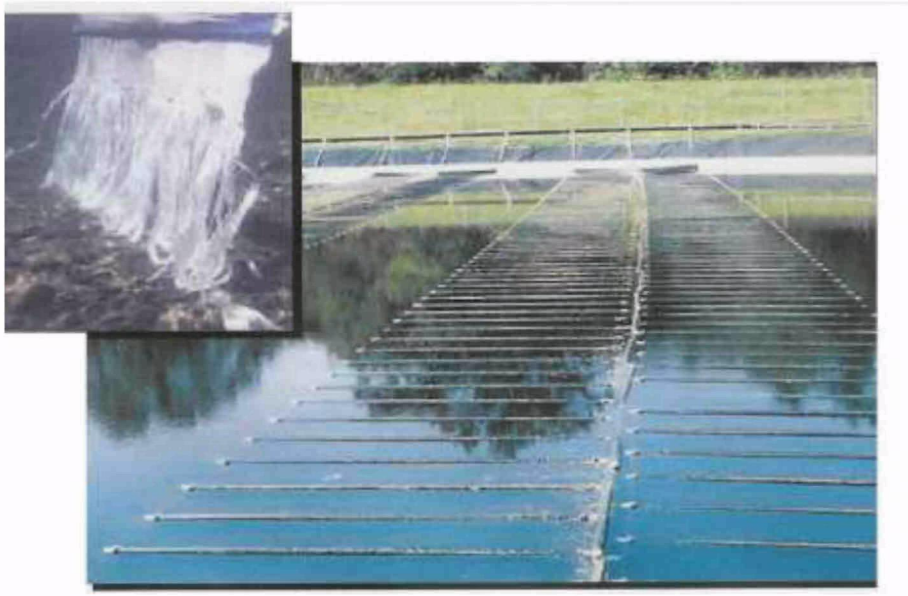


Figure 6-10. The AquaMat® Process

Table 6-10. Nelson AquaMat® Biomass Carrier, Larchmont, Georgia (Burnett et al., 2004).

Reported Average Effluent Quality	
BOD_5 (mg/L)	6
TSS (mg/L)	10
NH_3 (mg/L)	0.1

6.4.1.16 Moving Bed Biofilm Reactor™ Process

6.4.1.16.1 Description

The Moving Bed Biofilm Reactor™ (MBBR™) is marketed by AnoxKaldnes North America, Inc., Providence, Rhode Island. The process is similar to the AquaMat® except that thousands of small polyethylene wheels are suspended in the pond (Figure 6-12). With a sufficient number of the "wheels", adequate surface area is provided for growth of nitrifiers. An aerated pond in Johnstown, Colorado has been successfully upgraded using the MBBR™.

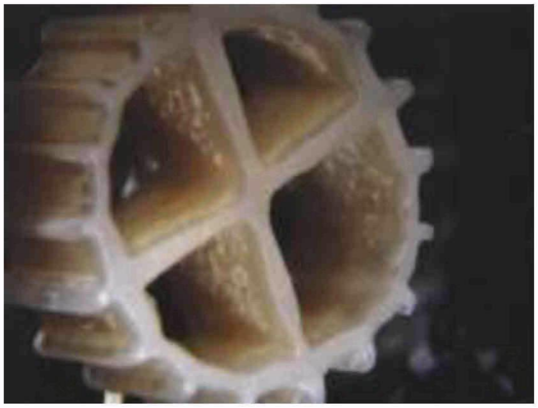


Figure 6-11. A MBBR™ "wheel."

6.4.1.17 Poo-Gloo™ (Wastewater Compliance Systems, Inc.)

This patented device, developed at the University of Utah, consists of several concentrically nested domes that provide substrate for bacteria. They are placed on the bottom of a pond, creating a dark environment with robust air and wastewater mixing which removes contaminants from the water. Figure 6-13 shows a diagram of the device.

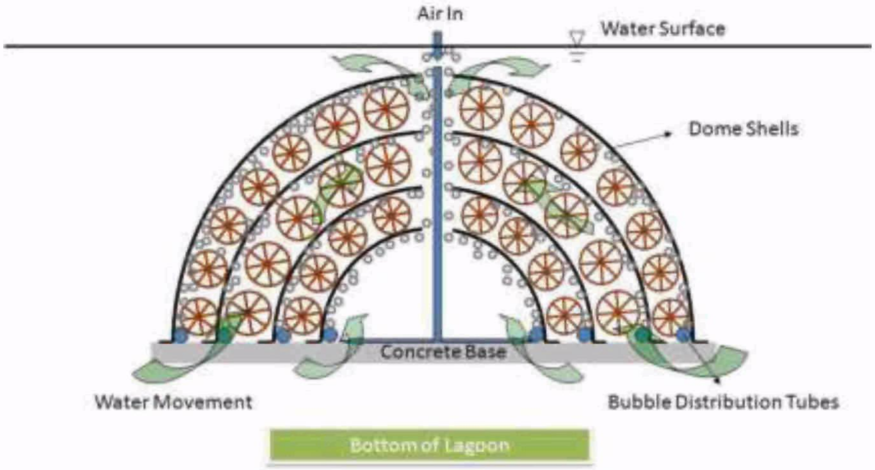


Figure 6-12. Schematic of a Poo-Gloo device cross-section.

6.4.2 Other Processes

Partial denitrification has been achieved by most of the systems described above, although the nitrogen removal pathways are not well understood. Several other commercial SBR systems and biomass carrier systems are available. Their use in ponds appears to be limited. The principle is the same and it appears reasonable to expect these proprietary systems would work. The manufacturers of the products have unique experience working with pond systems. In addition to the products, the companies have experience with floating baffle curtains for partitioning, installation of equipment without removing existing ponds from service, cost-effective and efficient aeration systems for large surface area installations, and optimizing complete mix and partial mix aeration regimes.

6.5 REMOVAL OF PHOSPHORUS

In general, removal of phosphorus (*P)* has not been required for wastewaters that receive pond treatment, but there are a number of exceptions for systems in the north central United States and Canada. It is expected that *P* removal will become a nation-wide requirement. The following sections present what has been done to control the discharge of *P* from wastewater treatment ponds.

6.5.1 Batch Chemical Treatment

In order to meet a *P* requirement of 1 mg/L for discharge to the Great Lakes, an approach using in-pond chemical treatment in controlled-discharge ponds was developed in Canada. Alum, ferric chloride and lime were tested using a motorboat for distribution and mixing of the chemical. A typical alum dosage might be 150 mg/L to produce an effluent from the controlled-discharge pond that contains less than 1 mg/L of *P* and less than 20 mg/L BOD_5 and TSS. The sludge buildup from the additional chemicals is insignificant and would allow years of operation before requiring cleaning. The costs for this method were very reasonable and much less than conventional *P* removal methods. It has been applied successfully in several Mid-western states (U.S. EPA, 1992). The procedure does require a long-term management plan that includes calibrating applications to minimize use and monitoring of sludge quality.

6.5.2 Continuous-Overflow Chemical Treatment

Studies of in-pond precipitation of *P*, BOD_5, and TSS were conducted in Ontario, Canada. The primary objective of the chemical dosing process was to test for the removal of *P* with ferric chloride ($FeCl_3$), alum and lime. Ferric chloride doses of 20 mg/L and alum doses of 225 mg/L, when added continuously to the pond influent, effectively maintained pond effluent *P* levels below 1 mg/L over a 2-year period. Hydrated lime at dosages up to 400 mg/L was not effective in consistently reducing *P* below 1 mg/L (1-3 mg/L was achieved), and yielded no BOD_5 reduction, while slightly increasing the TSS concentration. Ferric chloride reduced effluent BOD_5 from 17 to 11 mg/L and TSS from 28 to 21 mg/L; alum produced no BOD_5 reduction and a slight TSS reduction (from 43 to 28-34 mg/L). Direct chemical addition appears to be effective only for *P* removal.

A six-cell pond system located in Waldorf, Maryland, was modified to operate as two three-cell units in parallel. Alum was added one system, while the other was the control. Each system contained an aerated first cell. Alum addition to the third cell of the system proved to be more efficient in removing total *P*, BOD_5, and TSS than alum addition to the first cell. Total *P* reduction averaged 81 percent when alum was added to the inlet to the third cell and 60 percent when alum was added to the inlet of the first cell. Total *P* removal in the control ponds averaged 37 percent. When alum was added to the third cell, the effluent total *P* concentration averaged 2.5 mg/L, with the control units averaging 8.3 mg/L. Improvements in BOD_5 and TSS removal by alum addition were more difficult to detect, and at times increases in effluent concentrations were observed.

Thirty-seven pond systems in Michigan and Minnesota using chemical treatment to remove *P* were studied (U.S. EPA, 1992). In Minnesota liquid alum was added to the secondary cells of eleven facultative ponds using a motorboat. These ponds were designed with a hydraulic residence time of 180 days and discharge in the spring and fall. The system used is essentially the same as that developed in Ontario, Canada to achieve a total *P* effluent concentration of 1.0 mg/L. Influent concentrations ranged from 1.5 - 6.0 mg/L and averaged approximately 3.3 mg/L. In general, the facilities satisfied the requirement for 1.0 mg/L with several minor excursions outside the limit by 10 percent. In 2010, 38 percent of the treatment ponds in Minnesota were treating the wastewater for *P* removal (Steve Duerre, pers. comm., 2010). The majority use alum, although other chemicals, such as potassium permanganate, are being evaluated as possible substitutes. All of the ponds are able to meet the *P* limit. The chemical is applied twice a year, prior to the seasonal spring and fall discharges.

The State of Michigan evaluated 26 ponds that had been in operation ranging from 1 to 20 years. Both facultative and aerated ponds were evaluated. Discharge was seasonal (once or twice a year) or continuous, (varying from 24 hours/day, 7 days/week to 8 hours/day, 5 days/week). The chemicals were added to a clarifier following the pond system. None of the systems applied the chemicals by boat. The influent total *P* concentration in the Michigan systems ranged from 0.5 to 15 mg/L, with an average of 4.1 mg/L. By 2010, more than 300 ponds in Michigan have or will have a *P* limit of 1 mg/L (Dan Holmquist, pers. comm.). Phosphorus removal has been successful as long as the chemical flocculant is added at the appropriate rate at the end of the pond system, i.e., the clarifier or the maturation pond.

A description of the facilities and the influent and effluent *P* concentrations is shown in Table 6-11.

Table 6-11. Phosphorus Removal in Ponds (from U.S. EPA, 1992).

Location	Discharge schedule	Chemical treatment	Facility description	Phosphorus (mg/L)	
				Influent	Effluent
Belding[a]	Continuous	Alum	5-cell pond	4.0	0.6 - 0.7
Bessemer[a]	Continuous	Alum polymer	3-cell pond with clarifier	1.8 – 2.0	0.6 – 0.9
Coopersville[a]	Continuous	$FeCl_3$	4-cell pond	5.0	0.3
Kalamazoo Lake[a]	Continuous	$FeCl_3$ polymer	3-cell pond	6.0 – 7.0	0.5 – 0.6
Elk Rapids[a]	Continuous	$FeCl_3$	3-cell pond with clarifier	3.2 – 4.3	0.6 – 0.7
Carson City[a]	Seasonal	Alum	5-celll pond	6.0 – 7.0	4.0
Fowlerville[a]	Seasonal	$FeCl_3$	6-cell pond	2.5 – 3.5	0.8
Remus[a]	Seasonal	$FeCl_3$	4-cell pond	4.7	0.4 – 1.0
Serpent Lake[b]	Seasonal	Alum	3-cell pond	1.8 – 2.8	0.6 – 0.7
Grand Portage[b]	Seasonal	Alum	2-cell pond	2.9 – 3.3	0.3 – 1.2

[a]Michigan, [b]Minnesota

CHAPTER 7

UPGRADING POND EFFLUENTS

7.1 INTRODUCTION

There are two general ways to upgrade pond effluents: adding a solids removal step or making modifications to the pond process. The selection of the appropriate method to achieve a desired effluent quality depends upon the design conditions and effluent limits imposed on the facility. The various methods are discussed in the following sections: Solids Removal Methods and Operation Modifications and Additions to Typical Designs.

7.2 SOLIDS REMOVAL METHODS

The occasional high concentration of TSS in the effluent, which can exceed 100 mg/L, has been a major operational challenge to pond systems. The solids are composed primarily of algae and other pond detritus, not wastewater solids. These high concentrations usually occur, during the summer months. Solids removal mechanisms include the use of intermittent sand filters, recirculating sand filters, rock filters, coagulation-flocculation and dissolved air flotation. Nolte & Associates (1992) conducted a review of the literature covering recirculating sand filters and intermittent sand filters.

7.2.1 Intermittent Sand Filtration

Intermittent sand filtration applies pond effluent to a sand filter bed on a periodic or intermittent basis. The use of intermittent sand filters has a long and successful history of treating wastewaters (Massachusetts Board of Health, 1912; Grantham et al., 1949; Furman et al., 1955). A summary of the design characteristics and performance of several systems employed in Massachusetts around 1900 is presented in Table 7-1. These systems were treating raw or primary effluent wastewater and producing an excellent effluent. A typical intermittent sand filter is shown in Figure 7-1.

Table 7-1. Design and Performance of Early Massachusetts Intermittent Sand Filters (Mass. Board of Health, 1912; Mancl and Peeples, 1991).

Location	Year Started	Loading Rate (gal/d/ac)	Filter Depth (in)	Sand Size (mm)	Ammonia Removal		BOD_5 Removal	
					Influent (mg/L)	Effluent (mg/L)	Influent (mg/L)	Effluent (mg/L)
Andover	1902	3500	48-60	0.15-0.2	-	-	-	-
Brockton	-	-	-	-	40.7	1.5	314	6.2
Concord	1899	83,000	-	-	-	-	-	-
Farmington	-	-	70	0.06-0.12	27.3	2.7	-	-
Gardner	1891	122,000	60	0.12-0.18	21.2	7.5	139	9.5
Leicester	-	-	-	-	-	-	321	13.1
Natick	-	-	-	-	12.4	2.3	-	-
Spencer	1897	61,000	48	0.18-0.34	16	2.1	116	6.9

See p. xiv for conversion table.

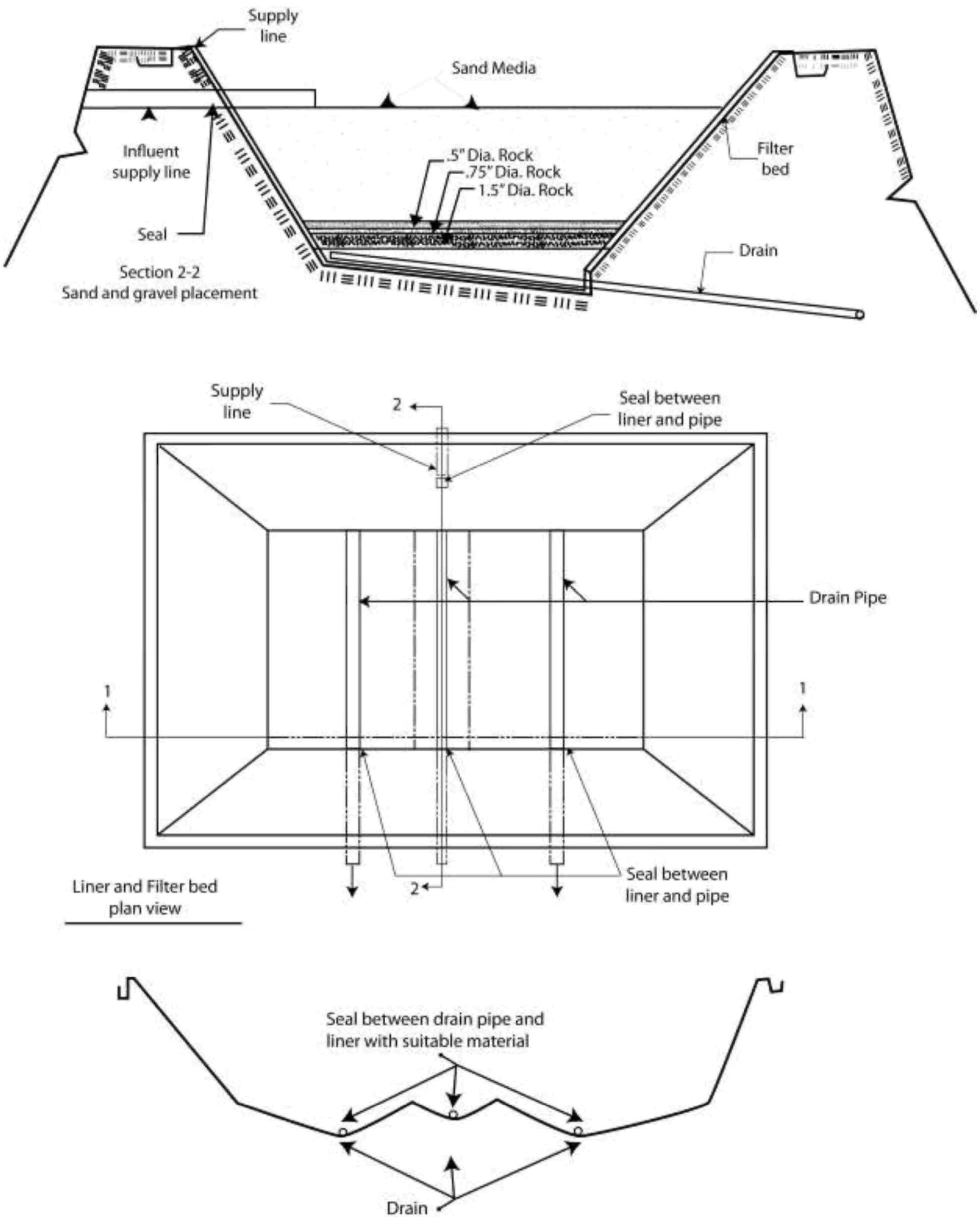


Figure 7-1. Cross-sectional and plan views of a typical intermittent sand filter (U.S. EPA, 1983a).

Intermittent sand filtration is capable of polishing pond effluents at relatively low cost and is similar to the practice of slow sand filtration in potable water treatment. As the wastewater passes through the bed, TSS and other organic matter are removed through a combination of physical straining and biological degradation processes. The particulate matter collects in the top 5 - 8 cm (2 - 3 in) of the filter bed. This accumulation eventually clogs the surface and prevents effective infiltration of additional effluent. At that time, the bed is taken out of service, the top layer of clogged sand removed, and the unit is put back into service. The removed sand can be washed and reused or discarded.

7.2.1.1 Summary of Performance

Summaries of the performance of intermittent sand filters treating pond effluents conducted during the 1970's and 1980's are presented in Tables 7-2 and 7-3. Table 7-2 is a summary of studies reported in the literature and EPA documents, and Table 7-3 is a summary of results from field investigations at three full-scale systems consisting of ponds followed by intermittent sand filters. These are the most extensive studies conducted in the US. Though there are some effluent concentration above the 30/30 (TSS/BOD_5 mg/L) limit, on the whole, the results demonstrate that it is possible to produce an effluent with TSS and BOD_5 less than 15 mg/L from anaerobic, facultative and aerated ponds followed by intermittent sand filters with effective sizes less than or equal to 0.3 mm.

It should be noted that Mt. Shasta Wastewater Treatment Plant retired the intermittent sand filter bed and has been using dissolved air flotation to remove algae since 2000 (see Section 7.2.5). The treatment process consists of headworks, four oxidation ponds, ballast lagoon dosing basin, dissolved air flotation system, intermittent backwash filter, chlorine contact chamber, declorination system and discharge line. The treated wastewater can be discharged to any of three locations, depending on water quality and time of year: the Sacramento River, a leach field located adjacent to Highway 89, or the Mt. Shasta Resort Golf Course (http://ci.mt-shasta.ca.us/publicworks/wastewater.php). The intermittent sand filter bed was determined to be too labor intensive, although it worked fairly well (Jackie Brown, pers. comm., 2010).

Table 7-2. Intermittent Sand Filter Performance Treating Pond Effluents[a].

Pond Type	UC[b]	Loading Rate	TSS Inf.	TSS Eff.	TSS Rem.	VSS Inf.	VSS Eff.	VSS Rem.	BOD Inf.	BOD Eff.	BOD Rem.	Reference
		mgd/ac	mg/L	mg/L	%	mg/L	mg/L	%	mg/L	mg/L	%	
Facultative [c]	5.8	0.1	13.7	4.0	71	9.2	2.0	78	6.3	1.2	82	Marshall and Middlebrooks 1974
		0.2	13.7	4.8	65	9.2	21	77	6.3	1.3	80	
		0.31	13.7	6.0	56	9.2	2.3	75	6.3	2.0	69	
Facultative	9.74	0.2	30.0	3.5	88	23.0	1.3	94	19.5	1.9	90	Earnest et al., 1978
		0.4	30.1	2.9	90	22.5	1.3	94	19.5	1.9	90	
		0.6	34.0	5.9	83	25.9	3.1	88	25.6	4.2	84	
		0.8	23.9	4.7	80	15.2	1.2	92	2.8	1.8	36	
		1.0	28.5	5.1	82	21.5	2.5	88	13.5	2.6	81	
Facultative	6.2	0.5	32.4	8.6	74	21.9	3.3	85	10.7	1.8	83	Hill et al., 1977
		1.0	32.4	7.8	76	21.9	3.2	85	10.7	2.0	82	
		1.5	32.4	6.4	80	21.9	3.3	85	10.7	2.3	79	
Facultative [c]	9.73	0.25	70.7	10.1	86	38.8	6.5	83	20.2	6.6	67	Bishop et al., 1977
		1.0	68.7	32.9	52	36.6	11.3	69	19.6	11.7	40	
Aerated	9.73	0.5	158	52.5	67	71.1	13.2	81	34.4	5.1	85	Bishop et al., 1977
		1.0	68.7	32.9	52	36.6	11.3	69	19.6	11.7	40	
Anaerobic	NA	0.1	353	45.5	87	264	28.1	84	123	19.5	84	Messinger, 1976
		0.35	208	46.5	78	162	35.3	78	108	43.7	60	
		0.5	194	45.1	77	175	35.7	80	107	67.6	37	
Facultative	9.7	0.2	23.0	2.7	88	17.8	1.0	95	10.9	1.1	90	Tupyi et al., 1979
		0.4	20.8	3.5	83	18.5	2.3	88	11.5	2.6	77	

TSS = Total suspended solids; VSS = Volatile suspended Solids; BOD = Biochemical oxygen demand
[a]Results for best overall performing 0.17 mm effective size (e.s.) filters
[b]U.C. = Uniformity constant
[c]Dairy waste

Table 7-3. Mean Performance Data for Three Full-Scale Pond-Intermittent Sand Filter Systems (Russel et al., 1979 in Crites, 2006).

Parameter	Mt. Shasta CA			Moriarty NM			Ailey GA		
	Pond Eff	Filter Eff	Facility Eff	Pond Eff	Filter Eff	Facility Eff	Pond Effluent	Filter Effluent	Facility Effluent
BOD_5 (mg/L)	22	11	8	30	17	17	22	8	6
Soluble BOD_5 (mg/L)	7	4	5	17	16	16	10	6	5
TSS (mg/L)	49	18	16	81	13	13	43	15	13
VSS (mg/L)	34	13	10	64	9	9	32	8	6
FC (col/100ml)	292	30	<2	290	18	34	55	8	<1
pH	87	68	66	8.9	8.0	8.0	8.9	7.1	6.8
DO (mg/L)	12.4	5.5	5.3	10.9	8.3	8.3	10.2	7.4	7.9
COD (mg/L)	100	87	68	84	43	43	57	32	25
Soluble COD (mg/L)	71	64	50	67	34	34	41	23	16
Akl (mg/L as $CaCO_3$)	75	51	42	293	260	260	84	76	69
TP (mg-P/L)	3.88	3.09	2.72	4.02	2.8	2.8	3.10	2.67	2.45
TKN (mg-N/L)	11.1	7.5	5.2	22	121	121	7.3	4.1	2.2
NH_3 (mg-N/L)	5.56	1.83	1.76	16	9.16	9.16	0.658	0.402	0.31
Org-N (mg N/L)	56	5.7	3.4	5.7	3.3	3.3	6.7	3.8	1.9
$NO_2^=$ (mg-N/L)	0.56	7.7	0.020	159	1.66	1.66	0.56	77	0.020
NO_3^- (mg-N/L)	0.78	43	45	0.09	4.09	4.09	349,175	21583	29360
Total Algal Count	$4x10^5$	$1x10^5$	$1x10^5$	$8x10^5$	$3x10^4$	$3x10^4$	NA	NA	0.070
Flow (mgd)	NA	NA	0.488	NA	0.046	NA			

NA = Not Available

Rich and Wahlberg (1990) evaluated the performance of five facultative pond-intermittent sand filter systems located in South Carolina and Georgia. A summary of the design characteristics and performance of these systems is shown in Table 7-4. The systems provided superior performance when compared with ten aerated pond systems not using intermittent sand filtration. Six of the 10 aerated pond systems consisted of one aerated cell followed by a polishing pond; three were designed as dual-power (aeration reduced in succeeding cells), multi-cellular systems, and one was a single cell dual-power system. Using data reported by Niku et al. (1981), the performance of the facultative pond-intermittent sand filter systems compared favorably with activated sludge plants.

Table 7-4. Design Characteristics and Performance of Facultative Pond-Intermittent Sand Filter Systems (Rich and Wahlberg, 1990).

Design Flow	Present Flow	HRT	Filter Dosing[a]	BOD_5		TSS		NH_3	
m^3/L	% of Design	d	$m^3/m^2/d$	gm/m^3 50%	gm/m^3 95%	gm/m^3 50%	gm/m^3 95%	gm/m^3 50%	gm/m^3 95%
303	56	93	0.03	9	28	12	41	0.9	4
303	79	70	0.37	6	22	7	29	0.4	1.2
568	48	59	0.47	7	17	11	30	-	-
378	66	52	0.37	9	21	11	25	0.9	2.4
568	37	55	0.31	6	17	6	16	1.3	5.4

[a]Based on design flow rate

Truax and Shindala (1994) reported the results of an extensive evaluation of facultative pond-intermittent sand filter systems using four grades of sand with effective sizes of 0.18 - 0.70 mm and uniformity coefficients ranging from 1.4 - 7.0 (Appendix C, Tables C-7-1 and C-7-2). Performance was directly related to the effective size of the sand and hydraulic loading rate. With effective size sands of 0.37 mm or less and hydraulic loading rates of 0.2 $m^3/m^2/d$, effluents with BOD_5 and TSS of less than 15 mg/L were obtained. TKN concentrations were reduced from 11.6 mg/L to 4.3 mg/L at the 0.2 $m^3/m^2/d$ loading rate. The experiments were conducted in a mild climate, and it is not known whether similar *N* removal rates would be achieved during cold months in more severe climates.

Melcer et al. (1995) reported the performance of a full-scale aerated pond-intermittent system located in New Hamburg, Ontario, that had been in operation since 1980. Results for 1990 and for January to August of 1991 are presented in Table 7-5. Surface loading rates for both periods were 3.24 $m^3/m^2/d$, with influent BOD_5, TSS and TKN concentrations of 12, 16 and 19 mg/L, respectively. Filter effluent quality exceeded requirements with BOD_5, TSS and TKN concentrations being less than 2 mg/L.

Table 7-5. Performance of Aerated Pond-Intermittent Sand Filter, New Hamburg, Ontario Plant (Melcer et al., 1995).

Location in System	Parameter	1990	1991 (Jan-Aug)
	Average Flow Rate (m^3/d)	1676	1673
	Max Flow Rate (m^3/d)	4530	3990
Influent	BOD_5, mg/L	186	120
	TSS, mg/L	314	171
	TKN, mg/L	45	44
	TP, mg/L	9.3	9.5
Aerated Cell	HRT (d)	7	7
	BOD_5 Loading (kg/m^3/d)	0.03	0.02
Aerated Cell Effluent	BOD_5, mg/L	34	36
	TSS, mg/L	44	44
	TP, mg/L	6	5
Facultative Pond	HRT (d)	165	165
	Avg. BOD_5 loading (kg/1000 m^2/d)	0.51	0.55
Cell 2 Effluent	BOD_5, mg/L	12	11
	TSS, mg/L	16	18
	TKN, mg/L	19	18
	NH_3, mg/L	15	14
	T*N*, mg/L	1.1	0.8
	T*P*, mg/L	1.2	0.7
Filter	Annual Surface Loading, m^3/m^2	195	153
	Surface Loading, L/m^2/d	3240	3240
		Mar-Dec	**Mar-Aug**
Filter Effluent	BOD_5, mg/L	2	2
	TSS, mg/L	1.7	1.1
	TKN, mg/L	2	1.1
	NH_3, mg/L	1.2	0.6
	T*N*, mg/L	7	9
	T*P*, mg/L	0.5	0.4

7.2.1.2 Operating Periods

The length of filter run is a function of the effective size of the sand and the quantity of solids deposited on the surface of the filter. EPA (1983a) and several publications (Marshall and Middlebrooks, 1974; Messinger, 1976; Earnest et al., 1978; Hill et al., 1977; Bishop et al., 1977; Tupyi et al., 1979; Russel et al., 1983) contain extensive information on the relationship between solids deposited on the surface of a filter and the length of run time. Truax and Shindala (1994) also reported similar run times.

7.2.1.3 Maintenance Requirements

Maintenance is directly related to the quantity of solids applied to the surface of the filter, and this is related to the concentration of solids in the influent to the filter and the hydraulic loading rate. Filters with low hydraulic loading rates tend to operate for extended periods. With such extended operating periods, maintenance consists of routine inspection of the filter, removing weeds, and an occasional cleaning by removing the top 5 - 8 cm of sand after allowing the filter to dry out. Early control of weeds is the key to good maintenance. The use of chemicals is not advised. In Wisconsin, where there are many sand filters, the O&M manuals advise that the sand beds can be tilled if the weeds are very small. Once they have grown, however, they need to be removed manually (Jack Saltes, Wisconsin Department of Natural Resources, pers. comm., 2010).

7.2.1.4 Hydraulic Loading Rates

Typical hydraulic loading rates on a single-stage filter range from 0.37 - 0.56 $m^3/m^2/d$. If the TSS in the influent to the filter routinely exceeds 50 mg/L, the hydraulic loading rate should be reduced to 0.19 - 0.37 $m^3/m^3/d$ to increase the filter run. In cold weather locations, the lower end of the range is recommended to avoid having to clean the filter during the winter months.

7.2.1.5 Design of Intermittent Sand Filters

Algae removal from pond effluent is almost totally a function of the sand size used. With a required BOD_5 and TSS below 30 mg/L, a single-stage filter with medium sand (effective size = 0.3 mm) will produce a reasonable filter run. If better effluent quality is required, finer sand (effective size = 0.15 - 0.2 mm) or a two-stage filtration system with the finer sand in the second stage should be used.

The total filter area required for a single-stage operation is calculated by dividing the expected influent flow rate by the hydraulic loading rate selected for the system. One spare filter unit should be included to permit continuous operation, since the cleaning process may require several days. An alternate approach is to provide temporary storage in the pond units. Three filter beds are the preferred arrangement to permit maximum flexibility. In small systems that depend on manual cleaning, the individual bed should not be bigger than about 90 m^2. Larger systems with mechanical cleaning equipment could have individual filter beds up to 5000 m^2.

The design depth of sand in the bed should be at least 45 cm with a sufficient depth for at least one year of cleaning cycles. A single cleaning operation may remove 2.5 - 5 cm of sand. A 30-day filter run would then require an additional 30 cm of sand. In the typical case, an initial bed depth of about 90 cm of sand is usually provided. A graded gravel layer 30 - 45 cm separates the sand layer from the under drains. The bottom layer is graded so that its effective size is four times as great as the openings in the under-drain piping. The successive layers of gravel are progressively finer to prevent intrusion of sand. An alternative is to use gravel around the underdrain piping and then a permeable geo-textile membrane to separate the sand from the gravel. Further details on design and performance are presented in the U.S. EPA (1983a), Reed et al. (1995) and Crites et al. (2006). A design example for an intermittent sand filter treating a pond effluent is presented in Example C-7-1 in Appendix C.

7.2.2 Rock Filters

A rock filter operates by allowing pond effluent to travel through a submerged porous rock bed, causing algae to settle out on the rock surfaces as the liquid flows through the void spaces. The accumulated algae are then biologically degraded. Algae removal with rock filters has been studied extensively at Eudora, Kansas; California, Missouri; and Veneta, Oregon (USEPA, 1983a). Rock filters have been installed throughout the United States and the world, and performance has varied (USEPA, 1983a; Middlebrooks, 1988; and Saidam et al., 1995). A diagram of the Veneta rock filter is shown in Figure 7-2. The West Monroe, Louisiana rock filters were essentially the same as the one in Veneta, but the filters received higher loading rates. Several rock filters of various designs have been constructed in Illinois with varied success. Many of the Illinois filters produced an excellent effluent, but the designs varied widely (Menninga, pers. comm., 1986). Figure 7-3 contains diagrams of the various types of rock filters in use in Illinois. Snider (pers. comm., 1998) designed a rock filter for Prineville, Oregon and knew of one built at Harrisburg, Oregon. Performance and design detail are not available, but Snider indicated that the systems were designed using information from the Veneta system.

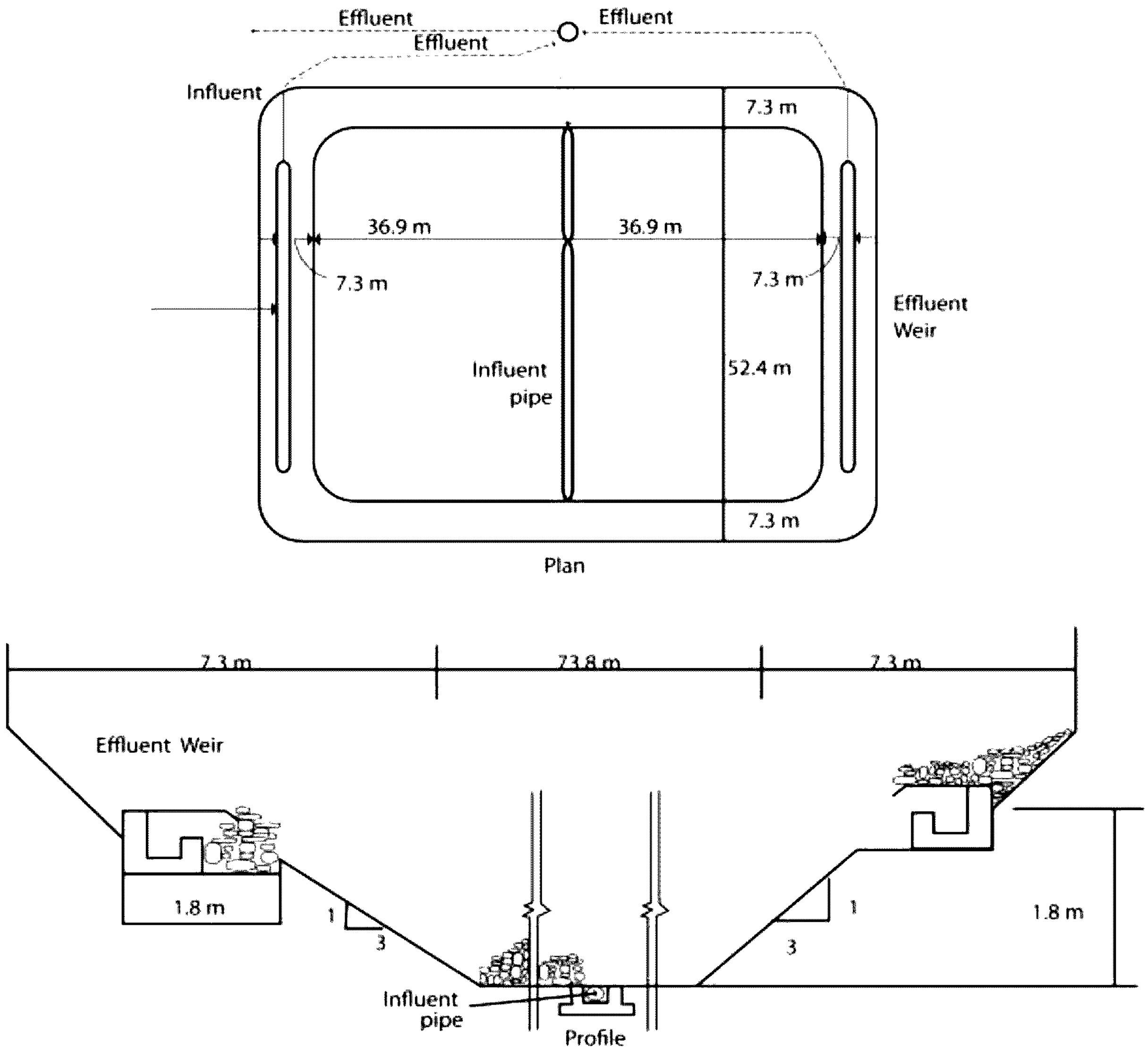


Figure 7-2. Rock filter at Veneta, Oregon (Swanson and Williamson, 1980).

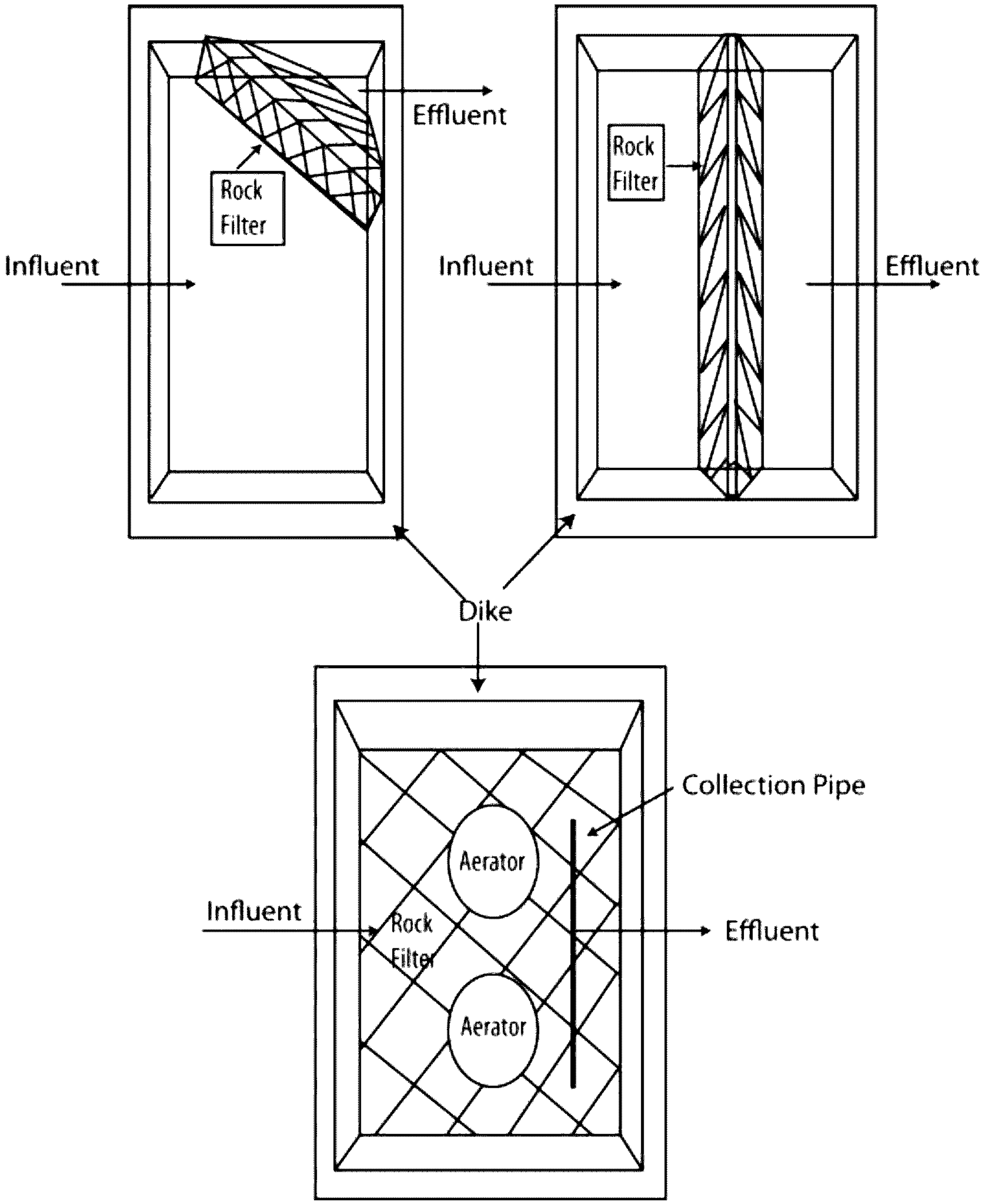


Figure 7-3. State of Illinois rock filter configurations (Menninga, pers. comm.,1986).

The principal advantages of the rock filter are the relatively low construction cost and simple operation. Odor problems can occur, and the design life for the filters and the cleaning procedures has not yet been firmly established. Several units have been operating successfully for over 20 years.

Archer and O'Brien (2005) have used inter-pond rock filters to improve suspended solids and nitrogen removal. Rock embankments across the ponds provide filtering, reduced short-circuiting, and increased surface area to grow nitrifying bacteria.

7.2.2.1 Performance of Rock Filters

7.2.2.2 Veneta, Oregon

Based on data from filter systems in place in Veneta, it can be concluded that rock filter performance is mixed. Forms of *N* in the effluent from a study by Swanson and Williamson (1980) for the Veneta system are shown in Figure 7-4. Performance data for 1994 are shown in Table 7-6. After approximately 20 years of operation, the system was producing an effluent meeting secondary standards with regard to BOD_5, TSS and fecal coliform. Ammonia data were not collected routinely as it was not included in the discharge permit. Ammonia data were only collected on a regular basis during the winter months of the Swanson and Williamson (1980) study, and high NH_3 concentrations were observed in the effluent as shown in Figure 7-4. Occasional NH_3 measurements were made after the Swanson and Williamson study, and higher concentrations were observed during the winter, indicating that the process may not be suitable if a discharge must meet NH_3 effluent limits.

Table 7-6. Mean and Range of Performance Data for Veneta Wastewater Treatment Plant, 1994.

Constituent	Influent	Effluent
BOD_5, mg/L	138 (50-238)	17 (5-30)
TSS, mg/L	124 (50-202)	9 (2-27)
FC, MPN/100 mg/L	Not available	<10 (<10-20)
Flow, mgd	0.251 (0.159-0.452)	0.309 (0.079-0.526)

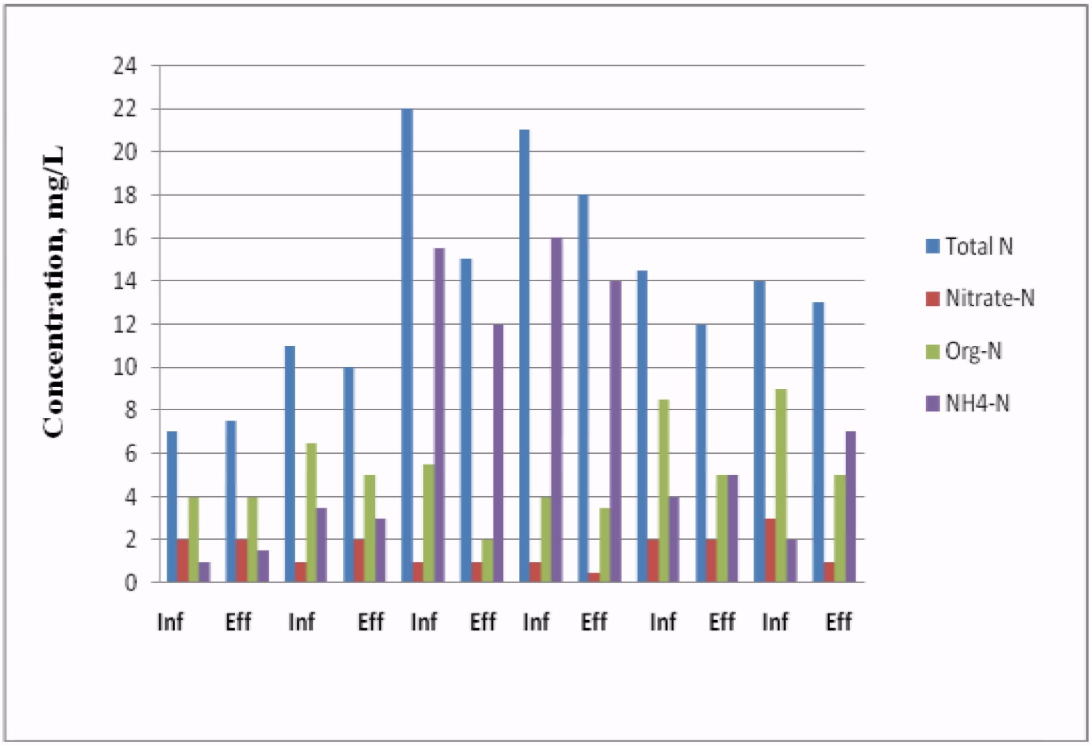


Figure 7-4. Nitrogen species in Veneta wastewater treatment rock filter. Nov-77; Jan, Feb, Mar, Apr and May-78 (Swanson and Williamson, 1980).

7.2.2.3 West Monroe, Louisiana

Stamberg et al. (1984) presented performance results for the two rock filters operating in West Monroe, Louisiana. The systems were loaded at higher hydraulic loading rates than that used for the Veneta facility (<0.3 m^3 of wastewater/d per rock m^3), and the TSS removals were less than those reported for the Veneta system. In general, the West Monroe systems produced effluent BOD_5 and TSS concentrations less than 30 mg/L, but while there were occasional exceedances of BOD_5 to 40 mg/L and TSS to 50 mg/L, only 12 out of over 100 samples exceeded 30 mg/L for either parameter. The design flow rates on the West and East filter were 3.5 and 1.8 mgd, respectively, and the flow rates frequently exceeded the design rate by a factor of 2 to 3. This resulted in an increase in the loading rate by a factor of 2 to 3, which greatly exceeded the Veneta loading rate.

7.2.2.4 Jordan Rock Filters

Saidam et al. (1995) performed a series of studies of rock filters treating pond effluent in Assram, Jordan. The filters were arranged in three trains, the first train consisting of two filters in series, with the first filter containing rock and having an average diameter of 18 cm followed by a filter containing local gravel (wadi gravel) with an average diameter of 11.6 cm. The second train contained the same rock as used in the first filter, but with an average diameter of 2.4 cm. The wadi gravel was used in the first filter of the third train, and the second filter contained an aggregate with an average diameter of 1.27 cm. The filters in the three trains were operated in series, and the characteristics of the wastewater, hydraulic loading rates, and the characteristics of the effluents from the various filters are shown in Table 7-7. The removal efficiencies obtained in the first run for the various filters and the trains are summarized in Table 7-8. Even though the rock sizes of several of the filters were similar to what was used at Veneta and West Monroe, the hydraulic loading rates exceeded the maximum recommended value of 0.3 $m^3/m^3/d$ and the quality of the effluents was much lower. There was insufficient DO in the influent to oxidize NH_3, and considering the temperature of the influent wastewater and the H_2S in the effluent, it is likely that the filters were anaerobic. On the other hand, TSS was lowered by 60 percent and fecal coliform levels met WHO guidelines for unrestricted use of the effluent for agricultural purposes (WHO, 2003).

Table 7-7. Performance of Rock Filters (Saidam et al., 1995).

Unit	Hydraulic Loading Rate m^3/m^3-d	Run	T °C	DO mg/L	TSS mg/L	BOD_5 mg/L	TFCC mpn/ 100mg/L	NH_4-N mg/L
INFLUENT		1	25.7	3.2	201	95	1.10E+04	85
		2	21	4.8	234	105	6.3E+04	93
		3	14.0	4.0	213	122	9.6+05	97
		4	15.0	3.5	101	108	1.6W+04	71
FIRST TRAIN					131	61	2.2E+03	
Rock Filter 1	0.498	1	25.1	1.2	200	81	5.7E+04	89
Avg. Diameter – 18 cm	0.634	2	20.0	1.5	156	100	8.1E+05	96
Voids=49%	0.5 to .58	3	13.4	1.0	76	77	1.4E+04	96
Surface Area =17 m^2/m^3	0.5 to .58	4	13.0	2.1				72
					78	36	1.00E+03	
Wadi Gravel Filter 1	0.386	1	25.2	1.9	161	66	4.2E+04	91
Avg. Diameter=11.6 cm	0.634	2			129	77	4.7E+05	
Voids=41%	0.5 to .58	3	13.4	1.0	66	74	1.10E+04	97
Surface Area =25 $^2/m^3$	0.5 to .58	4	13.0	1.9				71
SECOND TRAIN					130	53	1.9E+03	
Rock Filter 2	0.311	1	25.3	1.1	203	79	5.00E+04	89
Avg. Diameter = 18 cm	0.634	2	19.7	1.4	164	87	8.6E+05	98
Voids=49%	0.5 to .58	3	13.3	1.0	88	92	1.00E+04	98
Surface Area =17 m^2/m^3	0.5 to .58	4	13.7	1.9				71
					102	51	1.5E+03 E	
Coarse Aggregate Filter 2	0.333	1	25.6	1.7	154	65	3.2E+04	89
Avg. Diameter=2.4 cm	0.634	2	19.9	1.4	134	73	5.4E+05	98
Voids=40%	0.5 to .58	3	13.3	1.0	60	87	6.5E+03	97
Surface Area =150 $^2/m^3$	0.5 to .58	4	15.0	1.9				71
THIRD TRAIN					109	48	1.6E+03	
Wadi Gravel Filter 3	0.274	1	25.7	1.6	206	76	6.8E+04	91
Avg. Diameter=11.6 cm	0.634	2	20.2	1.4	150	86	3.2E+05	96
Voids=41%	0.5 to .58	3	13.3	1.0	81	76	6.3E+03	97
Surface Area =25 $^2/m^3$	0.5 to .58	4	15.0	1.9				71
					79	42	6.4E+02	
Medium Aggregate Filter 3	0.442	1	25.9	2.0	121	72	3.3E+04	92
Avg. Diameter=1.27 cm	0.634	2	19.7	1.5	108	66	4.4E+05	96
Voids=28%	0.5 to .58	3	13.4	1.0	45	59	3.3E+03	100
Surface Area =327 $^2/m^3$	0.5 to .58	4	13.0	1.9				71

Table 7-8. Summary of Removal Efficiency in the First Run (Saidam et al., 1995).

Parameter	Percent Removal of Individual Filters						% Removal Per Train		
	Rock Filter 1	Wadi Gravel Filter 1	Rock Filter 2	Coarse Aggregate Filter 2	Wadi Gravel Filter 3	Medium Aggregate Filter 3	1st Train	2^{nd} Train	3^{rd} Train
TSS	34	41	35	22	46	25	61	49	59
BOD_5	36	41	44	4	49	13	62	46	56
COD	19	18	21	15	24	25	33	33	44
Total P	9	15	9	30	18	33	24	35	46
Total FC	80	55	83	21	85	60	90	86	94
Color	25	34	28	20	30	36	51	42	55
HLR $m^3/m^3/d$	0.498	0.386	0.311	0.333	0.274	0.442	-	-	-

7.2.2.5 New Zealand Rock Filters

Rock filters have been used in New Zealand for removing high concentrations of algae from pond effluents (Middlebrooks et al,2005). The systems were developed from sub-surface flow wetlands without plants. The rock ranged from 12 – 24 cm in diameter, with the coarser rocks at the inlet and outlet to distribute the flow evenly. A cross-section of the rock filter at Paeroa, New Zealand is shown in Figure 7-5.

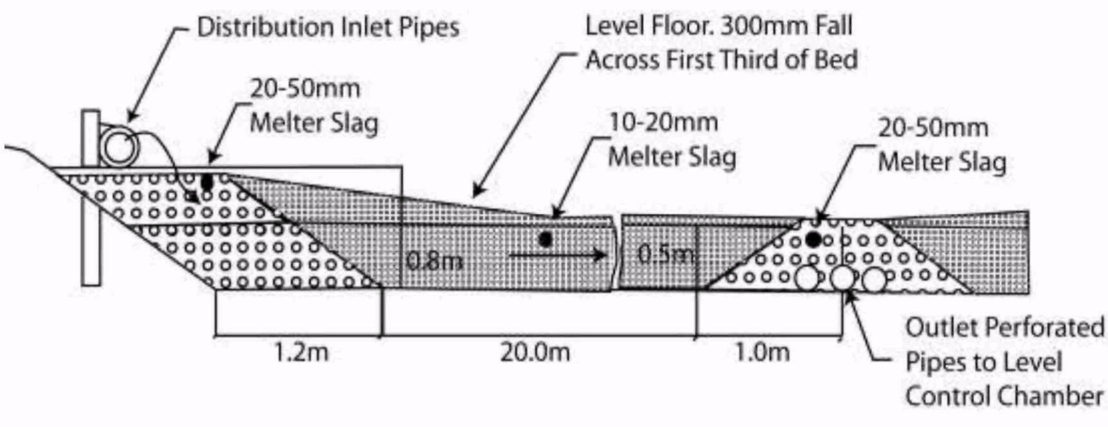


Figure 7-5. Cross-sectional view of Paeroa, New Zealand rock filter (Middlebrooks et al., 2005).

The rock filters are generally anoxic and there is little nitrification, however, there can be denitrification. The effluent is anaerobic and does emit H_2S on occasion. If the influent contains high concentrations of algae, organic N will increase in the effluent.

Three systems in New Zealand used steel slag, which has a high porosity and produces less H_2S. Some phosphorus removal was observed for the first years of operation. The filters followed partial mix aerated ponds, and have consistently produced TSS effluent concentrations less than 25 mg/L. Average removals have been less than 12 mg/L, even when influent solids were 100 mg/L or greater.

7.2.2.6 Design of Rock Filters

Rock filters have been designed using a number of parameters. A summary of the design parameters used for several locations is shown in Table 7-9. The parameters shown for the state of Illinois are the current standards and were not necessarily used to design the systems diagrammed in Figure 7-3. The critical factor in the design of rock filters appears to be the hydraulic loading rate. Rates less than 0.3 $m^3/m^3/d$ give the best results with rocks in the range of 8 - 20 cm and a depth of 2 m with the water applied in an up flow pattern. Design parameters and performance of some rock filters in New Zealand are shown in Table 7-10.

Table 7-9. Design Parameters for Rock Filter Systems in the United States (Oregon: Swanson and Williamson, 1980; Louisiana: Stamberg et al., 1984; Kansas and Missouri: U.S. EPA, 1983a).

Parameter	Veneta	W. Monroe	State of Illinois	Eudora	California
Hydraulic Loading Rate $m^3/m^2/d$	0.3	0.36	0.8	Up to 1.2 in the summer. 0.4 in winter & spring	0.4
Rock cm	7.5-20	5-13	8-15 Free of fines Soft weathering stone , and no flat rock	2.5	6-13
Aeration	None	None	Post-aeration ability necessary	None	None
Depth, m	2	1.8	Rock media must extend 0.3 m above water surface	1.5	1.68
Disinfection	Yes	Yes	Chlorination of post-aeration cell encouraged	Not Applicable	Yes

Table 7-10. Design Parameters and Performance of New Zealand Rock Filters (Middlebrooks, 2005).

		Waluku	Paeroa	Ngatea	Clarks Beach
Design flow (average)	m^3/day	3,000	2,067	460	375
Current flow (average)	m^3/day	1,800	2,100	250	290
Width	m	29.6	22	26.3	32
Length	m	97.4	131	136.0	62
No. of beds		10	8	2	2
Total rock filter area	m^2	28,868	23,056	7,154	3,875
Rock size	mm	20/10	20/10	20/10	20/10
Rock type		slag	slag	slag	greywacke
Rock depth	m	0.5	0.5-0.8	0.5-0.8	0.5-0.65
Rock filter loading rate (average)	mm/day	`62	91	35	75
Rock filter loading rate (average)	m^3/m^3 day	0.14	0.20	0.08	0.17
Average water depth	m	0.45	0.45	0.45	0.45
Hydraulic retention time (average)	days	3.3	2.2	5.8	1.5
Year constructed		1993	2000	2002	1998
Average water quality (mg/L)					
$CBOD_5$	average	6	5	3	
	95 percentile	11	19	6	
Suspended Solids	average	12	9	6	
	95 percentile	24	17	9	
NH_3	average	5	7	15	
	95 percentile	24	12	27	
Total N	average	8	10	19	
	95 percentile	20	17	36	

7.2.2.7 Aerated Rock Filter

To address the lack of NH_3 removal in rock filters, Mara and Johnson (2006) constructed an aerated rock filter with perforated pipe placed in the underdrain. They operated the aerated rock filter in parallel with a non-aerated control over an 18-month period. Facultative pond effluent containing approximately 10 mg/L of NH_3 was applied to the filters at a hydraulic rate of 150 L/m^2/d during the first eight months of operation and at 300 L/m^2/d thereafter. Ammonia concentrations in the aerated filter effluent were less than 3 mg/L, and NO_3^- concentrations were approximately 5 mg/L, while the control filter N concentrations were approximately 7 mg/L. Ammonia removal did not occur in the non-aerated control, and there was a statistically significant increase in the mean NH_3 concentration between the influent and effluent. Fecal coliform concentrations were reduced in the aerated filter from 103 to 104 per 100mL to a geometric mean count of 65 per 100 mL. BOD_5 and TSS removals were much higher in the aerated filter. The 95 percentile effluent concentrations in the aerated filter were 9 and 10 mg/L, respectively, while the effluent concentrations from the control were 38 and 43 mg/L.

Increasing the hydraulic loading rate from 150 to 300 L /m^2/d did not negatively affect the mean percentage BOD_5, NH_3 and fecal coliform removals. There was a slight reduction in the TSS removals. It was concluded that the use of aerated rock filters eliminates the need for maturation ponds to remove NH_3, and reduces the surface area required for maturation ponds at a flow rate of 200 L/person/d from approximately 5 m^2/person to 1.3 m^2/person with an aerated rock filter 0.5 m deep and loaded at 300 L/m^2/d. In winter, the facultative pond DO concentration was approximately 2 mg/L and approximately 8 mg/L in the aerated filter effluent. The control non-aerated filter effluent DO concentration was approximately 1 mg/L.

In a follow-up study Johnson and Mara (2007) conducted studies comparing a pilot-scale subsurface horizontal flow constructed wetland, a non-aerated rock filter and an aerated rock filter receiving effluent from a facultative pond loaded at 79 kg/ha/d. BOD_5, TSS and NH_3 concentrations were lower in the effluent from the aerated rock filter when compared with the non-aerated rock filter and the constructed wetland. A summary of the results are shown in Table 7-11.

Table 7-11. BOD_5, TSS and NH_3 Concentrations in the Effluents of the Facultative Pond, Aerated Rock Filter and Constructed Wetlands (Johnson and Mara, 2007).

Period	Parameter	Facultative Pond	Aerated Rock Filter	Constructed Wetland
Summer[a]	BOD_5 (mg/L)			
	Mean	39	4.5	20
	S.D[c]	9	1.5	7
	95%[d]	53	6	29
	TSS (mg/L)			
	Mean	58	4	26
	S.D.	27	2	19
	95%	99	7	52
	NH_3 (mg/L)			
	Mean	3.8	1.7	2
	S.D.	1.6	0.2	1.5
	95%	6	2	4.4
Winter[b]	BOD_5			
	Mean	41	4.2	21
	S.D.	14	2.7	8
	95%	58	8.1	32
	TSS			
	Mean	78	4.9	30
	S.D.	21	2.9	6
	95%	113	9	35
	Ammonia			
	Mean	10	4.7	9
	S.D.	1.4	2.4	1
	95%	12	8	10

[a] June-August 2004, [b] December 2004-February 2005. [c] Standard Deviation, [d] 95 percentile value

7.2.3 Normal Granular Media Filtration

Granular media filtration (rapid sand filters) separates liquids and solids. The simple design and operation process makes it applicable to wastewater streams containing up to 200 mg/L suspended solids. The process can be automated based on easily measured parameters with minimum operation and maintenance costs. On the other hand, regular granular media filtration is not as efficient for removing algae unless coagulants or flocculants have been added prior to filtration. Table 7-12 contains a summary of the results with direct granular media filtration.

Table 7-12. Summary of Direct Filtration with Rapid Sand Filters (d_{50} = diameter of 50 percent of sand).

Investigator	Coagulant	Filter Loading gpm/sf	Filter Depth cm	Sand Size mm	Findings
Borchardt and O'Melia (1961)	none	0.2-2	61	d_{50} = 0.32	Removal declines to 21-45% after 15 hr 50% algae removal
	Fe 7 mg/L	2.1	61	d_{50} = 0.40	
Davis and Borchardt (1966)	none	0.49	NA	d_{50} = 0.75	22% algae removal
	none	0.49		d_{50} = 0.29	34% algae removal
	none	1.9		d_{50} = 0.75	10% algae removal
	none	1.9		d_{50} = 0.29	2% algae removal
	Fe	NA	NA	d_{50} = 0.75	45% algae removal
Foss and Borchardt (1969)	none	2	91	d_{50} = 0.71	*p*H 2.5, 90% removal
Lynam et al. (1969)	none	1.1	28	d_{50} = 0.55	62% TSS removal
Kormanik and Cravens (1978)	none	-	-	-	11-45% TSS removal

Diatomateous earth filtration is capable of producing a high-quality effluent when treating wastewater treatment pond water, but the filter cycles are generally less than 3 hours. This results in excessive usage of backwash water and diatomateous earth, which increases costs and eliminates this method of filtration as an alternative for polishing wastewater treatment pond effluents.

7.2.4 Coagulation-Flocculation

Coagulation followed by sedimentation has been applied extensively for the removal of suspended and colloidal materials from water. Lime, alum and ferric salts are the most commonly used coagulating agents. Floc formation is sensitive to parameters such as *p*H, alkalinity, turbidity and temperature. Most of these variables have been studied, and their effects on the removal of water supply turbidity have been evaluated. In the case of the chemical treatment of wastewater treatment pond effluents, however, the data are not comprehensive.

Shindala and Stewart (1971) investigated chemical treatment of treatment pond effluents as a post-treatment process to remove the algae and to improve the quality of the effluent. They found that the optimum dosage for best removal of the parameters studied was 75-100 mg/L of alum. When this dosage was used, the removal of phosphate was 90 percent and the BOD_5 was 70 percent.

Tenney (1968) has shown that at a *p*H range of 2 to 4, algal flocculation was effective when a constant concentration of a cationic polyelectrolyte (10 mg/L of C-31) was used. Golueke and Oswald (1965) conducted a series of experiments to investigate the relation of hydrogen ion concentrations to algal flocculation. In this study, only H_2SO_4 was used, and only to lower the *p*H. Golueke and Oswald found that flocculation was most extensive at a *p*H value of 3, which agrees with Tenney's results and reported algal removals of about 80-90 percent. Algal removal efficiencies by cationic polyelectrolytes were not affected in the *p*H range of 6-10.

The California Department of Water Resources (1971) reported that of 60 polyelectrolytes tested, 17 compounds were effective with regard to coagulation of algae and were economically competitive when compared to mineral coagulation used alone. Generally, a dose of less than 10 mg/L of the polyelectrolytes was required for effective coagulation. A daily addition of 1 mg/L of $FeCl_3$ to the algal growth pond resulted in significant reductions in the required dosage of both organic and inorganic coagulants.

McGarry (1970) studied the coagulation of algae in treatment pond effluents and reported the results of a complete factorial designed experiment using the common jar test. Tests were performed to determine the economic feasibility of using polyelectrolytes as primary coagulants alone or in combination with alum. McGarry also investigated some of the independent variables that affected the flocculation process, such as concentration of alum, flocculation turbulence, concentration of polyelectrolytes, *p*H after the addition of coagulants, chemical dispersal conditions, and high rate oxidation pond suspension characteristics. Alum was found to be effective for coagulation of algae from high rate oxidation pond effluent. The lowest cost per unit algal removal was obtained with alum alone (75-100 mg/L).

Al-Layla and Middlebrooks (1975) evaluated the effects of temperature on algae removal using coagulation-flocculation-sedimentation. Removal at a given alum dosage decreased as the temperature increased. Maximum algae removal generally occurred at an alum dosage of approximately 300 mg/L at 10 °C. At higher temperatures, alum dosages as high as 600 mg/L did not produce removals equivalent to the results obtained at 10 °C with 300 mg/L of alum. The settling time required to achieve significant removals, flocculation time, organic carbon removal, total *P* removal, and turbidity removal were found to vary inversely as the temperature of the wastewater increased.

Dryden and Stern (1968) and Parker (1976) reported on the performance and operating costs of a coagulation-flocculation system followed by sedimentation, filtration, and chlorination, with discharge to recreational lakes. This system, in Lancaster, California, probably has the longest operating record of any coagulation-flocculation system treating wastewater treatment pond effluent. The TSS concentrations of influent coming to the plant have ranged from about 120 to 175 mg/L, and the plant has produced an effluent with a turbidity of less than 1 Jackson turbidity unit (JTU) most of the time. Aluminum sulfate [$Al_2(SO_4)_3$] dosages have ranged from 200 to 360 mg/L. The design capacity is 1893 m^3/d (0.5 mgd).

Coagulation-flocculation is not easily controlled and requires expert operating personnel at all times. A large volume of sludge may be produced, which can introduce an additional operating cost.

7.2.5 Dissolved Air Flotation

Several studies have shown the dissolved air flotation process to be an efficient and a cost-effective means of algae removal from wastewater treatment pond effluents. The performance obtained in several of these studies is summarized in Table 7-13.

Table 7-13. Summary of Typical Dissolved Air Flotation Performance.

Location and Reference	Coagulant and Dose (mg/L)	Overflow Rate (gpm/sf)	Detention Time (minutes)	BOD_5		
				Influent (mg/L)	Effluent (mg/L)	% Removed
Stockton [1] Parker (1976)	Alum, 225 Acid added to pH 6.4	2.7[a]	17[a]	46	5	89
Lubbock[2] Ort (1972)	Lime[c], 150	NA	12[b]	280-450	1.3	>99
Eldorado[3] Komline-Sanderson Engineering (1972)	Alum, 200	4.0[c]	8[c]	93	<3	<97
Logan[4] Bare (1971)	Alum, 300	1.3-2.4[d]	NA	NA	NA	NA
Sunnyvale[1] Stone et al., (1975)	Alum, 175 Acid added to *p*H 6.0 to 6.3	2.0[e]	11[e]	NA	NA	NA
Stockton[1] Parker (1976)	Alum, 225 Acid added to *p*H 6.4	2.7[a]	17[a]	104	20	81
Lubbock[2] Ort (1972)	Lime[c], 150	NA	12[b]	240-360	0-50	>79
Eldorado[3] Komline-Sanderson Engineering (1972)	Alum, 200	4.0[c]	8[c]	450	36	92
Logan[4] Bare (1971)	Alum, 300	1.3-2.4[d]	NA	100	4	96
Sunnyvale[1] Stone et al., (1975)	Alum, 175 Acid added to *p*H 6.0 to 6.3	2.0[e]	11[e]	150	30	80

[1]California, [2]Texas, [3]Arizona, [4]Utah
[a] 33% pressurized (35-60 psi) recycle
[b]30% pressurized (50 psi) recycle
[c] 100% pressurized recycle
[d] 25% pressurized (45 psi) recycle
[e] 27% pressurized (55-70 psi) recycle

Three basic types of dissolved air flotation are employed to treat wastewaters: total, partial and recycle pressurization. These three types are illustrated by flow diagrams in

Figure 7-6. In the total pressurization system, the entire wastewater stream is injected with air, pressurized and held in a retention tank before entering the flotation cell. The flow is direct, and all recycled effluent is repressurized. In partial pressurization, only part of the wastewater stream is pressurized, and the remainder of the flow bypasses the air dissolution system and enters the separator directly. Recycling serves to protect the pump during periods of low flow, but it does load the separator hydraulically. Partial pressurization requires a smaller pump and a smaller pressurization system. In recycle pressurization, clarified effluent is recycled for the purpose of adding air and then is injected into the raw wastewater. Approximately 20-50 percent of the effluent is pressurized in this system. The recycle flow is blended with the raw water flow in the flotation cell or in an inlet manifold.

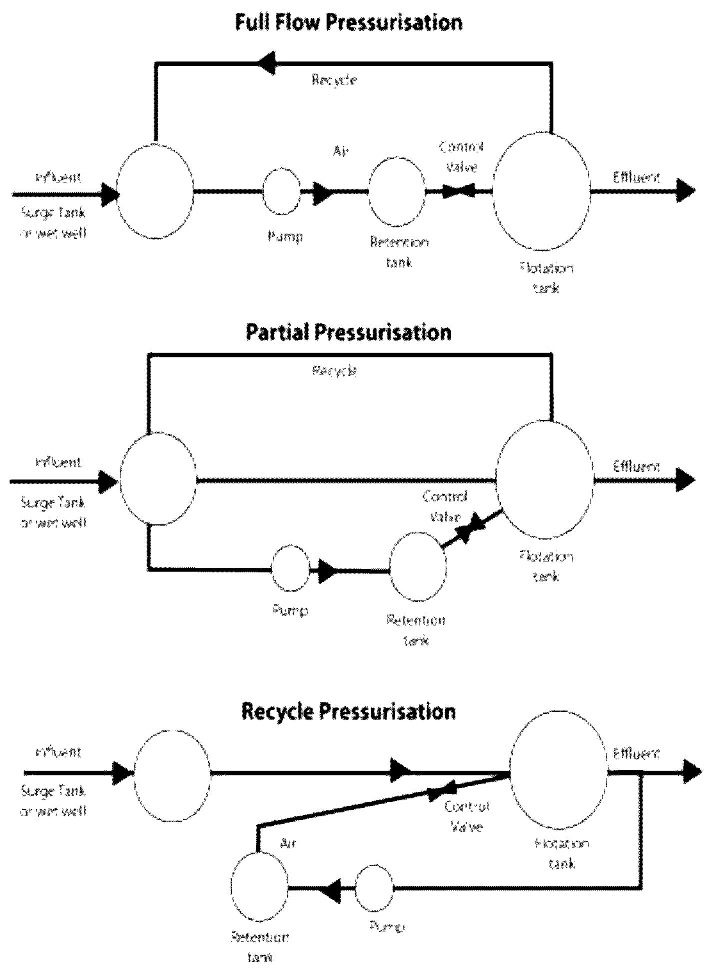


Figure 7-6. Types of dissolved air flotation systems (Snider, 1976).

Important parameters in the design of a flotation system are hydraulic loading rate (including recycle), concentration of TSS contained within the flow, coagulant dosage, and the air-to-solids ratio required to achieve efficient removal. Pilot-plant studies by Stone et al. (1975), Bare (1971) and Snider (1976) have shown the maximum hydraulic loading rate to range between 81.5 - 101.8 L/min/m^2. The most efficient air-to-solids ratio was found to be 0.019 - 1.0 (Bare 1971). Solids concentrations during Bare's studies were 125 mg/L. Experimental results with the removal of algae indicate that lower hydraulic rates and air-to-solids ratios than those recommended by the manufacturers of industrial equipment should be employed when attempting to remove algae.

In combined sedimentation flotation pilot-plant studies at Windhoek, Namibia, van Vuuren and van Duuren (1965) reported effective hydraulic loading rates to range between 11.2 and 30.5 L/min/m^2, with flotation provided by the naturally dissolved gases. Because air was not added, air-solids ratios were not reported. They also noted that it was necessary to use from 125 - 175 mg/L of $Al_2(SO_4)_3$ to flocculate the effluent containing from 25 - 40 mg/L of algae. Subsequent reports on a total flotation system by van Vuuren et al. (1965) stated that a dose of 400 mg/L of $Al_2(SO_4)_3$ was required to flocculate a 110 mg/L algal suspension sufficiently to obtain a removal that was satisfactory for consumptive reuse of the water. Based on data provided by Parker et al. (1973), Stone et al. (1975), Bare (1971), and Snider (1976), it appears that a much lower dose of alum can be applied to produce an effluent that will meet present discharge standards.

Dissolved air flotation with the application of coagulants performs essentially the same function as coagulation-flocculation-sedimentation, except that a much smaller system is required with the flotation device. Flotation will occur in shallow tanks with hydraulic residence times of 7-20 min, compared with hours in deep sedimentation tanks. Overflow rates can be as high as 81.5-101.8 L/min-m^2 with flotation; whereas, a value of less than 40.7 L/min-m^2 is recommended with sedimentation. However, it must be pointed out that the sedimentation process is much simpler to operate and maintain than the flotation process, and when applied to small systems, consideration must be given to this factor.

The flotation process does not require a separate flocculation unit, and this has definite advantages. It has been shown that it is best to add alum at the point of pressure release where mixing occurs so that the chemicals are well dispersed. Brown and Caldwell (1976) designed two tertiary treatment plants that employ flotation, and have developed design considerations that should be applied when employing flotation. These features are not included in standard flotation units and should be incorporated to ensure good algae removal (Parker, 1976).

In addition to incorporating various mechanical improvements, Brown and Caldwell recommended that the tank surface be protected from excessive wind currents to prevent float movement to one side of the tank. It was also recommended that the flotation tank be covered in rainy climates to prevent the breakdown of the floc. Another proposed alternative is to store the wastewater in treatment ponds during the rainy season and then operate the flotation process at a higher rate during dry weather.

Dissolved air flotation thickening (DAFT) has been used at the Stockton, California regional wastewater treatment facility for many years to remove algae from the treatment ponds ahead of the tertiary filtration process. Performance results for the period June -

October 2005 are shown in Figures 7-7 and 7-8. Average pond influent TSS concentrations averaged 74 mg/L (range: 20 - 223). Effluent concentrations averaged 34 mg/L, (range: 15 – 105). The percentage removal averaged 50 percent. In 2009-2010, the DAFT process tanks and internal equipment underwent major rehabilitation. Additional skimmer arms were added to improve removal of floating algae, and the initial results indicate improved performance (Figure 7-9).

DAFT influent is secondary effluent that has received further treatment in facultative ponds, then flows through a constructed wetlands that was put in service in 2007. Alum is fed to the DAFT influent for chemical conditioning of the algae solids. Performance results available for 2010 show the influent TSS concentrations average 70 mg/L and effluent TSS concentrations average 17 mg/L, for an average removal efficiency of 76 percent (Larry Parlin, pers. comm. 2010).

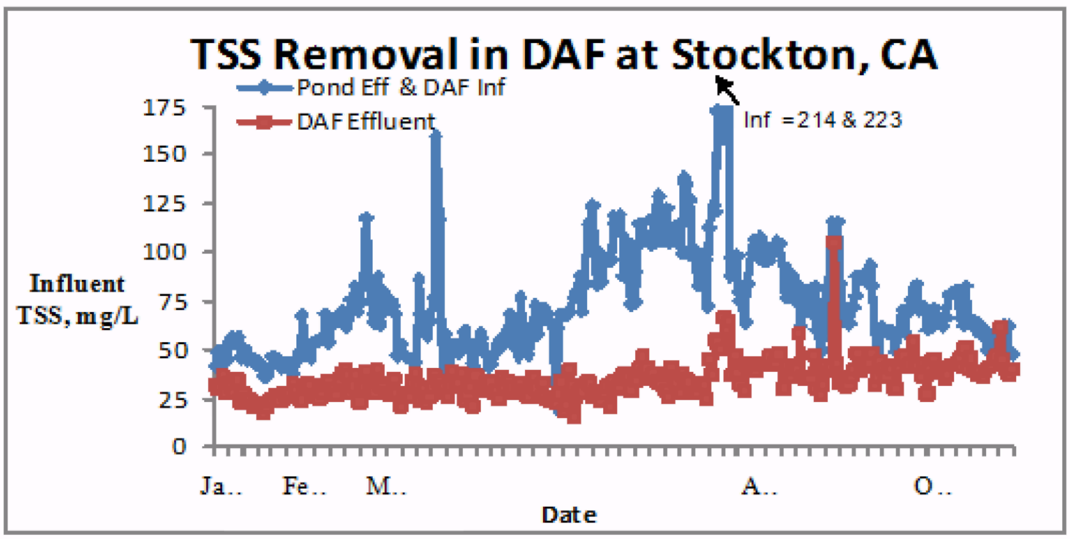


Figure 7-7. TSS removal from pond effluent in dissolved air flotation with alum addition (Middlebrooks, 2005).

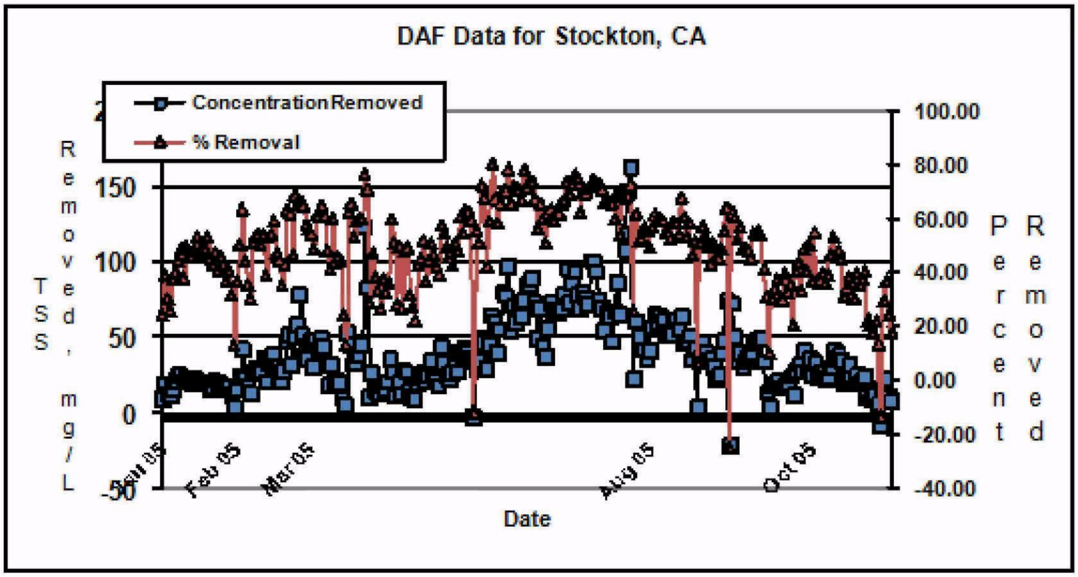


Figure 7-8. Concentration and percent TSS removal from pond effluent in dissolved air flotation with alum addition (Middlebrooks, 2005).

Figure 7-9 Dissolved air floatation thickening (DAFT) at the Stockton, California wastewater treatment facility (Parlin, pers. comm. 2010).

Alum-algae sludge was returned to the wastewater treatment ponds for over three years at Sunnyvale, California with no apparent detrimental effect (Farnham, pers. comm., 1981). No sludge banks, floating mats of material, or increased TSS concentrations in the pond effluent have been observed. Returning the float to the pond system is an operational option, at least for a few years. Most estimates of a period of time that sludge can be returned range from 10 to 20 years.

Sludge disposal from a dissolved air flotation system can present considerable challenges. Alum-algae sludge is very difficult to dewater and discard. Centrifugation and vacuum filtration of raw alum-algae sludge have produced marginal results. Indications are that lime coagulation may prove to be as effective as alum to produce sludge that is more easily dewatered.

Brown and Caldwell (1976) evaluated heat treatment of alum-algae sludges using the Porteous, Zimpro® low-oxidation, and Zimpro® high-oxidation processes without great effect. The Purifa process, using chlorine to stabilize the sludge, produced a sludge that was dewaterable on sand beds or in a pond. If algae are killed before entering an anaerobic digester, the proportion of volatile matter destruction and dewatering can provide more useful results. But, as with the other sludge treatment and disposal processes, additional operations and costs are incurred, which may make the option of dissolved air flotation less competitive financially.

7.3 OPERATIONS MODIFICATIONS AND ADDITIONS

7.3.1 Autoflocculation and Phase Isolation

Autoflocculation of algae (natural settling under specific environmental conditions) has been observed in some studies (Golueke and Oswald, 1965; McGriff and McKinney, 1971; McKinney, 1971; Hill et al., 1977). Chlorella was the predominant alga occurring in most of the cultures. Laboratory-scale continuous experiments with mixtures of activated sludge and algae have produced large bacteria-algae flocs with good settling characteristics (Hill et al., 1977; Hill and Shindala, 1977). Floating algal blankets have been reported in the presence of chemical coagulants in some cases (Shindala and Stewart, 1971; van Vuuren and van Duuren, 1965). This may be caused by the entrapment of gas bubbles produced during metabolism or by the fact that, at a particular stage in the growth cycle, algae have neutral buoyancy. In an 11,355 L/hr (3000 g/h) pilotplant that combined flocculation and sedimentation, a floating algal blanket was formed with alum doses of 125 -170 mg/L. About 50 percent of the algae was able to be skimmed from the surface (van Vuuren and van Duuren, 1965). Given the unpredictable occurrence of conditions necessary for autoflocculation, it can not be considered a reliable method for removing algae from wastewater treatment ponds.

Phase isolation is defined as the operation of a pond system to create natural conditions favorable to settling of algae and some success has been reported based on this phenomenon to remove algae from pond effluents. The results of a study by McGriff (1981) of a full-scale operation of a phase isolation system were not consistent.

Oswald and Green, (2000), enhanced algal growth is in a high rate pond with a raceway configuration and a slow-moving paddle wheel to keep algae suspended. This concentrated algal slurry is sent to a settling basin, where the algae can be concentrated further and sent to a drying bed. There is potential to use the algal slurry for feed supplement, soil fertilization and amendment and, most recently, for biofuel production (Woertz et al., 2009, Brune et al., 2009).

7.3.2 Baffles and Attached Growth

The enhancement of attached microbial growth in oxidation ponds is an apparently practical solution for maintaining biological populations while still obtaining the treatment desired. Although baffles are considered useful primarily to ensure good mixing and to eliminate the problem of short-circuiting, they provide a substrate for bacteria, algae, and other microorganisms to grow (Reynolds et al., 1975; Polprasert and Agarwalla, 1995). In general, attached growth surpasses suspended growth if sufficient surface area is available. In anaerobic or facultative ponds with baffling or biological disks, the microbiological community consists of a gradient of algae to photosynthetic, chromogenic bacteria and, finally, to nonphotosynthetic, nonchromogenic bacteria (Reynolds et al., 1975). In these experiments, the microbial growth associated with the baffled system was identified as the mechanism that produced a more effective treatment. Simple fixed baffles constructed of wood or plastic, floating plastic baffles used to improve hydraulic characteristics, or, indeed, any surface can provide a substrate on which microbial growth can take place.

Polprasert and Agarwalla (1995) demonstrated the significance of biofilm biomass growing on the side walls and bottoms of ponds and presented a model for substrate utilization in facultative ponds using first-order reactions for both suspended and biofilm biomass.

7.3.3 Land Application

The design and operation of land treatment systems is described in detail in Reed et al., (1995), Crites et al., (2000) and U.S. EPA (2006). These publications should be consulted before designing a land application system to polish a pond effluent. Ecological conditions will dictate whether this is as an option that should be considered.

7.3.4 Macrophyte and Animal Systems

Various macrophytic floating plans have been used to reduce algal concentrations and TSS in maturation ponds. Rittman and McCarty (2001). Detailed design information can be obtained in Reed et al., (1995), Pearson and Green (1995), Mara et al. (1996), Pearson et al. (2000) and Shilton (2005).

7.3.4.1 Floating Plants

Water hyacinths (*Eichhornia crassipes*), duckweed (*Lemna* spp), pennywort (*Centella asiatica*), and water ferns (*Azolla* spp.) appear to offer the greatest potential for wastewater treatment. Each has its own environmental requirements, and hyacinths, pennywort, and duckweeds are the only floating plants that have been evaluated in pilot - or full-scale systems. Detailed design considerations are presented in Reed et al. (1995). Information about the use of these plants to improve wastewater quality for reuse can be found in Rose (1999).

7.3.4.2 Submerged Plants

Submerged aquatic macrophytes for treatment of wastewaters have been studied extensively in the laboratory, greenhouses, a pilot study by McNabb (1976), and in large scale wetland storm water treatment systems designed to remove *P* to less than 20 mg/L (South Florida Water Management District, 2003).

7.3.4.3 *Daphnia* and Brine Shrimp

Daphnia spp. are filter feeders and their main contribution to wastewater treatment is the removal of suspended solids, particularly algae (U.S. EPA, 2002). *Daphnia* is sensitive to the concentration of NH_3 in wastewater, which is toxic to invertebrates. To be effective, shading is required to prevent the growth of algae that will result in high *p*H values during the daytime. The addition of acid and gentle aeration may be necessary.

7.3.4.4 Fish

Fish have been grown in treated wastewaters for centuries, and, where toxics are not encountered, the process has been successful. Many species of fish have been used in wastewater treatment, but fish activity is temperature dependent. Most grow successfully in warm water. Catfish and minnows are exceptions. Dissolved oxygen concentrations are critical and the presence of NH_3 is toxic to the young of the species. Detailed studies of fish in wastewater treatment ponds have been conducted by Coleman (1974) and Henderson (1979). Numerous studies of fish culture have been conducted around the world. Polprasert and Koottatep (2005) presented an excellent summary of the use of algae eating fish in pond systems.

7.4 CONTROL OF ALGAE AND DESIGN OF SETTLING BASINS

Control of algae in wastewater treatment pond effluents has been a major concern throughout the history of the use of these systems. Algae grow in maturation and polishing ponds following all types of treatment processes, which increases the TSS in the effluent. State design standards requiring long detention times in the final cell in a pond system have inadvertently exacerbated the problem. In recognition of the difference between the source of the TSS in the influent and the effluent, the state of Minnesota has mandated a higher TSS limit of 45 mg/L for ponds. (Steve Duerre, pers. comm.)

It has been established that few, if any, of the solids in pond effluents are fecal matter or material entering the pond system. This has led to much discussion about the necessity to remove algae from pond effluents. Although the concern that the TSS might harbor human pathogens may not be realistic, when the algae die, settle out and decay, they do create some O_2 demand on the receiving stream. The concern about decay and O_2 consumption has led to investigations of the most effective methods to remove algae and how to design systems to minimize growth in the settling basins. Toms et al. (1975) studied algal growth rates in polishing ponds receiving activated sludge effluents for 18 months. They concluded that growth rates for the dominant species were less than 0.48 /d, and if the HRT was less than two days, algal growth would not be a problem. At HRT less than 2.5 days, the effluent TSS decreased. Uhlmann (1971) reported no algal growth in hyper-fertilized ponds when the detention times were less than 2.5 days. Toms et al. (1975) evaluated one- and four-cell polishing ponds and found that for HRT beyond 2.5 days the TSS increased in both ponds, but significant growth did not occur until after 4 - 5 days in the four-cell pond.

Algae require light to grow, and as light penetration is reduced with increasing depth, it might be hypothesized that increasing the depth of a maturation or polishing pond would help to reduce algal growth. As most pond cells are trapezoidal, there is little to be gained by increasing the depth beyond three to four meters. Without mechanical mixing, thermal stratification occurs in ponds, providing an excellent environment for algae to grow. Disturbing stratification will reduce algal growth. Rich (1999) recommends some degree of aeration for pond cells to control algae. The higher aeration rate will suspend more solids. The resulting reduction in light transmission helps to reduce the rate of algal growth.

7.4.1 Control of Algal Growth by Shading, Barley Straw and Ultra Sound

7.4.1.1 Dyes have been applied to small ponds to control algal growth. However, EPA has not approved dyes for use in municipal or industrial wastewater ponds. Aquashade®, a mixture of blue and yellow dyes, is marketed as a means of controlling algae in backyard garden pools and large business park and residential development ponds. The product is registered with EPA for these uses.

7.4.1.2 Fabric Structures

Operators of ponds in Colorado and other locations have constructed structures suspending opaque greenhouse fabrics to reduce or eliminate light transmittance in small wastewater ponds. A partially covered pond using a fabric located in Naturita, Colorado is shown in Figure 7-10.

Figure 7-10. Photograph of shading for control of algal growth in Naturita, Colorado (R. Bowman, pers. comm., 2000).

The screening effect has been successful, but in some cases fabrics were not fastened adequately and they were damaged by the wind. Covering the final pond with adequate protection from the wind should reduce or eliminate algal growth. With full coverage of the surface, anaerobic conditions may develop and aeration of the effluent may be necessary to meet discharge standards. Partial shading in correct proportions should reduce the possibility of creating anaerobic conditions.

7.4.1.3 Barley Straw

In 1980 it was observed that the addition of barley straw to a lake reduced the algal concentration. Placing barley straw in ponds has been proposed as a means of controlling algal growth. Details for the application of barley straw is given in IACR-Centre for Aquatic Plant Management (1999) and the state of Illinois guidance for application and discussion of how to classify barley straw in this application is found in Appendix H. Figure 7-11 shows a barley straw application in the final cell in an aerated pond system in New Baden, Illinois (Zhou et al., 2005).

During decomposition, the chemicals listed in Table 7-11 are released to the water and inhibit the growth of algae (Everall and Lees, 1997). The acceptability of this method of algal control by regulatory agencies has not been resolved.

Figure 7-11. A barley straw boom in cell 3, New Baden, Illinois wastewater pond system.

Table 7-14. List of Chemicals Produced by Decomposing Straw (Everall and Lees, 1997).

Acetic Acid

3-Methylbutanoic Acid

2-Methylbutanonic Acid

Hexanoic Acid

Octanoic Acid
Nonanoic Acid
Decanoic Acid
Dodecanoic Acid
Tetradecanoic Acid
Hexadecanoic Acid
1-Methylnaphthalene
2-(1,1-Dimethlyethyl Phenol)
2,6-Dimethoxy-4-(2-propenyl) Phenol
2,3-Dihydrobenzofuron
5,6,7,7A-Tetrahydro-4,4,7A-trimethyl-2(4H) benzofuranone
1,1,4,4-Tetramethyl-2,6-bis(methylene) cyclohexone
1-Hexacosene
11 Unidentified

7.4.1.4 Ultra Sound

Ultra sound devices have been used for algal control in golf course ponds, large residential area ponds, and water treatment storage ponds, but limited data are available for municipal pond systems. A microcosm study at the Centre for Aquatic Plant Management (CAPM) in Reading, Berkshire, United Kingdom evaluated the efficacy of several treatment options to control algae (Clarke, 2004). Methods included an ultrasonic device, a recirculating pump, bacteria, barley straw, Aquavantage (electromagnet treatment), EcoFlow (fixed magnet) and a control. The results of the experiments are summarized in Figure 7-12.

According to Clarke (2004), none of the treatments appeared to remove the algae to a level that would meet water quality requirements. Differences in the level of algae could be seen, but some of the four replicate tanks in all treatments remained turbid and green. The only tanks that were clear were found to be populated by *Daphnia spp*., an invertebrate herbivore. Clarke reported that no significant differences could be found between treatments. The variability and experimental challenges made it difficult to draw conclusions as to the possible causes of either growth or inhibition of growth.

The CAPM investigated the mode of action of ultrasound on algae. Clarke reported *Spirogyra* and *Selenastrum* were damaged irreversibly by the treatment.

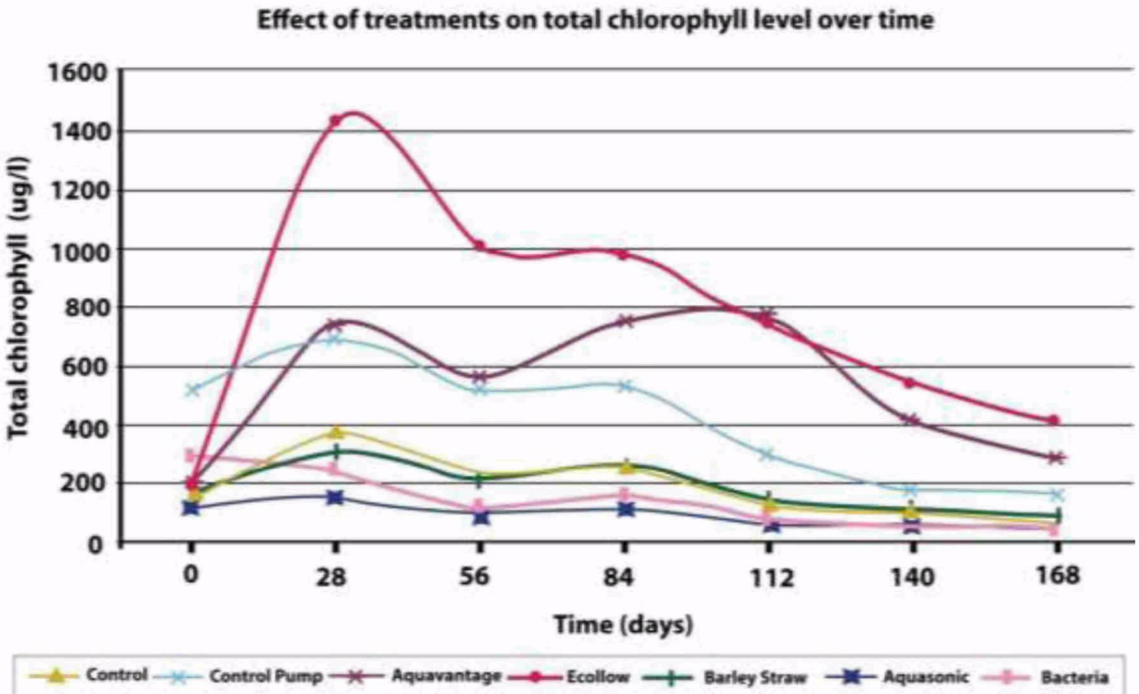


Figure 7-12. Change in chlorophyll over time under different treatment conditions (Clarke, 2004).

7.5 COMPARISON OF VARIOUS DESIGN PROCEDURES

The variety of configurations and objectives of the design approaches for nutrient removal make it difficult to make direct comparisons to determine which will be the most effective for a given site. Reasonable reaction rates must be selected, but if the pond hydraulic system is designed and constructed so that the theoretical HRT is approached, reasonable success can be assured with all of the design methods. Short-circuiting is the greatest deterrent to successful pond performance, barring any toxic effects. The importance of the hydraulic design of a pond system to achieve water quality objectives cannot be overemphasized.

7.6 OPERATIONAL MODIFICATIONS TO FACULTATIVE PONDS

7.6.1 Controlled Discharge Ponds

No rational or empirical design model exists specifically for the design of controlled discharge wastewater ponds. The unique features of controlled discharge ponds are long-term retention and periodic, controlled discharge usually once or twice a year. Rational and empirical design models applied to facultative pond design may also be applied to the design of controlled discharge ponds, provided allowance is made for the required larger storage volumes. Application of the ideal plug flow model developed for facultative ponds can be applied to controlled discharge ponds if HRTs of less than 120 days are considered. A study of 49 controlled discharge ponds in Michigan indicated that discharge periods vary from less than 5 days to more than 31 days, and residence times were 120 days or greater (Pierce, 1974). Ponds of this type have operated satisfactorily in the north-central United States using the following design criteria:

- Overall organic loading: 22-28 kg BOD_5/ha/d (20-25 lb BOD_5/ac/d)
- Liquid depth: Not more than 2 m (6 ft) for the first cell, not more than 2.5 m (8 ft) for subsequent cells
- Hydraulic detention: At least 6 months of storage above the 0.6 m (2 ft) liquid level (including precipitation), but not less than the period of ice cover
- Number of cells: At least 3 for reliability, with piping flexibility for parallel or series operation

The design of the controlled discharge pond must include an analysis showing that receiving stream water quality standards will be maintained during discharge intervals, and that the receiving watercourses can accommodate the discharge rate from the pond. The design must also include a recommended discharge schedule.

Selecting the optimum day and hour for release of the pond contents is critical to the success of this method. The operation and maintenance manual must include instructions on how to correlate pond discharge with effluent and stream quality. The pond contents and stream must be carefully monitored before and during the release of the pond contents.

In a typical program, discharge of effluents follows a consistent pattern for all ponds. The following steps are usually taken:

- Isolate the cell to be discharged, usually the final one in the series, by shutting off the valve on the inlet line from the preceding cell.

- Arrange to analyze samples for BOD_5, TSS, VSS, *p*H, and other parameters which may be required for a particular location.

- Plan work so as to be able to spend full time on control of the discharge throughout the period.

- Sample contents of the cell to be discharged for DO, noting turbidity, color, and any unusual conditions.

- Monitor conditions in the stream to receive the effluent.

- Notify the state regulatory agency of results of these observations and plans for discharge and obtain approval.

- If discharge is approved, commence discharge, and continue so long as weather is favorable, DO is near or above saturation values, and turbidity is not excessive following the prearranged discharge flow pattern among the cells.

    - Draw down the last 2 cells in the series (if there are 3 or more) to about 46 - 60 cm (18 - 24 in) after isolation, interrupting the discharge for a week or more to divert raw waste to a cell that has been drawn down, and resting the initial cell before its discharge.

- When the first cell is drawn down to about 60 cm (24 in) depth, the usual series flow pattern, without discharge, is resumed.
- During discharge to the receiving waters, samples should be taken at least 3 times each day near the discharge pipe for immediate DO analysis. Additional testing may be required for TSS.

Experience with these ponds is limited to northern states with seasonal and climatic influences on algal growth. See Appendix G for step-by-step instructions for controlled discharge operation (Minnesota Pollution Control Authority). The process will be quite effective for BOD_5 removal in any location and will also work with a more frequent discharge cycle than semi-annually, depending on receiving water conditions and requirements. Operating the isolation cell on a fill-and-draw batch basis is similar to the "phase isolation" technique.

7.6.2 Complete Retention Ponds

In areas of the United States where the moisture deficit (evaporation minus rainfall) exceeds 75 cm (30 in) annually, a complete retention wastewater pond may prove to be the most economical method of disposal. Complete retention ponds must be sized to provide the necessary surface area to evaporate the total annual wastewater volume plus the precipitation that would fall on the pond. The system should be designed for the maximum wet year and minimum evaporation year of record if overflow is not permissible under any circumstances. Less-stringent design standards may be appropriate in situations where occasional overflow is acceptable or an alternative disposal area is available under emergency conditions.

Monthly evaporation and precipitation rates must be known to properly size the system. Complete retention ponds usually require large land areas, and these areas may not be productive once they have been committed to this type of system. Land for this system must be naturally flat or be shaped to provide ponds that are uniform in depth, and have large surface areas. The design procedure for a complete retention wastewater pond system is presented in the following example.

7.6.2.1 Design Conditions

See Appendix C, Example C-7-3.

7.6.3 Hydrograph Controlled Release

The hydrograph controlled release (HCR) pond is a variation of the controlled discharge pond. This management practice was first put into practice in the southern United States, but can be used successfully in most areas of the world. In this case the discharge periods are controlled by a gauging station in the receiving stream and are allowed to occur during high flow periods. During low flow periods, the effluent is stored in the HCR pond.

The process design uses conventional facultative or aerated ponds for the basic treatment, followed by the HCR cell for storage and/or discharge. No treatment allowances are made during design for the residence time in the HCR cell; its sole function is storage. Depending on stream flow conditions, storage needs may range from 30 - 120 days. The design maximum water level in the HCR cell is typically about 2.4 m (8 ft), with the minimum water level at 0.6 m (2 ft). Other physical elements are similar to conventional pond systems. The major advantage of the HCR system is the possibility of utilizing lower discharge standards during high flow conditions as compared to a system designed for very stringent low flow requirements operated on a continuous basis. A summary of the design approach is shown in Appendix B.

Table 7-15. Hydrograph Controlled Release Pond Design Basics Used in United States.

a. Basic Principle: At critical low river flow, BOD_5 and TSS loadings are reduced by restricting effluent discharge rates rather than decreasing concentration of pollutants. Zirschsky and Thomas (1987).
b. Pond system must be sized to retain wastewater during low flow (Q10/7). Use existing ponds or build storage ponds. Q10/7 = once-in-10-year low flow rate for 7-day period. Zirschsky and Thomas (1987).
c. Assimilative capacity of receiving stream must be established by studying historical data or estimated using techniques such as that proposed by Hill and Zitta (1982).

Zirschsky and Thomas (1987) performed a nationwide assessment of HCR systems, which showed that they are effective, economical and simple to operate. HCR systems were also found to be an effective means of upgrading a pond effluent.

7.7 COMBINED SYSTEMS

In certain situations it is desirable to design pond systems in combinations, i.e., an anaerobic or an aerated pond (Li et al., 2006) followed by a facultative or a polishing pond. These combinations use the same design as the individual ponds. For example, the aerated pond would be designed as described in Chapter 3, Section 3.4, and the predicted effluent quality from this unit would be the influent quality for the facultative pond, which would be designed as described in Chapter 3, Section 3.3. Many of the proprietary systems described in Chapter 4 are combinations of various types of ponds.

7.8 PERFORMANCE COMPARISONS WITH OTHER REMOVAL METHODS

Designers and owners of small systems are strongly encouraged to use as simple a technology as feasible. Experience has shown that small communities or larger municipalities without properly trained operating personnel and access to spare parts, inevitably encounter serious maintenance problems using sophisticated technology and frequently fail to meet effluent standards. Methods discussed in this chapter that require good maintenance and operator skills are dissolved air flotation, centrifugation, coagulation-flocculation, and granular media filtration (rapid sand or mixed-media filters with chemical addition). At locations where operation and maintenance are available, these processes can be made to work well.

In summary, there are many methods of removing or controlling algae concentrations in pond effluents. Selection of the proper method for a particular site is dependent on many variables. Small communities with limited resources and untrained operating personnel should select as simple a system as is suitable to the site situation.

In rural areas with adequate land, ponds such as controlled discharge ponds or hydrograph controlled release ponds are an appropriate choice. In arid areas, the total containment pond should be considered. Performance by these types of treatment is controlled by selecting the time of discharge and can be managed to produce an effluent (BOD_5 and TSS < 30 mg/L) that meets compliance standards.

Where land is limited and resources and personnel are not available, it is best to utilize relatively simple methods to control algae in effluents. Intermittent sand filters, application of effluent to farmlands, overland flow, rapid infiltration, constructed wetlands, and rock filters are reasonable choices. Intermittent sand filters with low application rates and a warm climate will provide nitrification. Application to farm land will reduce both N and P, while producing a satisfactory effluent.

CHAPTER 8

COST AND ENERGY REQUIREMENTS

8.1 INTRODUCTION

Costs associated with wastewater treatment facilities fall under one of two categories: capital costs and operations and maintenance (O&M) costs. The price of energy makes up a significant portion of O&M costs for most wastewater treatment facilities. Although O&M cost data for many of the pond types and polishing methods are relatively limited, it is understood that these costs are generally lower than for conventional systems. Data presented in the following sections vary widely, but are thought to be reasonable estimates to serve as guides in budgeting for the costs associated with a treatment system. It should be kept in mind, however, that the data have different constraints that may be applicable to a specific design. Conventional estimating procedures should be used during final design.

8.2 CAPITAL COSTS

Construction cost data presented in this section were extracted from EPA reports (U. S. EPA, 1980c; U.S. EPA, 1999, 2000a, 2006) and bid Summary Sheets provided by the various EPA regions (R8: Brobst, 2007; R5: Martin, 2007; R9: McNaughton, 2007; Oklahoma: Rajaraman, 2007). The costs extracted from the EPA report (1980c) were indexed to Kansas City/St. Joseph, Missouri during the fourth quarter of 1978. These data were projected for Kansas City to 2006 and the bid sheets corrected to Kansas City as a baseline using the ENR CC Indices (www.enr.com), which are available by subscription. General information about construction costs is available in Fact Sheet 5: Treatment Series, Lagoons, Performance and Cost of Decentralized Unit Processes (werf.org/AM/Template). To compare costs of ponds with other types of treatment, it is suggested that the engineer consult relevant references for her/his region.

Construction costs only are represented in Figures 8-1 through 8-4. Associated costs include administration/legal, preliminary, land, structures, right-of-way, mobilization, architect/engineer (A/E) basic fees, other A/E fees, project inspection costs, land development, relocation, demolition and removal, bond interest, indirect costs, miscellaneous, equipment, and contingencies. These represent approximately 50 percent of the construction costs.

Figure 8-1 contains both the 1978 corrected data and the data from the bid summary sheets for flow through ponds (facultative). Figure 8-2 contains data extracted from the low bid on the bid summary sheets for flow through ponds. Predicted construction costs using the equations of best fit from Figure 8-1 and 8-2 result in similar estimates, but the estimates deviate considerably from individual construction cost values.

The low flow rates presented in the bid summary sheet data are lumped together with several other data points that does not seem to influence the fit of the data. With one exception, the R^2 of the bid summary sheet data is = 0.705, a relatively good fit. Therefore, with low-flow systems, it is probably prudent to use the bid sheet projection equations with the best coefficient of determination (R^2) to estimate the cost of a flow-through pond.

Insufficient data were available for the non-discharging and aerated ponds, therefore they could not be compared to the bid summary sheet data combined with the updated Kansas City data. The data points from the bid sheets agreed reasonably well with the up dated information.

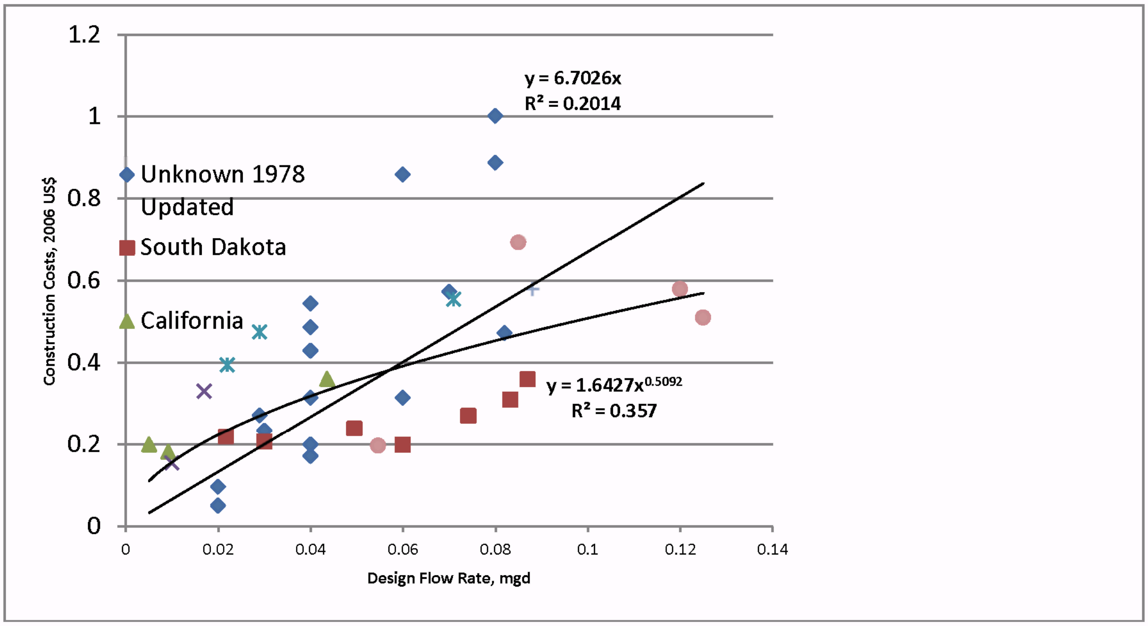


Figure 8-1. Construction costs vs. DFR for flow-through ponds (facultative), Kansas City, 2006. (DFR < 500,000 L/d [0.130 MGD]) . See p. xiv for conversion table.

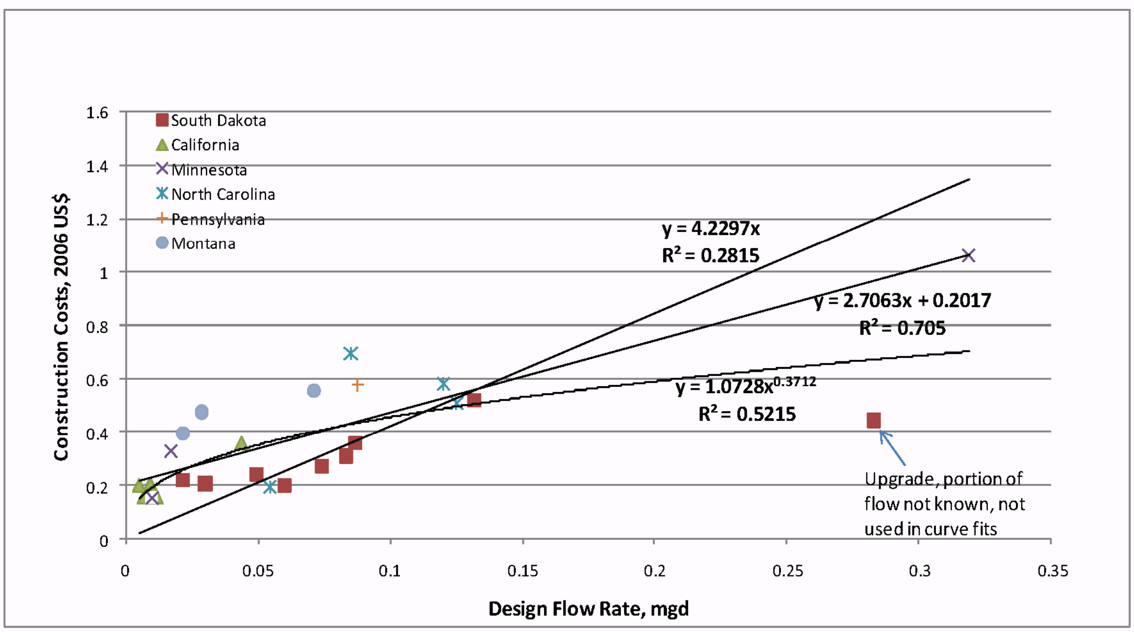


Figure 8-2. Data bid tabulations: construction costs vs. DFR for flow-through ponds, Kansas City, 2006.

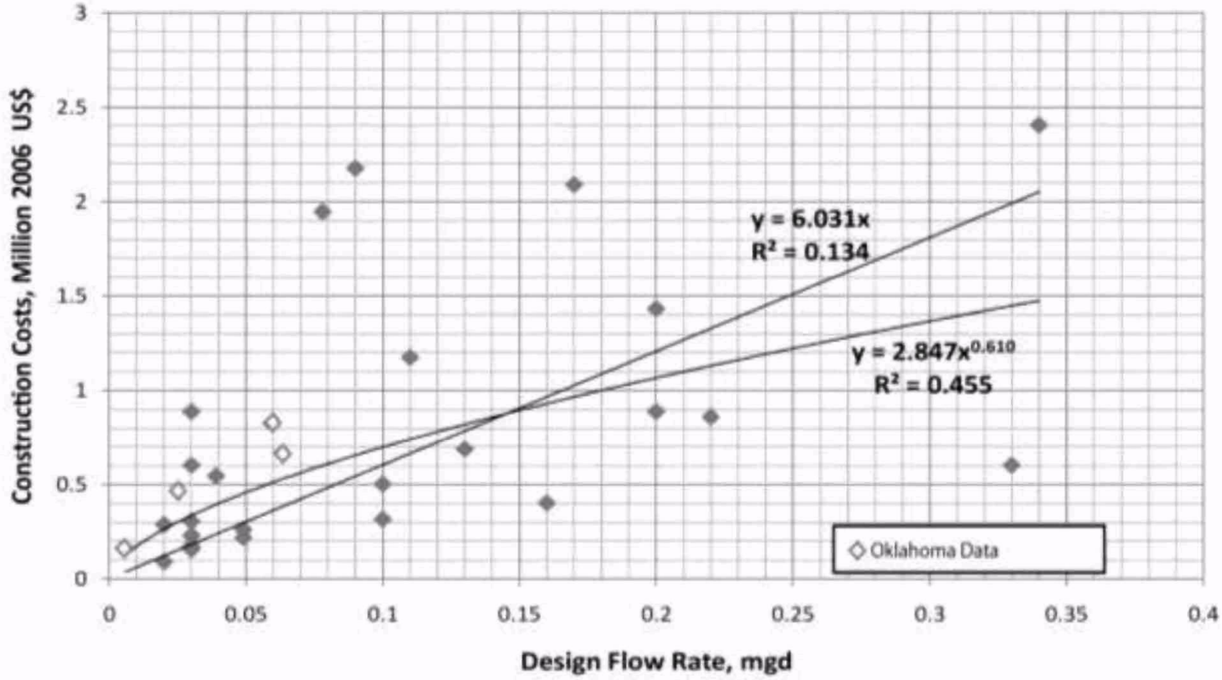


Figure 8-3. Construction costs vs. DFR for nondischarging ponds, Kansas City, 2006.

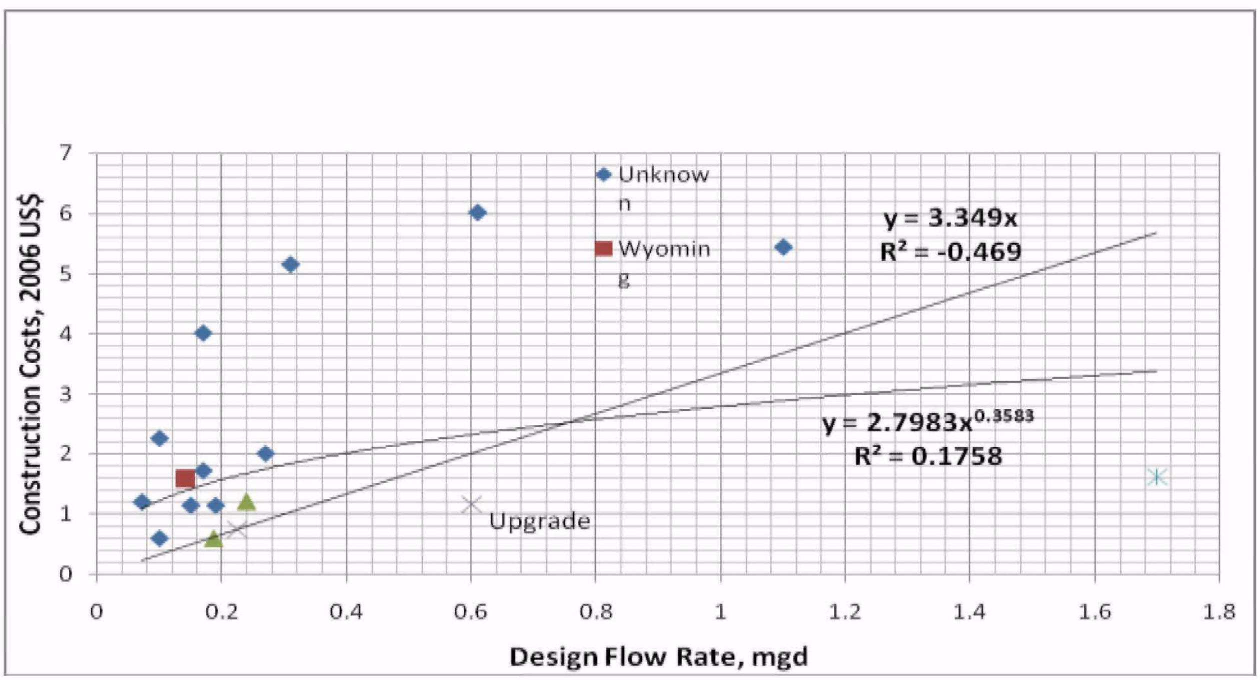


Figure 8-4. Construction costs vs. DFR for aerated ponds, Kansas City, 2006 - Q = 0 to 1.2 MGD.

8.3 UPDATING COSTS

Costs may be up dated to other cities by using the ratio of the 2006 ENR CC Index for Kansas City to the ENR CC index for the location of interest as shown in the following equation:

$$\text{Updated Construction Costs} = \frac{\text{ENR CC Index for City of Choice}}{\text{ENRCC Index for Kansas City (12/2006)}}$$

8.4 COST DATA FOR UPGRADING METHODS

There are many options for the upgrading of pond systems, but accurate cost data for all of the systems are not available. Wetlands, land application, granular media filtration, dissolved air flotation, and sequencing batch reactors cost data are relatively expensive as is shown in the following sections. Data for rock filters, intermittent sand filters, fish production, hyacinth systems and other plant applications are limited. General cost estimation techniques are presented for the systems with limited data.

8.4.1 Wetlands

8.4.1.1 Free Water Surface Wetlands (FWS) Cost Estimation

The available cost data for FWS constructed wetlands are difficult to interpret given the number of design constraints placed on the various systems. The size required and resulting costs will vary depending upon whether the systems are designed to remove BOD_5, TSS, NH_3 or total N.

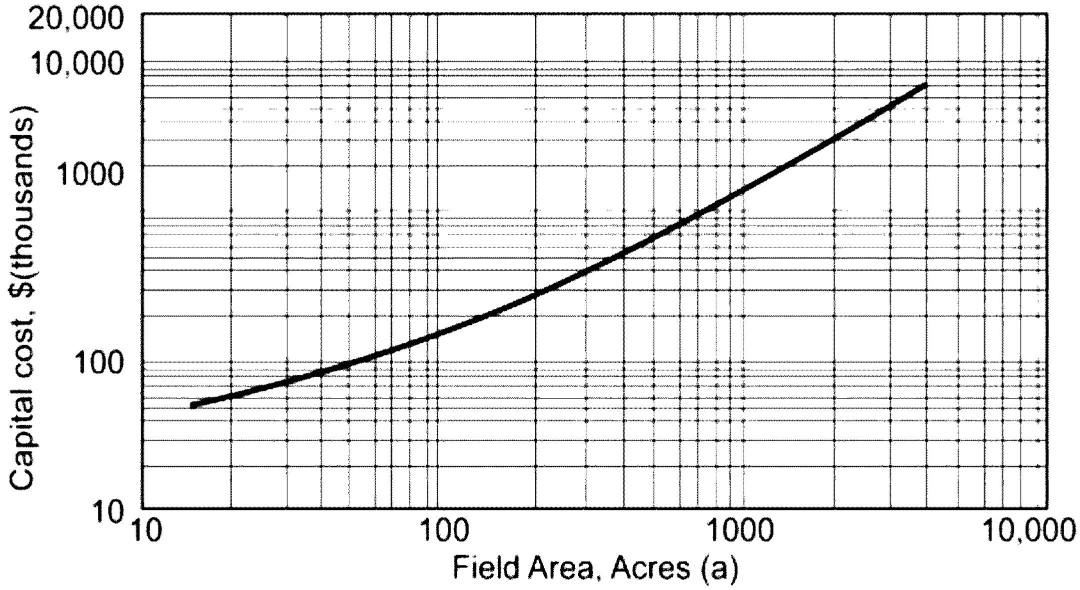
Further information about costs for FWS constructed wetlands can be found in U.S. EPA, (2000b) and Crites et al. (2006).

8.4.1.2 Subsurface Flow (SSF)

Available cost data for SSF constructed wetlands are hard to interpret because of design constraints that may be placed on the particular system. It is not clear what the design parameters were for most of the systems. Further information about costs for SSF wetlands can be obtained in U.S. EPA (2000b) and Crites et al. (2006).

8.4.2 Land Application Cost Estimation

A detailed discussion of the various types of land application treatment of wastewaters can be found in U.S. EPA (2006), Shilton (2005) and Crites et al. (2006). There are three basic land application methods: slow rate, overland flow, and soil aquifer treatment or rapid infiltration. Capital costs and labor costs were compiled in U.S. EPA 2006 for an ENR CCI of 6076. Construction costs and labor, materials and energy costs for center pivot irrigation, solid set irrigation, gated pipe overland flow and rapid infiltration are shown in Figures 8-5 through 8-8. Land application systems should only be designed by an engineer who has first-hand experience or has studied the above references.



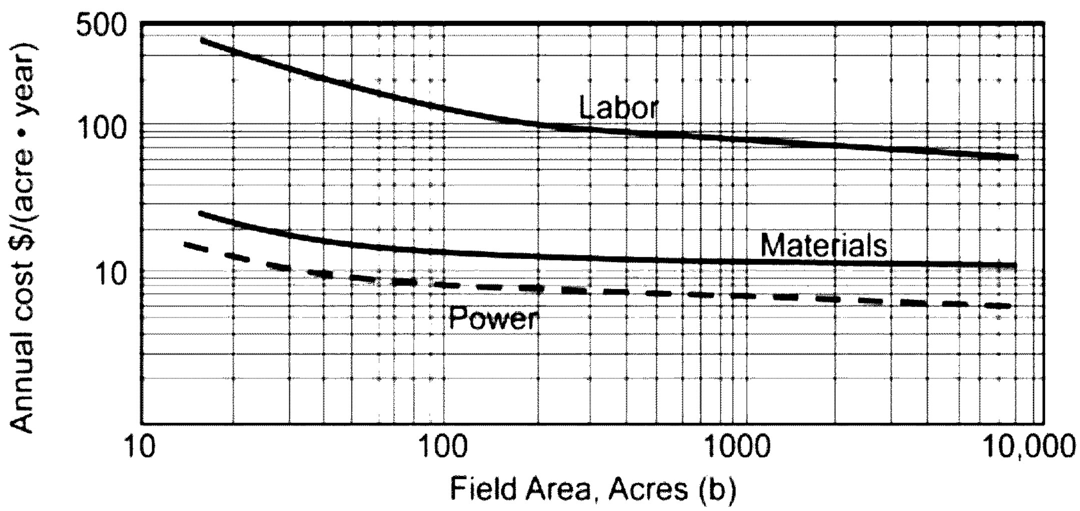


Figure 8-5. Center pivot sprinkling costs, ENR CCI = 6076: (A) capital cost; (B) operation and maintenance cost (U.S. EPA, 2006).

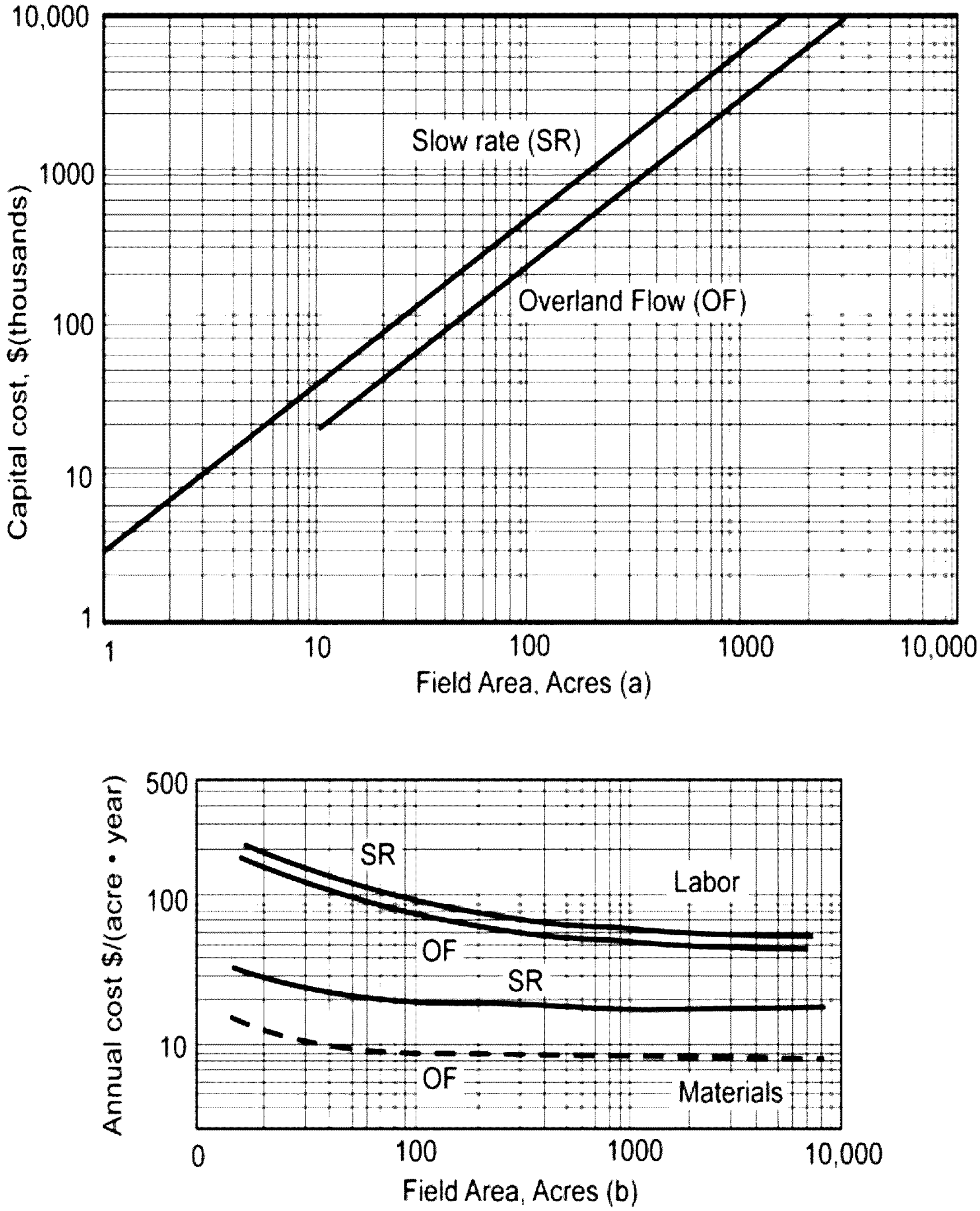


Figure 8-6. Solid set sprinkling (buried) costs, ENR CCI = 6076: (A) capital cost; (B) operation and maintenance cost (U.S. EPA, 2006).

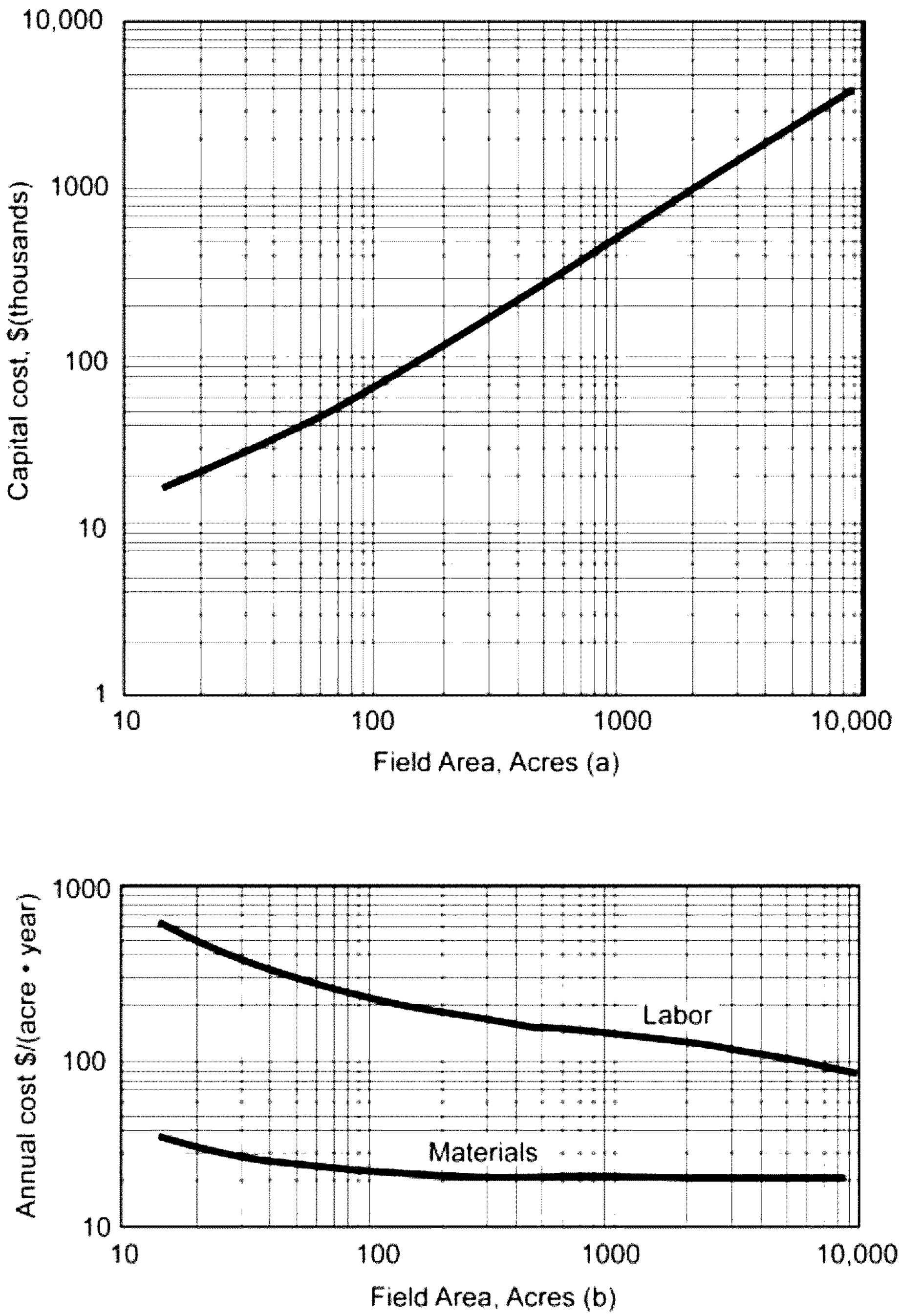
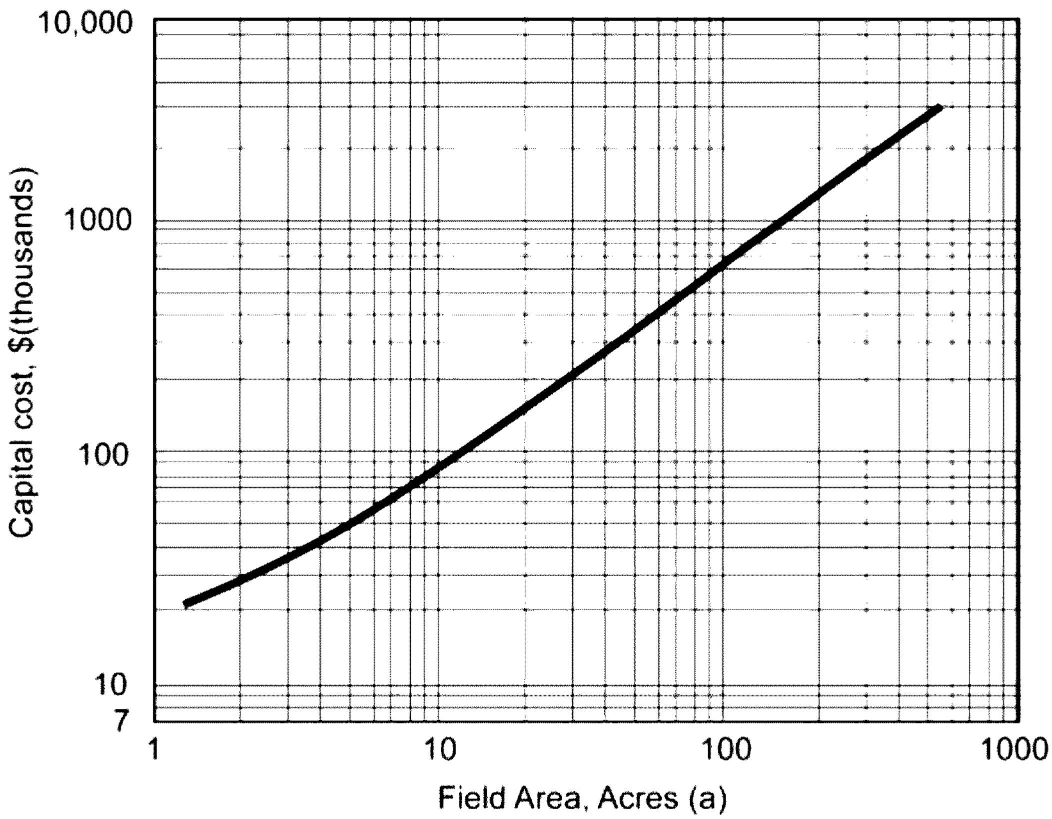


Figure 8-7. Gated pipe – overland flow or ridge-and-furrow slow rate costs, ENR CCI = 6076: (A) capital cost; (B) operation and maintenance cost (U.S. EPA, 2006).



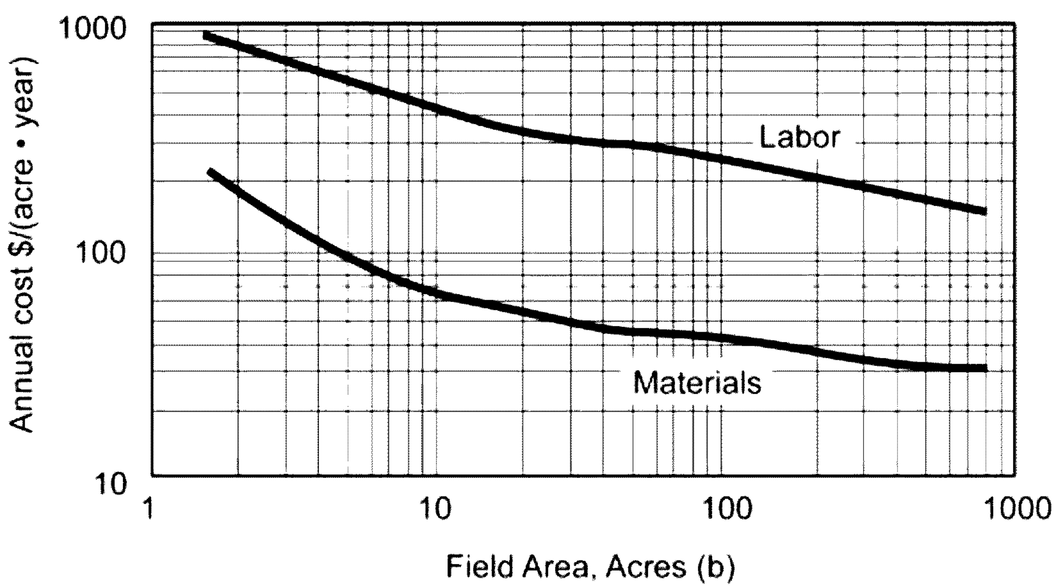


Figure 8-8. Rapid infiltration basin costs, ENR CCI = 6076.: (a) capital cost; (b) operation and maintenance cost (U.S. EPA, 2006).

8.4.3 Granular Media Filtration Cost Estimation

The relationships shown in Figures 8-9 and 8-10 were taken from the U.S. EPA (2000a) concerning the Centralized Waste Treatment (CWT) point source category. The data may not be totally accurate for pond systems, but are reasonable enough to provide guidance with regard to preliminary designs.

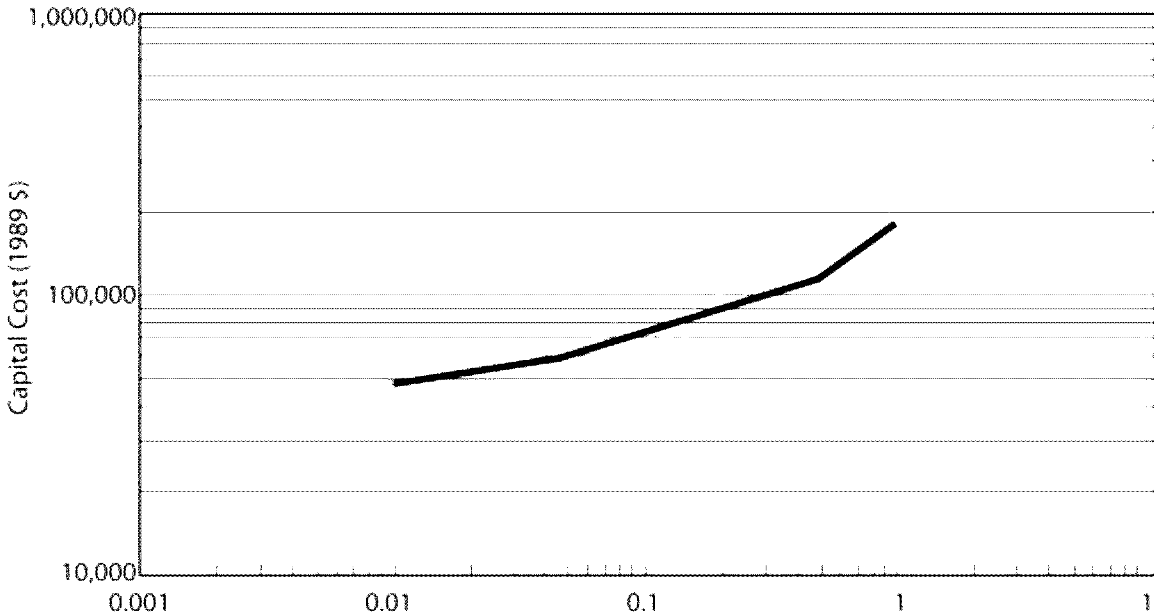


Figure 8-9. Mixed-media filtration capital costs, ENR CCI = 6076 (U.S. EPA, 2000a).

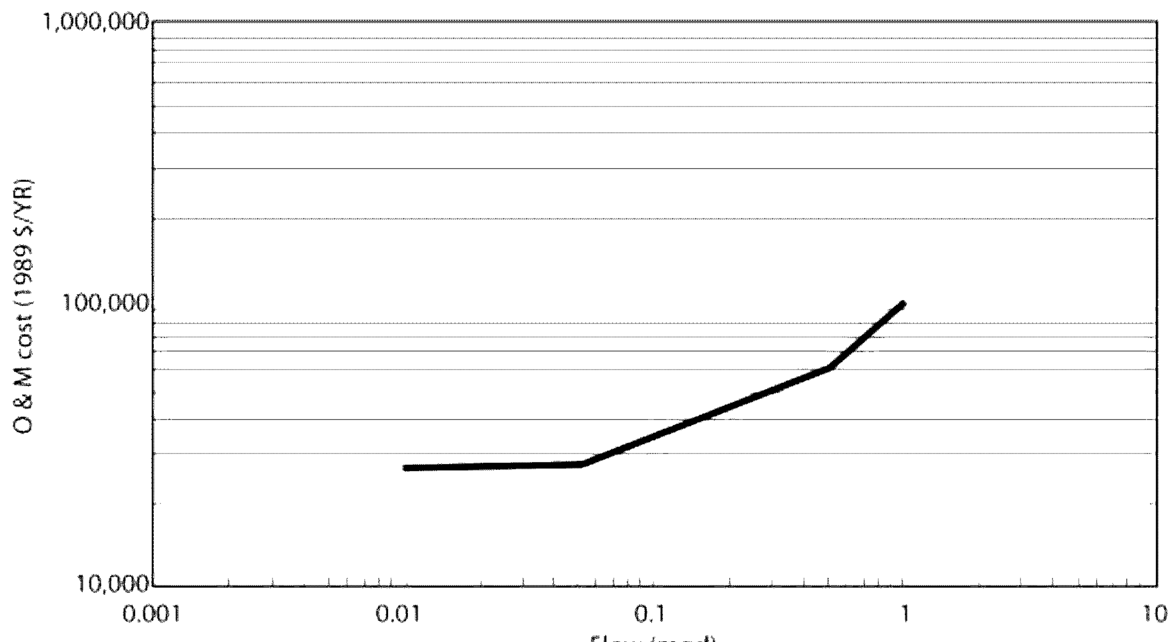


Figure 8-10. Mixed-media filtration O&M costs, ENR CCI = 6076 (U.S. EPA, 2000a).

8.4.4 Dissolved Air Flotation (DAF) Cost Estimation

The relationships shown in Figures 8-11 and 8-12 were taken from the U.S. EPA (2000a) concerning the CWT point source category. Again, the data may not be totally accurate for pond systems, but are reasonable enough to provide guidance with regard to preliminary design.

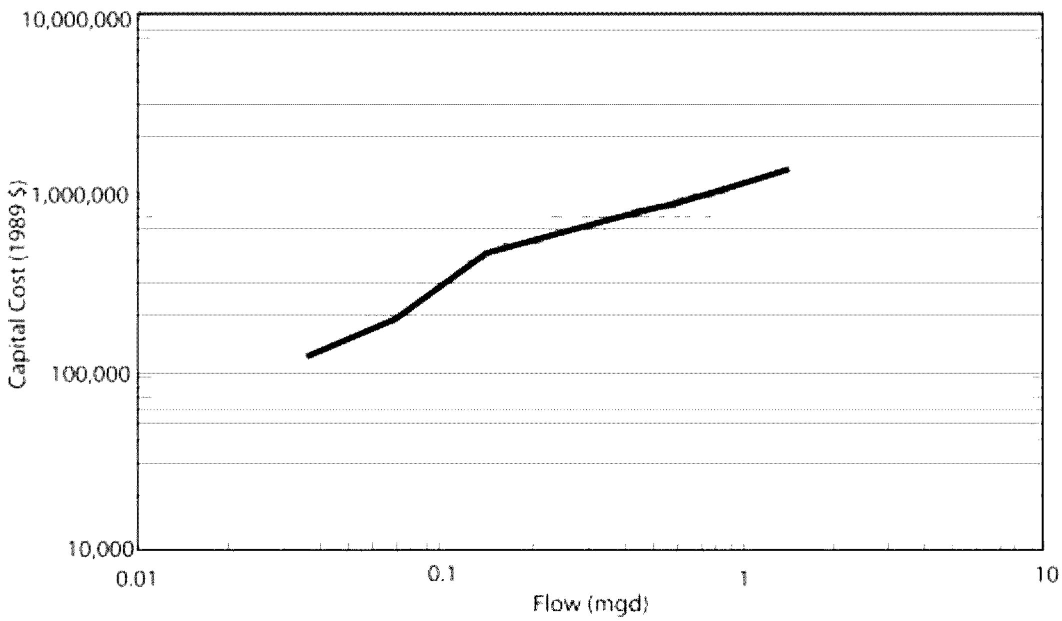


Figure 8-11. Dissolved air flotation capital costs, ENR CCI = 6076 (U.S. EPA, 2000a).

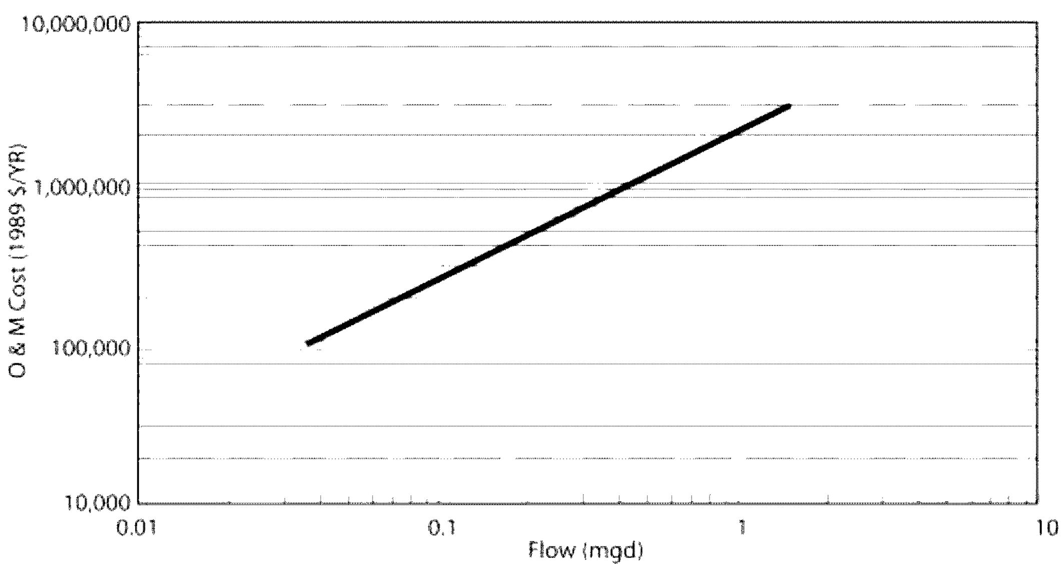


Figure 8-12. Dissolved air flotation capital costs, ENR CCI = 6076 (U.S. EPA, 2000a).

8.4.5 Sequencing Batch Reactor (SBR) Cost Estimation

The relationships shown in Figure 8-1 were taken from the U.S. EPA (2000a) concerning the CWT point source category. The data may not be totally accurate for pond systems, but are reasonable enough to provide guidance with regard to preliminary design.

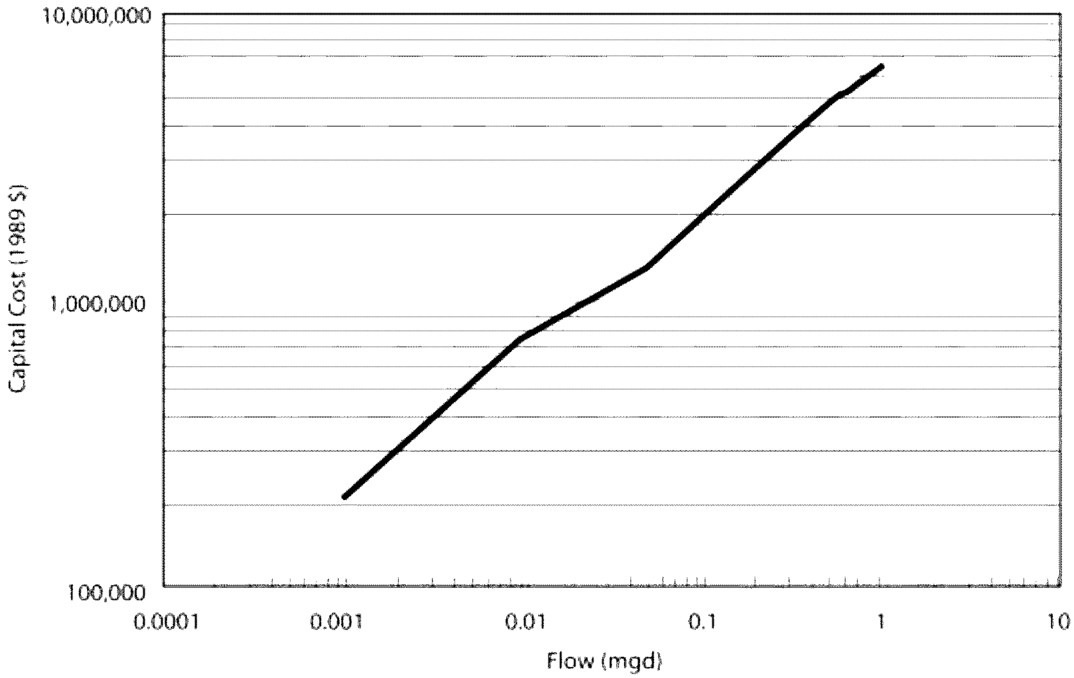


Figure 8-13. Sequencing batch reactor capital costs, ENR CCI = 6076 (U.S. EPA, 2000a).

8.4.6 Intermittent Sand Filter Cost Estimation
Given the limited cost data for intermittent sand filtration, a spreadsheet and tables were developed to assist design engineers in estimating costs associated with this polishing technique (Appendix C).

8.4.7 Intermittent Rock Filter Cost Estimation Procedure
See Section 8.4.6.

8.5 ENERGY REQUIREMENTS
Energy consumption is a major factor in the operation of wastewater treatment facilities. Many of the plans for water pollution management in the United States were developed before the cost of energy and the limitations of energy resources had to be taken into consideration. As wastewater treatment facilities are built to incorporate current treatment technology and to meet regulatory performance standards, the cost of the energy to run the processes must be considered more carefully in the designing and planning of the facilities. Planners and designers should seek out the most recent information on energy requirements so as to develop a system that incorporates the most efficient and affordable type and use of energy to treat wastewater to meet regulatory requirements consistently and reliably. Wherever possible, self-sustaining elements,

such as alternative energy sources, capture and use of energy produced (i.e., CH_4), and uses of by-products (e.g., algae) should be considered.

8.5.1 Effluent Quality and Energy Requirements

Expected effluent quality and energy requirements for various wastewater treatment processes are shown in Table 8-3. Energy requirements and effluent quality are not directly related. Facultative ponds and land application processes can produce excellent quality effluent with smaller energy budgets. The same is true for several other combinations.

Table 8-1. Total Annual Energy for Typical 1 mgd System Including Electrical plus Fuel, expressed as 1000 kwh/yr (Middlebrooks et al., 1981).

TREATMENT SYSTEM	EFFLUENT QUALITY, mg/L				ENERGY (1000 KwH/YR)
	BOD_5	*TSS*	*P*	*N*	
Rapid infiltration (facultative pond)	5	1	2	10	150
Slow rate, ridge & furrow (facultative pond)	1	1	0.1	3	181
Overland flow (facultative pond)	5	5	5	3	226
Facultative pond + intermittent sand filter	15	15	-	10	241
Facultative pond + microscreens	30	30		15	281
Aerated pond + intermittent sand filter	15	15		20	506
Extended aeration + sludge drying	20	20	-	-	683
Extended aeration + intermittent sand filter	15	15	-		708
Trickling filter + anaerobic digestion	30	30	-	-	783
RBC + anaerobic digestion	30	30			794
Trickling filter + gravity filtration	20	10	-	-	805
Trickling filter + N removal + filter	20	10	-	5	838
Activated sludge + anaerobic digestion	20	20	-	-	889
Activated sludge + anaerobic digestion + filter	15	10	-	-	911
Activated sludge + nitrification + filter	15	10	-	-	1,051
Activated sludge + sludge incineration	20	20	-	-	1,440
Activated sludge + AWT	<10	5	< 1	< 1	3,809
Physical chemical advanced secondary	30	10	1	-	4,464

CHAPTER 9

OPERATION AND MAINTENANCE

9.1 INTRODUCTION

The information presented in this chapter covers various topics related to operations and maintenance, including control testing, safety, troubleshooting and optimizing operations of a pond system. Much that has been written on these topics in the past 30 years remains valuable, but may not be readily available. This chapter summarizes much of that information and brings it up to date. It introduces the basic tools used to monitor an operating pond treatment system and provides guidance when the system does not seem to be functioning as designed. See also Appendices E (Troubleshooting) and F (Wisconsin Department of Natural Resources Study Guide *Introduction to Advanced Stabilization Ponds and Aerated Lagoons* (www.dnr.wigov/org/es/science/opert/doc).

9.2 TERMINOLOGY

9.2.1 Basic Nomenclature

A pond system is typically a number of earthen basins connected together to treat raw wastewater.

9.2.2 Types of Pond Systems

9.2.2.1 Anaerobic Ponds

Anaerobic pond cells receive such a heavy organic loading that there is no aerobic zone.

9.2.2.2 Facultative Ponds

Facultative pond cells are 1.2-2.4 m (4 - 8 ft) deep with an aerobic surface layer and an anaerobic bottom layer.

9.2.2.3 Aerated Ponds

The O_2 in the aerated pond cells is supplied or supplemented by surface mechanical or diffused aeration equipment.

9.2.3 Flow Configuration

Pond systems can be operated either in series or parallel.

9.2.3.1 Series

In series operation the influent wastewater flows into the primary cell, then to the secondary cell and finally to the polishing cell before being discharged.

9.2.3.2 Parallel

In parallel operation, the operator has the option of splitting the flow. This is usually done in equal parts between the first two cells. Facultative ponds are commonly operated in parallel under winter conditions. As water temperatures drop, biological activity is reduced and the primary cell of a facultative pond system can become organically overloaded. To prevent this, the flow is sent to the first two cells at the same time, which reduces the organic load to each one. As water temperatures warm up in spring, biological activity increases, and the flow regime can return to series operations.

9.2.3.3 Recirculation

The operational flexibility to recirculate wastewater in a treatment plant or pump back treated wastewater from the end of the process to the first series of cells can be used to great advantage. The operator can introduce treated wastewater with a higher dissolved O_2 concentration into the first series of cells that have a higher organic loading. Recirculation also tends to smooth out the performance of the system. Entering flow varies significantly over 24 hours, and recirculation can create a more uniform flow rate.

9.3 CONTROL TESTING INFORMATION

For a facility to be consistently in compliance with discharge permit requirements, adequate process controls must be in place. The influent quantity and quality should be monitored on a regular schedule to provide the information needed to treat the wastewater stream adequately and operate the facility properly. It is also necessary to monitor the processes within the system in order to solve any effluent water quality problems. The influent, the internal pond processes and the plant effluent should be evaluated on a regular basis.

The wastewater must be analyzed for a number of water quality parameters. Typical tests and measurements include flow, temperature, *p*H, DO, BOD_5, soluble BOD_5 ($SBOD_5$), $CBOD_5$, TSS, NH_3, P, coliform (fecal or total) and chlorine (Cl^-) residual. The results of these tests are used to determine whether the treatment process is reducing the wastewater contaminants and to predict the impact of potential operational changes. Some of the tests can be performed with basic equipment (flow, temperature, *p*H, DO and Cl^- residual) and some require more time, specialized equipment and technical expertise (BOD_5, $SBOD_5$, $CBOD_5$, TSS, NH_3 and coliform). The operator of a small pond system may elect to collect samples for these tests and have a commercial lab complete some of the analyses. (U.S. EPA, 1977a).

All system operators will need to review the facility discharge permit to determine sample parameters to be tested, sample type, location and frequency needed to meet permit compliance. Operators of systems that have a controlled discharge will need to perform tests during the period prior to and during discharge as required by the regulatory agency. Those systems operating on a hydrograph controlled release discharge basis must monitor the treatment plant process and the quantity and quality of both the effluent and the receiving stream.

9.3.1 Sample Collection

Samples collected for analysis must be representative of the water being tested, which requires that they be taken at a location where the wastewater is well mixed and not subject to short circuiting. If the sample is to be stored before testing, it must be refrigerated. Sample containers must always be cleaned to method specifications before sampling to avoid confounding the results with background contamination. Temperature, *p*H, Cl^- residual and DO should always be taken in the field to prevent false readings and should be taken at the same time each day. These parameters should be measured at other times of the day from time to time to gain an understanding of the changes that occur throughout the day.

9.3.2 Types of Samples

9.3.2.1 Grab

Grab samples, taken at no set time or flow, are used to measure temperature, *p*H, DO, fecal and total coliform and Cl^- residual. As raw sewage flow varies in content, as well as volume, over the course of the day, samples taken at sunrise and in mid-afternoon and analyzed separately will yield the most information. Grab samples of effluent from controlled discharge ponds should be taken during discharge, perhaps one sample every two hours, but then should be combined into one composite sample (see Section 9.3.2.2). The operator should review state guidelines for specific information.

9.3.2.2 Composite Samples

9.3.2.2.1 Volumetric Composite

A volumetric sample is taken by collecting individual predetermined sample volumes at regular intervals over a selected period of time, usually using a sampling device designed for this purpose (see Section 9.3.3). The samples must be refrigerated. They are then mixed together and considered to be a representative sample for whatever analysis is being performed.

9.3.2.2.2 Flow Proportional Composite

A flow proportional composite is taken by collecting individual samples at regular intervals over a selected period of time. A flow measurement is taken and recorded at the time the individual sample is collected. All samples must be refrigerated. At the end of the sampling period, each sample is stirred and an amount that is proportional to the flow at the time the sample was taken is poured into the composite container.

9.3.2.2.3 Automatic Samplers

There are numerous types of automatic samplers on the market. Some are self-contained with battery packs while others must have an external power source. They take samples at chosen intervals, some as frequently as every 10 minutes, and composite the samples as they are collected. The samplers can be connected to existing flow measuring devices or may have built-in flumes that deliver flow-proportional composite samples. This equipment is most useful for sampling raw wastewater flow, but can be used for effluent as well.

9.3.3 Handling and Preservation of Samples

Sewage samples rapidly undergo biochemical changes if they are subjected to summer temperatures or freezing temperatures or if exposed to sunlight. Thus, collected samples should be transferred as soon as possible to a refrigerator. Keeping samples at a temperature of 4 °C reduces post-collection biochemical changes for 24 hours. Samples taken for bacterial analyses should be collected separately and analyzed or sent to a laboratory for analysis within 30 hours of collection (http://www.epa.gov/OGWDW/methods/methods.html).

Containers used for sample storage should be as clean as required for the specified method analysis. Stoppered glass bottles or wide-mouthed jars are preferred and are easiest to use for mixing and cleaning. Bacterial samples should be collected in sterile containers.

9.3.4 Sample Point Locations

9.3.4.1 Pond Influent

Samples of the raw wastewater can be collected at the wet well of the influent pump station, a manhole at the inlet diversion control structure, or the influent headworks.

9.3.4.2 In Pond

Pond composite samples should consist of four equal portions taken from four corners of the pond. The sample should be collected 2.4 m out from the water's edge and 0.3 m below the water surface or at the transfer structures between the cells if these are present. Care should be taken to avoid stirring up material from the pond bottom and should not be taken near mechanical aerators or during or immediately after high wind or strong storms, as these processes may stir solids into the water column.

9.3.4.3 Effluent

Effluent samples can be collected from the final cell outlet structure or at a well-mixed location in the outfall channel prior to mixing with any dilution waters such as the receiving stream waters.

9.3.5 Tests and Measurements

Test results, along with visual indicators, are used by the operator to evaluate whether the pond is in discharge permit compliance. The following sections describe the tests.

9.3.5.1 Temperature

The temperature of the influent wastewater can be used to detect inflow and infiltration (I&I) and some industrial wastes. A sudden increase in temperature may indicate the presence of warm industrial wastes. On the other hand, influent temperatures may cool rapidly in late fall and early winter. In one case, an investigation of the collection system revealed that owners of poorly insulated homes were bleeding their internal plumbing systems to prevent freezing of the pipes. This cold water diluted the influent sewage strength and cooled the temperature, which reduced the ability of the system to treat the waste.

Temperature can also be used to predict treatment efficiency and mode of operation (parallel or series) and estimate the necessary HRT. As influent water temperature cools, a facultative pond system may need to be changed from series to parallel operation to reduce organic loading to each cell. A mechanically aerated pond system subject to cooler ambient temperature may need to have all cells in series operation to obtain the correct HRT for continuous permit compliance. Conversely, as influent water temperature increases, a facultative pond system may need to be changed from parallel to series operation. The operator of a mechanically aerated pond system may choose to remove an individual cell from operation to reduce the overall HRT and prevent a possible algal overgrowth condition.

9.3.5.2 Flow

Keeping a record of accurate flow measurements is essential for successful operation and control and troubleshooting pond systems. Influent flow measurement can be used to detect I&I

problems, determine the HRT of a cell, calculate the organic loading to a cell, provide data for determining mode of operation and calculate appropriate chemical dosages. Effluent flow measurement is a requirement of the discharge permit and can be used to calculate chemical dosage needed for disinfection. Most states require both influent and effluent flow measurement to determine the extent of infiltration and/or exfiltration from the cells, depending on the distance of the system to groundwater.

9.3.5.3 *p*H Value

Large fluxes in influent *p*H may signal an industrial and/or septic waste problem. The range of *p*H for normal domestic influent waste is 6.8 - 7.5, depending on alkalinity and hardness of the water. The *p*H is a good indicator of the health of the pond system. Pond cells that have a dark green color generally have a high number of green algae and a corresponding higher *p*H. Algae take up CO_2 in the photosynthetic process. If CO_2 is not available, the algae will utilize a carbon source from the HCO_3^- alkalinity, which drives up the *p*H to 9.5 or above. At night, both the algae and aerobic bacteria utilize O_2 and produce CO_2. The CO_2 in solution forms carbonic acid (H_2CO_3) and drives the *p*H down. These diurnal *p*H patterns are indicators of internal pond conditions. Pond cells that appear black or gray in color and have a decreasing *p*H value (< 6.8) may be septic or moving toward a septic condition.

9.3.5.4 Dissolved Oxygen

Dissolved oxygen is an essential indicator of aerobic biological activity. The DO test is performed on a grab sample and must be performed immediately. The easiest method for analyzing for DO is with a portable meter. The test should be performed at sunrise and again around 2 - 3 p.m. Large fluctuations in the primary cell may signal problems with the influent, such as shock loading or toxic waste problems. Some fluctuation in day-to-day DO in the pond system is expected. The operator should plot daily readings and identify trends in concentrations. A decreasing trend in DO in the early morning test may indicate an increasing organic load, a developing short-circuiting problem or an algal overgrowth problem. All measures should be taken to avoid the DO concentration dropping to zero. This will cause incomplete treatment of the wastewater and will result in discharge permit violations. The operator may have to take corrective action, such as increasing aerator running time or aeration capacity or switching to parallel operation. An increasing trend in the DO concentration, on the other hand, may allow the operator to decrease aerator running time or switch from parallel to series operation.

9.3.5.5 Dissolved Oxygen Profile

A DO profile is developed by taking a series of DO measurements at 0.3 m increments from top to bottom on an individual cell. An informal grid system is established to ensure uniform coverage of the cell (Figure 9-1). This test is best performed by two people for safety reasons and data recording needs. A boat and a portable temperature-compensating DO meter with a long probe are required. The probe should be marked off in 0.3 m increments from the tip of the probe to a length that will allow the operator to have the probe touch the bottom of the cell from the boat. The water depth at which the probe goes slack should be recorded.

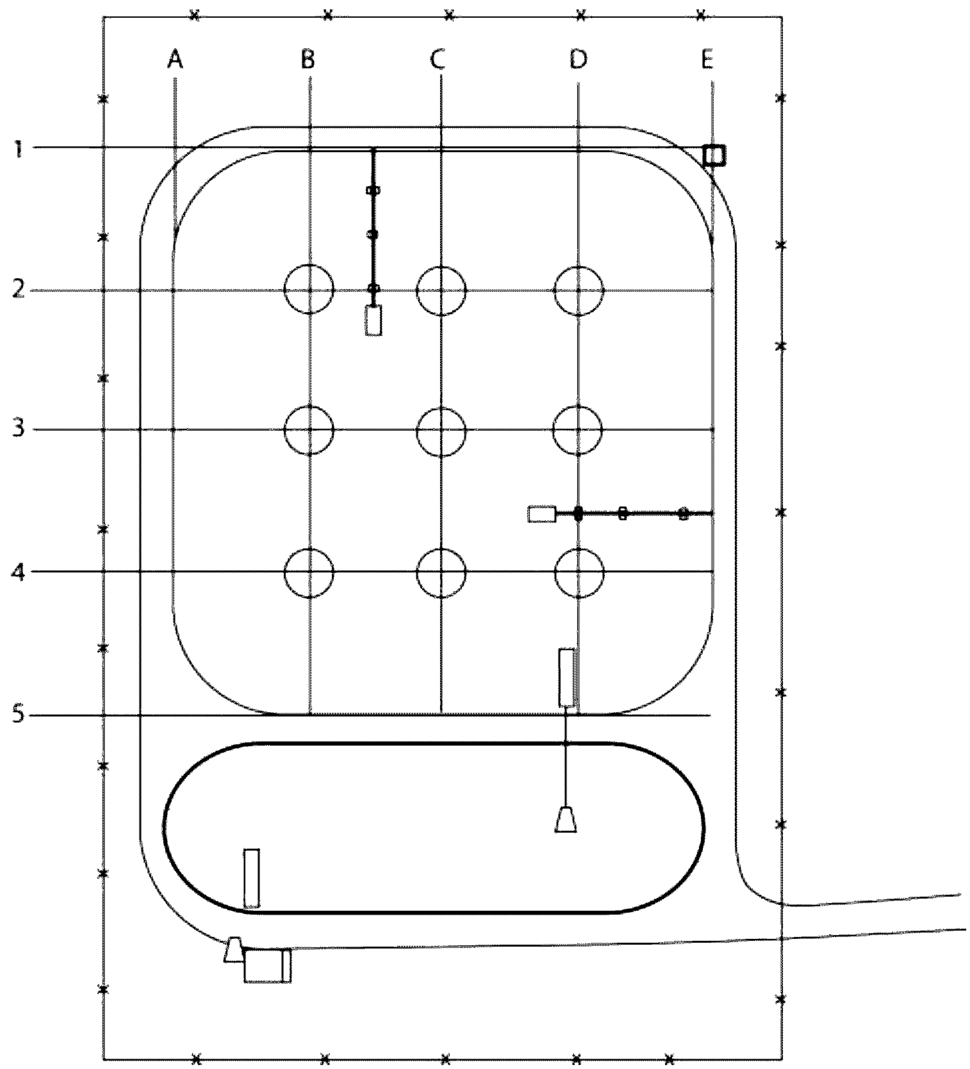


Figure 9-1. Sampling grid system (Richard and Bowman, 1991). Sampling points are located where lines with letters and numerals intersect.

This test allows the operator to determine if there is a DO deficit during evening hours when O_2 depletion is greatest due to biological activity. The test will also indicate if the pond is completely mixed or stratified; identify areas of low DO or dead spots; draw attention to short circuiting; and help define bottom pond contours and areas of possible sludge build-up. The test should be performed just as it is getting light. As is seen in Figure 9-2, the lowest O_2 reading is noted just prior to sunrise. Once it is daylight, the algae photosynthesize, which may become a supersaturated DO condition in early afternoon. As light intensity lessens and photosynthetic activity diminishes, O_2 is depleted. During the night, if the DO level in the ponds drops to zero, aerobic decomposition of organic matter stops, causing incomplete treatment. This can lead to BOD_5 permit violations.

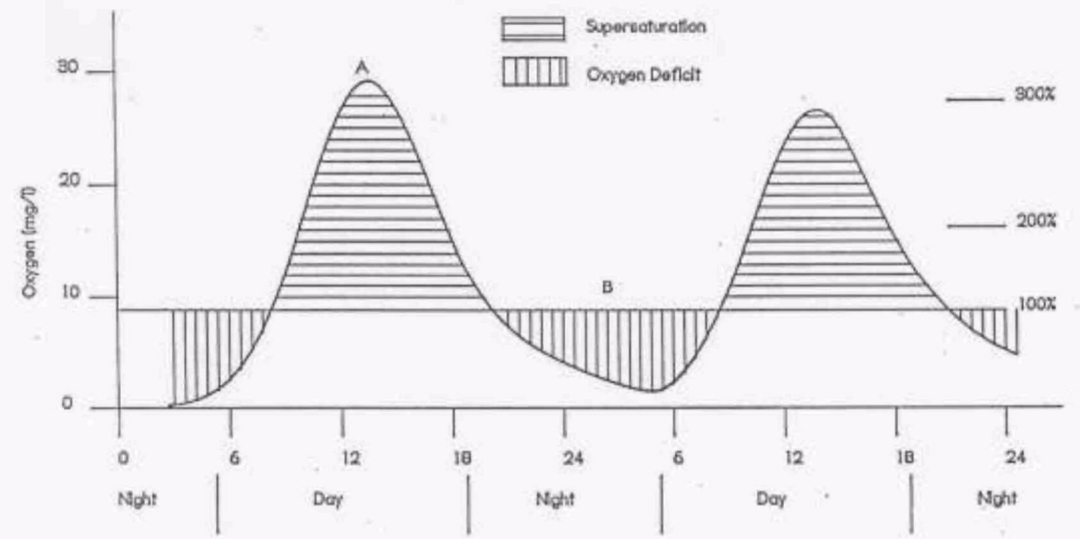


Figure Diurnal Oxygen Concentration in a Lagoon.

NOTE: The graph indicates that the lowest dissolved oxygen levels occur early in the morning due to the aerobic bacteria and algae utilizing oxygen during the night. The highest dissolved oxygen level is late in the afternoon due to algal photosynthesis. It is likely the aerators must run more at night to compensate for the algae metabolizing oxygen. It is important to insure proper aerator operation during the day for adequate mixing and to eliminate potential stratification.

Figure 9-2. Diurnal O_2 curve (Richard and Bowman, 1997).

9.3.5.6 Chlorine Residual

The *Cl* residual test determines the amount of *Cl* present after the detention time for disinfection (to destroy fecal coliform) has been met. The general rule is that there should be 0.5 mg/L remaining after a contact time of 1 hour. This grab sample must be tested immediately. The operator should review the National Pollution Discharge Elimination System (NPDES) permit, as some states may not require disinfection. Some states may designate a prescribed coliform minimum, and some may require dechlorination after disinfection.

9.3.5.7 BOD_5

The BOD_5 test measures the amount of O_2 used (depleted) by the microorganisms in metabolizing the organic material in a sample of wastewater. It is an indirect measurement of the relative organic strength of a sample. The test is performed over five days in a controlled environment. The sample is placed in an incubator at 20 $^{\circ}$C with no light or additional O_2. The test is used to determine the influent wastes organic strength, calculate the organic loading to the pond system or an individual cell, make decisions about operational changes and determine whether the effluent is in compliance with the discharge permit.

The BOD_5 test can be misleading if there is a high concentration of algae in the effluent discharge. The BOD_5 test is an O_2 depletion test that is run in the dark. When algae are in the dark for long periods of time they cannot photosynthesize and will instead use O_2. The up take by the algae also reduces the O_2 concentration at the end of the five-day period. The result is a calculated BOD_5 value that is higher than it would be if it measured the organic loading from wastewater alone.

9.3.5.8 Soluble BOD_5

For this test, the BOD_5 sample is filtered using a 0.45 μ filter before the test is run. The filter should be observed for any obvious change and the color noted. The filtrate is then prepared in the same way and run at 20 °C, in the dark. It is recommended that a standard BOD_5 test and a soluble (filtered) BOD_5 test be run at the same time and the results compared. This is a valuable troubleshooting tool that will be discussed in Section 9.3.6.

9.3.5.9 Carbonaceous BOD_5 ($CBOD_5$)

It has long been recognized that ponds will nitrify (convert NH_3 to NO_3^-) under certain conditions. This process uses approximately 1.8 kg O_2 per 453 g of NH_3 converted to NO_3^- and approximately 4.1 kg of alkalinity per 453 g NH_3 converted NO_3^- (see Section 1.3.4.5 for a discussion of alkalinity). The nitrification process will continue in the BOD_5 bottle causing a higher BOD_5 test result. To compensate for this nitrogenous BOD in effluent BOD_5 testing, a $CBOD_5$ test can be substituted for the BOD_5 test. A $CBOD_5$ test is a BOD_5 test with inhibitor added to prevent O_2 depletion due to nitrification. To compensate for nitrification effects, regulatory agencies may allow operators to amend the discharge permit to substitute a $CBOD_5$ effluent limit for the BOD_5 parameter.

9.3.5.10 Suspended Solids

The suspended solids (TSS) test measures the dry weight of solids retained on a glass fiber or Millipore™ filter and is expressed in mg/L. Equipment required for this test includes a drying oven, a desiccator and a weighing balance. Tests must always be run on composited samples from both the influent and effluent.

Suspended solids removal is as important as BOD_5 removal in preventing stream pollution. In normal domestic sewage, the concentration of TSS and BOD_5 are similar. The origin of the suspended solids in the influent is not the same as in the effluent, however, as the former comes from the sewage, while the latter from algae growing in the final pond. As a result, the TSS may be higher than the BOD_5 in the effluent from pond systems.

9.3.5.11 TSS to BOD_5 Ratio

Effluent BOD_5 violations are often accompanied by high effluent TSS concentrations. Table 9-1 presents TSS to BOD_5 ratios that may indicate the cause of violations.

Table 9-1. TSS to BOD_5 Ratios as Problem Indicators (Richard and Bowman , 1997).

TSS to BOD_5 Ratio	Cause(s)
<1	old sludge solubilization and release of soluble BOD_5 nitrification in the BOD_5 test bottle
1	poor treatment or short circuiting with untreated wastewater mixing with the effluent
1.5	normal for most pond systems
2.0-3.0	algal overgrowth loss of old sludge particles

9.3.5.12 Microscopic Solids Analysis

The microscopic solids analysis is a test performed with samples from each pond cell taken at the transfer point and the final effluent before chlorination (Richard and Bowman, 1991). This requires the microscopic examination of fresh pond samples using a phase contrast microscope capable of achieving a total magnification of 1000X. Solids types present and their relative abundance are measured. Solids type categories include: (1) raw wastewater or sludge solids; (2) treatment solids (bacterial flocs); (3) filamentous bacteria; (4) sulfur bacteria; and (5) algae. These solids types are easily recognized with a little experience (Richard and Bowman 1997). The significance of these solids types is listed in Table 9-2.

Table 9-2. Problems Associated with Types of Solids (Richard and Bowman 1997).

Solids Type Present	**Indicated Problem (s)**
Raw wastewater solids	short-circuiting poor aeration and improper waste stabilization
Old sludge particles	sludge buildup and need for sludge removal
Treatment solids (bacterial flocs)	organic overloading or sludge accumulation
Filamentous bacteria	indicators of low oxygen conditions or septicity
Sulfur bacteria	anaerobic conditions and sulfides
Algae	algal overgrowth

9.3.5.13 Soluble and Total BOD_5

The difference between these two values is the amount of particulate BOD_5 present. A typical domestic wastewater contains 40 percent soluble and 60 percent particulate BOD_5. The soluble BOD_5 is rapidly removed in wastewater treatment and it is unusual to see a soluble BOD_5 in the effluent that is greater than 20 percent of the total. A particulate BOD_5 value greater than 70 percent of the total BOD_5 in the effluent indicates a solids loss problem. The microscopic examination is then used to identify the types of solids being lost (Richard and Bowman, 1997).

9.3.5.14 Microbial Tests

The coliform and other microbial tests (total and fecal coliform, *E. coli* and enterococci) indicate the possible presence or absence of pathogens (human disease-causing organisms). The sources of this group of organisms are the excreta of man, mammals, and birds. Tests are always run on grab samples collected in a sterile container. Recent regulations regarding which test method may be used for a particular discharge requirement may be found in the [Federal Register notice of 3/26, 2007 (http://www.epa.gov/fedrgstr/EPA-WATER/2007/March/Day-26/w1455.htm)].

9.3.5.15 Nitrogen

Wastewater contains organic N (protein) and NH_3. Organic N is converted to NH_3 by bacteria as protein is broken down. The NH_3 is further oxidized to $NO_2^{=}$ and then to NO_3^{-} by nitrifying bacteria. This latter step is called nitrification. In some cases, it is necessary to remove NO_3^{-} in order to control algal growth. Oxygen is removed under anaerobic conditions (NO_3^- is a source of O_2 for the anaerobic bacteria), and NO_3^- is reduced to nitrogen gas (N_2). This is the denitrification step.

Ponds will nitrify and produce low effluent NH_3 concentrations under certain conditions, particularly in the warmer months of the year. As wintertime cools the wastewater temperatures, longer HRTs are required to reduce both $CBOD_5$ and nitrogenous BOD_5. Ultimately, nitrification will cease at approximately 5 - 8 °C. In colder climates, ponds cease to nitrify and will actually produce NH_3 in the wintertime and early spring. Installation of floating insulated covers may extend the period of time a pond system will nitrify NH_3.
Low DO and low alkalinity can also limit nitrification. Typically, DO levels of 2.0 mg/L are required to optimize nitrification. Total carbonate (CO_3^{2-}) alkalinity of less than 60 mg/L usually limits nitrification. Nitrification can be increased or prolonged by raising the DO level in the pond and increasing the alkalinity by adding an inorganic C source such as lime.

9.3.6 Important Visual and Olfactory Observations

Operators' visual and olfactory observations are important pond troubleshooting tools. Color and odor can be important indicators of pond health and ability to meet discharge permit standards (Table 9-3).

Table 9-3. Important Indicators in Pond Troubleshooting (after Richard and Bowman, 1991).

Pond Appearance	Odor	Microscopic Observation	Problem	Solution
Clear	None	Little suspended material	None	None
Brown	Earthy	Small bacterial flocs	None; usually good operation	None
Grey-black floating sludge gas bubbles on pond surface	Septic-sewage	Precipitated sulfides in flocs; often filamentous sulfur bacteria	Organic overloading; low dissolved oxygen; influent sludge short circuiting	Increase aeration capacity, add baffles or additional cells, improve inlet-outlet design recirculation, remove sludge accumulation
Green	Grassy or earthy	Green algal bloom	Algal bloom, pH often >9; long detention time; organic under-loading	Recirculate; addition of a settling pond, land discharge
Floating mats of blue-green algae	Fishy	Blue-green bacterial bloom	See above	Remove cells from operation, decrease water depth to decrease HRT; CAUTION: MAY INCREASE ALGAL BLOOM
Red streaks	None or septic	High amounts of *Daphnia*	*Daphnia* overgrowth, often after algal bloom	Increase aerator running time, recirculate
Entirely red or pink	Septic (rotten egg odor)	Sulfur bacteria (*Chromatium* spp.)	Anaerobic; gross organic overloading and under aeration.	Increase aerator running time or increase aeration capacity, recirculate

9.3.7 Other Data

9.3.7.1 Weather

Weather plays a major role in an operator's ability to meet discharge permit requirements for a pond system consistently. The operator should keep a daily journal or log of periods of sunshine, cloudiness, air and pond temperature, precipitation (rain or snow levels should be recorded), and percentage of the individual cells that are ice covered. Prolonged periods of cloudiness may increase the effluent BOD_5 and require the operator to make a change from series to parallel operation in a facultative pond system. In an aerated pond system, the operator may be required to increase aerator running time to insure discharge permit compliance.

9.4 OPERATION AND MAINTENANCE FOR PONDS

The following sections are summaries from the Operations Manual: Stabilization Ponds, 1977, Office of Water Program Operations, U.S. EPA, Washington D.C. Another source of information about O&M for ponds is the Office of Water Programs at California State University, Sacramento, which offers training manuals and distance education courses for operators and managers on the safe operation and maintenance of wastewater collection systems, wastewater treatment plants, and utility management. Operations and maintenance of wastewater treatment ponds is found in *Operation of Wastewater Treatment Plants*, Volume 1, Chapter 9 (Kerri, 2002). To obtain more information about the program or to order the training manual, which is available in English and Spanish, go to the website at www.owp.csus.edu.

9.4.1 Operation and Maintenance Guidelines for Anaerobic Ponds

9.4.1.1 Anaerobic Ponds

A well-operating anaerobic pond is covered entirely with a dense scum blanket which helps to keep the pond anaerobic and minimizes foul odors.

9.4.1.2 Important Operation Considerations

- Keep the pond *p*H at or near neutral (*p*H = 7).
- Control odors by maintaining zero mg/L DO and a heavy scum blanket.
- Keep records of flow, HRT, *p*H, BOD_5 and TSS.
- Include information on volatile acids, scum and sludge depth.

9.4.1.3 On-site Attendance

Maintaining an aerobic pond in good condition requires full-time operator attention. These activities should be performed on a daily basis, on a regular schedule and as needed:

- Maintain mechanical equipment
- Keep pipelines, diversion boxes and screens clean
- Collecting samples
- Run lab tests
- Perform housekeeping

9.4.2 Operation and Maintenance Goals for Facultative Ponds

9.4.2.1 Pond Effluent Compliance Conditions

The pond effluent should:

- Meet the NPDES or other regulatory permit levels for BOD_5 and TSS for continuous flow systems.
- Discharge when the effluent has the best quality and will least affect the receiving stream.
- Have a deep green sparkling color in the primary pond.
- Have high DOs in the secondary or final cells.

9.4.2.2 Wave Action

The surface water should have wave action when wind is blowing. The absence of good wave action may indicate anaerobic conditions or an oily surface.

9.4.2.3 Maintenance

General maintenance guidelines:

- To maintain wave action, a pond should be free of weeds in the water or tall weeds on the banks.
- Dikes should be well seeded with grasses above the water line. Grass should be mowed regularly to prevent soil erosion and insect problems.
- Riprap, broken concrete rubble or a poured concrete erosion pad should be placed at the water's edge to prevent erosion of dikes.
- Inlet and outlet structures should be cleaned regularly to remove any floating debris, caked scum, or other trash that might produce odors or be unsightly.
- Mechanical equipment should be maintained according to a regular schedule. Maintenance records should be kept and be readily accessible.
- All pond operations should be listed on a posted schedule. The plant records should include weather data and basic test results such as flow, *p*H, DO, BOD_5, TSS and chlorine residuals.

9.4.3 Operation and Maintenance Goals for Aerated Ponds

In the past, many of the facultative pond systems were converted to aerated pond systems by the addition of mechanical aeration. This allowed the hydraulic and organic loading rates to increase, but caused many operational problems. Cell depths remain shallow (1.2 - 1.8 m and HRTs were typically lower than those used in facultative ponds, but greater than those used in aerated pond systems. Some problems were caused by the surface aerators mixing too deeply and scouring the bottom of the cells, compromising the integrity of the liners. The shallow depths and longer HRTs promoted algal overgrowth, making it difficult to meet discharge permit requirements in the summer months.

Aerated ponds require the same daily inspections and maintenance as any other treatment ponds. In addition, special attention must be paid to the aeration equipment. The following are minimal guidelines:

- Maintain a minimum of 1 mg/L DO throughout the pond at heaviest loading periods.
- Run the system so that surface mechanical aerators produce good turbulence and a light amount of froth.
- Monitor DO at aerated cell outlet daily.
- Keep large objects out of the pond to prevent damage to the aerator.

For diffused air systems that use a blower and pipelines to diffuse air over entire bottom of pond:

- Check the blower daily.
- Visually inspect the aeration pattern for dead spots or dead lines.
- Check for ruptures and repair them if necessary to maintain even distribution of air.
- Measure DO at several locations in the pond weekly and adjust air to maintain even distribution.

Periodic maintenance, such as lubrication, adjustment and replacement of parts, must be performed on a regular basis. A checklist of maintenance tasks and frequency, taken from the manufacturer's instructions bulletins, should be available and activities relating to maintenance recorded in a log book.

9.4.4. Pond System Checklist

A checklist is a handy tool for the operator to schedule activities. Most of the items are visual observations or maintenance needs that take little time if performed according to a regular schedule. Over time, the operator will develop ways to combine some of the duties. In many installations that are overseen regularly by a conscientious operator, the scheduled tasks can be accomplished in one to two hours a day, allowing the balance of the time to be used to complete laboratory work and other duties.

Table 9-4 is a sample O&M checklist for pond operation. Although it is not a complete list of everything the operator should be observing, it will serve as a guide for setting up a regular schedule and as a daily reminder. The schedule will help the operator organize work in a step-by-step fashion, which will also help operators coming on in relief during an emergency or new personnel who are not familiar with the system. The design engineer should develop a checklist for the system that is included in the O&M manual.

Table 9-4 Example Operation and Maintenance Checklist.

Operation and Maintenance	Frequency						
	Daily	Weekly	Monthly	3 mos.	6 mos.	Yearly	As Needed
Plant Survey							
Drive around perimeters of ponds taking note of the following conditions:							
Any buildup of scum on pond surface and discharge outlet boxes		x					
Signs of burrowing animals	x						

Operation and Maintenance	Frequency					
	Daily	Weekly	Monthly	3 mos.	6 mos.	Yearly	As Needed
Anaerobic conditions: noted by odor and black color, floating sludge, large number of gas bubbles	x						
Water-grown weeds	x						
Evidence of dike erosion	x						
Dike leakage	x						
Fence damage	x						
Ice buildup in winter						x	x
Evidence of short-circuiting	x						
A review of the information obtained from the observations should be included in the next year's planning activities.							
Plan, schedule, and correct problems found. Use troubleshooting section of this manual for information.						x	
Pretreatment							
Clean inlet and screens, and properly dispose of trash.	x						
Check inlet flow meter and float well.	x						
If discharge is once or twice per year, the discharge permit may require observations of the following:							
Odor		x					
Aquatic plant coverage of pond		x					
Pond depth		x					
Dike condition		x					
Ice cover		x					

Operation and Maintenance	Frequency						
	Daily	Weekly	Monthly	3 mos.	6 mos.	Yearly	As Needed
Flow (influent)	x						
Rainfall (or snowfall)	x						
Note: *Each state has requirements for data collected prior to and during discharge that are defined in the pond system discharge permit.*							
If discharge is continuous, the discharge permit may require the following information:							
Weather	x						
Flow	x						
Condition of all cells	x						
Depth of all cells	x						
Pond effluent:	x						
DO and *p*H grab sample	x						
Cl^- residual	x						
BOD_5 and TSS run on composited sampled							x
Microbial tests							x
kg (lb) of Cl^- used and remaining	x						
Other tests and frequency information will be defined in the individual permit.							
Mechanical Equipment							
Check mechanical equipment and perform scheduled preventive maintenance on the following pieces of equipment according to the manufacturer's recommendations:							
Pump stations:							
Remove debris	x						
Check pump operation	x						
Run emergency generator		x					
Log running times	x						

Operation and Maintenance	Frequency						
	Daily	Weekly	Monthly	3 mos.	6 mos.	Yearly	As Needed
Clean floats, bubblers, or other control devices		x					
Lubricate							x
Comminuting devices:							
Check cutters		x					
Lubricate							x
Aerators:							
Log running time	x						
Check amperage		x					
Chlorinators:							
Check feed rate	x						
Change cylinders							x
Flow measuring devices:							
Check and clean floats, etc.	x						
Verify accuracy		x					
Valves and gates:							
Check to see if set correctly	x						
Open and close to be sure they are operational		x					

9.4.5 Flexible Design to Improve Operation

9.4.5.1 Flow Regulation

Flow regulation is one of the most helpful operational tools. Without the flexibility to move water around where it is needed, the operator would be severely limited in his or her ability to troubleshoot and solve pond system problems. The following sections enumerate these options.

9.4.5.1.1 Single Cell Ponds

The only flexibility an operator has with a single cell pond is depth control. The water level may have to be varied seasonally or to control weeds and mosquitoes.

9.4.5.1.2 Multiple Cell Ponds

Multiple cell pond systems may be operated to optimize a number of different parameters. They may be operated so as to:

- Hold wastewater in the primary cell, especially during seasonal discharge operation.
- Move water from cell to cell to correct an O_2 deficiency problem.
- Control liquid depth to eliminate weeds or mosquitoes.
- Isolate a cell that has become anaerobic or to hold a toxic waste.
- Take advantage of both series and parallel operation to regulate loading.
- Rest a cell temporarily for recovery.
- Recirculate water from the last cell to the first cell, at a minimum. This allows the operator to increase the DO and to seed the first cell with algae. Remove an individual cell from operation which varies the HRT of the system, particularly in summer.

9.4.5.2 Baffles and Screens

Screens, often custom-made, are used around pond surface outlets to keep windblown weed and surface trash from entering a pipe.

Baffles may consist of pilings (5 by 24 cm) driven into the pond bottom. They are commonly used for a large variety of purposes, for example:

- Direct the flow of water, especially around inlets.
- Reduce or eliminate short circuiting.
- Allow selection of depth for pond draw-off and to keep surface scum and trash from entering.
- Provide a quiet zone ahead of a flow measuring device.
- Reduce the force of a pump discharge.

9.4.5.3 Inlet and Outlet Design

Submerged outlets should be used to prevent the discharge and/or transfer of floating material between ponds.

Variable depth draw-off is especially useful in parts of the country where algal overgrowth is a problem. The effluent should be able to be drawn from any depth in the pond cell, giving the operator the choice of transferring or discharging the best quality water. Variable depth discharge works best with surface mechanical aeration. Low discharge approach velocities are required to minimize the area of influence adjacent to the discharge structure.

There are numerous discharge structure designs that allow the operator to draw effluent in the area under the algal layer while staying 0.6 - 0.9 m above the benthic sludge layer. At a minimum, the pond should be designed with three draw-off points: the first below the algae layer, the second in approximately the mid-depth, and the third above the bottom of the pond.

The use of properly designed inlet and outlet manifolds may aid in the distribution and collection of wastewater flows and minimize short-circuiting.

Transverse perforated collection pipes may reduce approach velocities, increase the discharge collection area of influence and also help minimize short circuiting.

Having multiple inlets and outlets in the system design gives the operator greater flexibility to match the loading and discharge of a pond system more closely to environmental conditions.

9.4.5.4 Dike Erosion

Dike erosion from wave action can be prevented by using riprap in the form of rocks 8 - 48 cm laid along the water's edge. One unusual method employed was to sink 5 by 15 cm uprights into the pond floor extending above the water surface to dissipate the waves. In another case, the pond operator filled bags with a dry mix of sand, gravel and cement. These were laid side by side and stacked to form a system of riprap protection. Riprap should extend 0.3 m above and below extreme operating levels. Other forms of riprap or bank stabilization include cribbing (snow fence) laid on the bank and reed canary grass. Canary grass is effective in ponds that are deep, have steep slopes and a stable water level. If sod is used it should be at least 7.5 cm (3 in) square and placed not more than 1 m apart.

9.4.6 Pond Cleaning

When it becomes necessary to clean a pond, the operator should first contact the regulatory agency to find out about any special requirements. In most cases the operator and/or consulting engineer are required to develop a plan outlining the method of sludge removal, steps to be taken to ensure pond liner integrity, test of the sludge to be removed for volatile solids and metals, and describe the plan for ultimate disposal of the sludge. A separate permit may be required when land application of the sludge is selected for ultimate disposal. The regulatory agency may require site geological information (e.g., type of soil, slope of ground, depth to groundwater), type of crop grown and agronomic uptake rate of metals and nutrients of that crop, sludge testing, method of application, method of solids incorporation into the soil, irrigation practices, runoff control and disposal, and if applicable, vector control and monitoring requirements.

The most common method of sludge removal is to employ some type of sludge pumping equipment. Care should be taken to maintain the integrity of the pond liner. Damage to a pond liner may require repair or replacement, or at a minimum, increased monitoring and testing to ensure the pond is not adversely affecting surface water, groundwater and/or public health.

9.4.7 Procedures for Startup

9.4.7.1 Primary Cell

Spring or early summer is the best time for startup to avoid low temperatures and possible freezing. Fill primary cell(s) with water from a river or municipal system, if available, to the 0.6 m level. Begin to add the wastewater, keeping the *p*H above 9.5 and checking the DO daily (see Appendix G.) Algal blooms should appear in 7-14 d.

A good biological community will be established in about 60 d or less. The color will be a definite green, not blue or yellow-green. This procedure tends to avoid odorous anaerobic conditions and weed growth during the start-up phase.

If is it necessary to start in late fall or winter, the water level should be brought to 0.75 - 1 m and not discharged until late spring.

9.4.7.2 Filling Successive Ponds

- Begin filling when the water level in the first pond reaches a depth of 1 m.
- Add fresh water to a depth of 0.6 m.
- Begin adding water from previous pond observing the following:
  - Use top draw-off to achieve good transfer. Do not draw off from a level below the bottom 45 cm.
  - Do not allow the water depth in the previous pond(s) to fall below 1 m.
  - Equalize water depths in all ponds. This should be done in the following manner: Hold the discharge until all ponds are filled.

    Use effluent box with gates or valves to allow pumping of the effluent to any pond in the system if it is designed with this capability.

    Recycle the effluent continuously to the ponds with low water levels.

    Repeat the operation using 15 cm increments until ponds are at their operating depth.

Finally, start continuous or intermittent discharge, according to the system design.

9.4.8 Discharge Control Program for Seasonal Discharges

9.4.8.1 Preparation

- Make a note of conditions in the stream to receive discharge.
- Estimate duration of discharge and expected volume.
- Obtain state regulatory agency approval.
- Isolate cell to be discharged. Allow it to rest for at least one month, if possible.
- Arrange for daily sample analysis of BOD_5, TSS, *p*H, coliform and nutrients (if required).
- Plan other work so as to be able to devote full attention to control of discharge throughout the period.
- Sample contents of cell and analyze for DO; note and record turbidity, color and any unusual conditions.

9.4.8.2 Discharge Procedures

Ponds in a number of northern states are permitted to discharge effluent seasonally. Three or four weeks after ice break-up, the ponds generally return to normal operating conditions. Wastewater in the cells is tested and results are reported to the state. If the wastewater is of a quality suitable for discharging, the operator follows state guidelines for discharging. The NPDES permit contains information about the discharge quality.

The quality of the receiving stream is usually determined by the state water quality control agency as part of the discharge approval program. When discharge approval is obtained, proceed as follows:

- Begin the discharge program with the last cell in series.
- Draw off the discharge from the best level at a time when the discharge is acceptable.
- Stop the discharge when ponds are upset.

- Follow testing procedures outlined by the state regulatory agency.

9.5 SAFETY AROUND PONDS

9.5.1 Public Health

Operators and others conducting activities around treatment ponds must proceed with caution and make safety and public health a priority. Treatment ponds must be utilized for their designed purpose only, not for public recreation. The relative amount of water surface of treatment ponds is insignificant compared to the many natural bodies of open water in most localities. In some areas, however, treatment ponds represent the only sizeable body of water and have been sources of attraction for recreation purposes. Incidents of boating, ice skating, waterfowl hunting and even swimming in ponds have been reported. Recreational use should be discouraged and safety practices encouraged for several important reasons. Even though the efficiency of bacterial removal as measured by the MPN method is very high, the possibility of contamination or infection from pathogenic organisms does exist when a person comes in contact with wastewater in a treatment pond.

People can drown in treatment ponds. Clay and synthetic liners used in sealing ponds become very sticky when water is added. Should a person fall into a pond, the presence of liners would make it extremely difficult to get out. To discourage use of the ponds for recreation, the entire area should be fenced and warning signs displayed.

Another factor to be considered is the presence of mosquitoes. In a well-maintained pond system, mosquitoes usually are not a nuisance. According to studies by the U.S. Public Health Service, the density of the mosquito population is directly proportional to the extent of weed growth in a pond. Where weed growth in the ponds and along the water line of the dikes is negligible and where wind action on the pond is not unduly restricted, the likelihood of mosquitoes breeding is low.

9.5.2 Personal Hygiene

In the interest of the health of those who work around wastewater treatment ponds and their families, this list of *Do's* and *Don't's* for personal hygiene is presented.

- Never eat or put anything into your mouth without first washing your hands.
- Refrain from smoking while working in manholes.
- Wear gloves when working on pumps or other parts of the operation where hands may become contaminated.
- Don't wear work coveralls or rubber boots in the car or at home.
- Always clean any equipment, such as safety belts, harnesses, face masks, or gloves after use.
- Keep fingernails cut short and clean.

9.5.3 Safety

9.5.3.1 Sewer Maintenance Safety Precautions

- Remove and replace heavy manhole covers carefully and only with the proper tools.
- Descend into any manhole slowly and cautiously.

- Use the buddy system; do not enter without a spotter present.

See Section 9.5.3.4 for information regarding noxious gases that may be found in sewers.

9.5.3.2 Pumping Station and Treatment Pond Safety Precautions

- Maintain a high level of housekeeping. Keep floors, walls and equipment free from dirt, grease and debris. Keep tools properly stored when not in use.
- Keep walkways clean and free of slippery substances. If ice forms on walks, apply salt or sand or cover with earth or ashes that can be removed later.
- Be especially cautious when working with an electrical distribution system and related facilities. Never work on electrical equipment and wire with wet hands or in wet clothes or shoes. Always wear appropriate safety gloves for electrical work. Never use a switchbox as anything other than a switchbox.
- Keep all personnel safety conscious by providing training at regular intervals. Have specific safety instructions posted in appropriate places. Such instructions should include information as to how to contact the nearest medical center and fire station, rescue techniques, resuscitation and first aid techniques.
- Make certain that a sufficient number of trained and experienced personnel with proper equipment are assigned and present whenever it is necessary to perform any hazardous work.
- Staff should have boating safety training. Life preservers must be used whenever personnel are in a boat on treatment ponds. At least two people should work together around the ponds because of the danger of drowning and other accidents. Safety training for a pond operator should include life saving skills, including the ability to swim at least 30 m in normal work clothing.
- Sufficient fire extinguishers (Underwriter's Laboratories Approved) should be placed in readily accessible locations.

9.5.3.3 Body Infection and Disease Safety Precautions

Treat all cuts, skin abrasions and similar injuries promptly. When working with wastewater, the smallest cut or scratch is potentially dangerous and should be cleaned and treated immediately with a 2 percent solution of tincture of iodine. In addition, personnel should:

- Receive medical attention for all injuries.
- Be given first aid training.
- Be inoculated for waterborne diseases, particularly typhoid and para-typhoid fever.
- Keep a record of all immunizations in an employee health record.
- Review records annually for necessary boosters and new immunizations.

In the laboratory, always use appropriate laboratory equipment and supplies, and avoid any contamination by mouth. Don't take laboratory glassware for personal use. Paper cups should be provided in laboratories for drinking purposes. Never prepare or eat food in a laboratory.

9.5.3.4 Noxious Gases, Explosive Mixtures and Oxygen Deficiency

The principal air hazards associated with wastewater treatment are accumulations of sewer gas and its mixture with other gases or air, which may cause death or injury through explosion or asphyxiation as a result of O_2 deficiency. The term "sewer gas" is generally applied to the mixture of gases in sewers and manholes containing high percentages of CO_2, varying amounts of CH_4, H_2, H_2S, and low percentages of O_2. Such mixtures sometimes accumulate in sewers and manholes where organic matter has been deposited and has undergone decomposition. The actual hazards from sewer gas are the result of the explosive amount of CH_4, H_2S or in O_2 deficiency. Hydrogen sulfide is toxic at very low concentrations and a person's sensitivity to the odor is quickly deadened.

Chlorine gas, which is irritating to the eyes, respiratory tract and other mucous membranes, may settle in low-lying, still areas. The gas forms an acid in the presence of moisture. The gas may leak from cylinders and feed lines and diffuse and settle into these places.

9.5.3.5 Safety Equipment

A wastewater facility should have the following types of safety equipment:

- Detection equipment (for gases and O_2 deficiencies)
- Respirators (self-contained SCBA packs for O_2 deficiencies)
- Safety harnesses, lines and hoists
- Proper protective clothing, footwear and head gear
- Ventilation equipment
- Non sparking tools
- Communications equipment
- Portable air blower
- Explosion-proof lantern and other safe illumination
- Warning signs and barriers
- Emergency first aid kits
- Proper fire extinguishers
- Eye wash and shower stations in laboratory areas
- Safety goggles for work in laboratories and other dangerous areas

9.6 TROUBLESHOOTING

An operator or engineer must have a thorough understanding of the treatment process and tests required to diagnose problems affecting effluent quality (see Appendices E and F). This expertise must be brought to bear to achieve the highest level of process performance from a pond system and stay within the agency's budget. Prior to visiting the site, the following documents should be reviewed:

- Past inspection reports.
- The discharge permit.
- Discharge monitoring reports (DMRs) and plant performance records for a three-year period.
- Any noncompliance correspondence to compare to plant operation records and DMRs to look for trends.
- Plans and specifications to verify that they reflect current plant conditions.

- Current plant records to verify that it is operating within required parameters.
- Organic loading rates, surface loading rates and aeration capacity to determine whether the ponds are organically overloaded.
- O&M Manual to determine if the plant is being operated within the engineers' recommendations.

Once on site, the following items should be observed and reviewed:

- Plant appearance and maintenance, including weed control, dike vegetation, dike erosion and stability, fencing and out-building conditions.
- Individual cells: Note any floating material, such as grease, sludge, floating vegetation, or mats of blue-green algae; water depth; freeboard height; cell color; odor problems; septic conditions; sludge buildup.
- Influent and effluent flows for infiltration/inflow problems, influent septicity or odors, and unusually high effluent solids, and/or floating material.
- Plant influent and effluent parameters.
- Plant operation and maintenance and equipment records.
- Plant staffing for operations and maintenance.
- Safety equipment and procedures.
- Sampling locations, methods, frequency and weather records.

9.6.1 Common Causes of Pond Effluent Noncompliance

Pond effluent violations can be caused by organic overloading, short-circuiting, algal overgrowth conditions, sludge accumulation and nitrification, or partial nitrification. The following sections will describe some of the causes of effluent violation, troubleshooting tests and results, and present possible solutions to the problem.

9.6.1.1 Organic Overload

Organic overload is normally caused by influent organic shock loads or increased organic load with no corresponding increase in treatment plant capacity. This condition causes low dissolved O_2 concentrations (< 1.0 mg/L) and inhibits treatment. This can be verified by calculating the organic loading (BOD_5/d) and comparing it to design capacity. A DO test and DO profiles should be run at various times of the day to verify whether there is a continuous low DO condition.

The diagnostic troubleshooting tests will demonstrate high BOD_5, high $CBOD_5$, high soluble BOD_5, low DO, a low TSS to BOD_5 ratio and high NH_3. The immediate solution is to increase organic treatment capacity by increasing aeration. In the long term, a pretreatment program with collection system monitoring of those areas suspected of introducing high organic shock loads should be developed and implemented.

9.6.1.2 Short-circuiting

Short-circuiting normally occurs when untreated or partially treated wastewater does not have adequate detention time in the system for complete treatment. This can be caused by temperature stratification in the cells, poor inlet and/or outlet design, inadequate cell length-to - width ratio or cell shape, or poor mixing and improper aerator placement. Performing a DO profile test on an established grid system while recording both DO and temperatures in 0.3 m increments from the surface to the bottom should verify a short circuiting condition. The operator will note variations in DO and temperature indicating temperature stratification and or poor mixing. The diagnostic trouble shooting test will indicate high BOD_5, high soluble BOD_5, moderate TSS and high NH_3 levels. This condition should be verified with a microscopic examination of the effluent.

Possible solutions are relocating aerators, addition of directional aerators or mixers, adding baffles, recirculating to enhance mixing, and redesigning inlet and outlet structures to include manifolds or relocating structures.

9.6.1.3 Algal Overgrowth

Algal overgrowth is prevalent in the areas where there is a high number of sunny days during the year. This condition occurs predominantly in the spring and summer. Long detention times, shallow pond depths (1.2 - 1.8 m), abundant nutrients, warm water and sunshine promote algal growth. The diagnostic troubleshooting test results indicate high *p*H, high BOD_5, low $CBOD_5$, low soluble BOD_5, high TSS, a high TSS to BOD_5 ratio, low DO (early morning) and moderate-to-high NH_3 concentrations.

During the night, algae and aerobic bacteria will utilize O_2, potentially depleting the DO in the cells prior to sunrise. Lack of O_2 will cause incomplete treatment, possibly resulting in permit violation. A DO profile test run at sunrise will normally verify the lack of O_2 in the cells. A microscopic examination of the effluent and count of the algae will confirm the overgrowth condition.

Possible solutions include increasing the aerator running time at night. The operator may choose to reduce aerator running time during the day, allowing the algae to concentrate on the surface of the cells. The high concentration of algae at the surface will reduce sunlight penetration and may slow the algal growth rate. Drawing off the effluent from variable depths below the surface will also keep the algae in the cells, while allowing for the discharge of high quality water. The addition of floating covers will block the sunlight and, with the maintenance of adequate in-cell DO levels, produce a higher quality effluent. The addition of physical shade such as greenhouse fabric suspended above the surface of the cells, or chemicals such as Aquashade® or photo blue, used in accordance with EPA registry instructions (http://www.epa.gov/oppsrrd1/REDs/aquashade_red.pdf) may also prove effective in controlling algal growth.

9.6.1.4 Sludge Accumulation in Ponds

Sludge will accumulate in the bottom of pond cells over years of operation. Soluble organics are released from these benthic sludges and have the largest effect on ponds in the spring of the year. Diagnostic troubleshooting test results will indicate high $CBOD_5$, high soluble BOD_5, low to moderate TSS, a low TSS to BOD_5 ratio, low DO and high NH_3 concentrations. This condition can be verified with a microscopic examination. Increasing aerator running times may offer a temporary solution. Ultimately removal of the sludge from the bottom of the pond cells will be necessary. The operator must comply with all state and federal regulations and must take care to protect the pond liner during the process.

9.6.1.5 Nitrification or Partial Nitrification

Nitrification in ponds will occur under proper environmental conditions (in warmer water temperatures and adequate DO) and is most prevalent in late spring and summer. Complete nitrification would be indicated by low BOD_5, low $CBOD_5$, low to moderate TSS, moderate DO, low NH_3 and moderate NO_3^- levels.

Partial nitrification is common in ponds in late spring and summer when adequate DO levels are not maintained for complete treatment. Troubleshooting diagnostic tests will show high BOD_5, low $CBOD_5$, low soluble BOD_5, low to moderate TSS, a low TSS to BOD_5 ratio, low DO and moderate NH_3 concentrations.

Increased aeration (aerator running times) or, in some cases, increased aeration capacity may increase nitrification.

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APPENDIX A

State Design Criteria for Wastewater Ponds

APPENDIX A
STATE DESIGN CRITERIA FOR WASTEWATER STABILIZATION PONDS

Table A-1 Minimum Hydraulic Residence Time and Depth Requirements

	Minimum Hydraulic Residence Time (HRT)			Depth Requirements			
State	Controlled Discharge and Non-discharge	Facultative Flow-Through days	Aerated days	Facultative Cell ft	Controlled Discharge ft	Aerated Cell ft	Anaerobic Cell
Alabama							
Alaska	Each design evaluated on a case-by-case basis.						
Arizona	Stabilization alone does not meet state water quality requirements for sewage treatment facility. New facilities must meet "Best Available Demonstrated Control Technology" treatment standards. Exceptions require demonstrations unique site-specific characteristics and environmental factors. Economic hardship is not a listed criterion for exception to BADCT.						
Arkansas	Ten State	Second cell of two cell system two cell system must be designed at same loading rate as primary with min HRT of 30 days. Cells following primary of 3 or more cells will have combined HRT of 30 days. Final cell designed for settling.	t = HRT, d E = % BOD_5 removed k_1 = reaction rate, d^{-1} k_T = 0.12/d at 20 ^0C k_T = 0.06/d at 1 ^0C	Ten State	Ten State	Ten State	NA
California	All criteria controlled by Regional Board						
Colorado	NA	180	12-30 Polishing Pond 2-5 at avg flow	5	NA	8-20 Polishing pond 8-12 but as great as practical.	NA
Connecticut	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table
Delaware							
Florida	Case by Case Analysis						
Georgia	Ten State	Ten State	Ten State	Ten State	Ten State	Ten State	NA

NA = Not available

	Minimum Hydraulic Residence Time (HRT)			Depth Requirements			
State	**Controlled Discharge and Non-discharge**	**Facultative Flow-Through days**	**Aerated days**	**Facultative Cell ft**	**Controlled Discharge ft**	**Aerated Cell ft**	**Anaerobic Cell**
Hawaii							
Idaho	Must be considered with regard to environmental conditions.	Must be considered with regard to environmental conditions.	Must be considered with regard to environmental conditions.	Must be considered with regard to environmental conditions. Minimum operating depth is 2 ft.	Must be considered with regard to environmental conditions. Minimum operating depth is 2 ft.	Must be considered with regard to environmental conditions. Minimum operating depth is 2 ft.	Must be considered with regard to environmental conditions. Minimum operating depth is 2 ft.
Illinois	Ten State	Ten State	Ten State	Not less than 5 Min Operating 2	10 State	10-15	NA
Indiana	90	90	Ten State	5	5	Ten State	NA
Iowa	180 Design to be based on wettest 180 consecutive days	Not acceptable for secondary treatment	Partial Mix 180 Design to be based on wettest 30 consecutive days	Not acceptable for secondary treatment	Primary Cells Max 6 Secondary Max 8		NA
Kansas							
Kentucky	Ten State	Primary cell must have minimum hydaulic residence time of 90 days	Ten State Also based on organic loading rate of 150 lbs per acre-day	Ten State	Ten State	Ten State	NA
Louisiana							
Maine	Based on storage required.	Design standards in TR-16, Ten State Standards or other published literature accepted by DEP or EPA are to be considered. Recirculation required.	Design standards in TR-16, Ten State Standards or other published literature accepted by DEP or EPA are to be considered. Recirculation required.	Design standards in TR-16, Ten State Standards or other published literature accepted by DEP or EPA are to be considered. Recirculation required.	Design standards in TR-16, Ten State Standards or other published literature accepted by DEP or EPA are to be considered. Recirculation required.	Partial Mix Minimum 10 ft	Design standards in TR-16, Ten State Standards or other published literature accepted by DEP or EPA are to be considered. Recirculation required.
Maryland	NA	60	30	Minimum 3 ft Maximum 5 ft	NA	15	NA

NA = Not available

State	Minimum Hydraulic Residence Time (HRT)			Depth Requirements			
	Controlled Discharge and Non-discharge	Facultative Flow-Through days	Aerated days	Facultative Cell ft	Controlled Discharge ft	Aerated Cell ft	Anaerobic Cell
Massachusetts	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table
Michigan	Ten State	Ten State	Ten State	Ten State	Ten State	Ten State	NA
Minnesota 54 pg document with details for all aspects of lagoon systems.	Ten State	180-210 180 d between two and a max. depth of 6 feet	Varies with design $\boxed{t = \frac{E}{2.3k_1(100-E)}}$ Eq. used for each cell. Total HRT must equal min of 25 d to meet BOD of 25 mg/L. Total HRT must equal min of 35 d to meet BOD of 15 mg/L. t = HRT, d E = % BOD_5 removed k_1 = reaction rate, d^{-1} k_T = 0.12/d at 20 ^{0}C k_T = 0.06/d at 0.5 ^{0}C	Max 6	NA	10-15	NA
Mississippi	Hydrograph controlled release minimum storage 90 days	30 days at 4 ft. operating depth.	Partial Mix 18 days plus settling area of 1 day. Complete Mix not specified.	6	15 Max for storage cell 20	10-15	20 Day HRT 8-20 Water Depth
Missouri							
Montana	Primary 40-80 Based on volume between 2 ft and maximum depth. Secondary Based on volume between 1 ft and maximum depth.	Primary 40-80 Based on volume between 2 ft and maximum depth. Secondary Based on volume between 1 ft and maximum depth.	Partial Mix Min 20 days under aeration t = HRT, d E = % BOD_5 removed k_1 = reaction rate, d^{-1} k_T = 0.12/d at 20 ^{0}C	Secondary Cells 8	Secondary Cells 8	10-15	NA

NA = Not available

	Minimum Hydraulic Residence Time (HRT)			Depth Requirements			
State	Controlled Discharge and Non-discharge	Facultative Flow-Through days	Aerated days	Facultative Cell ft	Controlled Discharge ft	Aerated Cell ft	Anaerobic Cell
			k_T = 0.06/d at 0 ^{0}C				
Nebraska	Discharge limited to once or twice per year. Half or all of average flow must be stored.	Primary cells must have minimum HRT of 60 days and entire volume must have a minimum of HRT of 120 days. Area of initial cell should not be greater than approx. 2/3 of the total area.	Partial Mix BOD removal 30-60 Ammonia-N 80-90 TKN 100-120 Complete Mix 1.5 to 2.0 for 85% BOD removal. HRT of 7 to 10 for complete nitrification.	Primary 4-6 Final cells max. 8.	NA	Partial Mix 7-14 Complete Mix 10-20	As deep as possible. Not less than 10-15 ft.
Nevada	NA	$\frac{C_e}{C_0} = e^{-k_p t}$ t = HRT $k_P = k_{P20}(1.09)^{(T-20)}$ e = base natural log C_e = eff BOD C_0 = inf BOD k_{P20}= varies with load 0.045to 0.096 d^{-1}	$\frac{C_n}{C_0} = \frac{1}{\left[1+\left(\frac{k_{PM}t}{n}\right)\right]^n}$ t = HRT $k_{PMT} = k_{PM20}(1.036)^{(T-20)}$ C_e = eff BOD C_0 = inf BOD n = no. cells in series Same equation for partial mix and complete mix. k_{PM20}= 0.276 d^{-1} k_{CM20} = 2.5 d^{-1}	4-10	NA	6-20	NA
New Hampshire	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table
New Jersey	Each design evaluated on a case-by-case basis.						
New Mexico							
New York	Ten State and TR-16	Ten State and TR-16	Ten State and TR-16	Ten State and TR-16	Ten State and TR-16	10 State and TR-16	NA
North Carolina							
North Dakota	NA	Primary Cells 180 One-half of total	Can be reduced from 180 days with addition of	5 Min operating 2	NA	NA	NA

State	Minimum Hydraulic Residence Time (HRT) Controlled Discharge and Non-discharge	Facultative Flow-Through days	Aerated days	Depth Requirements Facultative Cell ft	Controlled Discharge ft	Aerated Cell ft	Anaerobic Cell
		surface area. 180 days based on total hydraulic loading.	aeration.				
Ohio	Based on calculated loading rates. Ten States	Based on calculated loading rates. Ten States	NA	Max. 7 Min. Operating 1.5	Max. 7 Min. Operating 1.5	Ten State	NA
Oklahoma							
Oregon	NA	NA	NA	3 to 5	NA	8-10	NA
Pennsylvania	90 days between 2 foot and max. operating depth. Mean oprating depth is max. operating depth plus minimum divided by two.	90-120	$\boxed{t = \frac{E}{2.3k_1(100-E)}}$ t = HRT, d E = % BOD_5 removed k_1 = reaction rate, d^{-1} k_T = 0.20/d at 20 ^{0}C k_T = 0.06/d at 0 ^{0}C	Primary cells 6 Secondary ponds depth 8 ft.	Primary cells 6	10-15	NA
Rhode Island	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table
South Carolina	No Specific Criteria. Evaluated on a case by case.						
South Dakota	Surface Area for total retention, A=I/WL. Summr or yr around, A = I/(H+WL) Winter months,	Primary Cell should be approx. 50-60% of total surface area. 180 d between two and a max. depth of 5 feet.	$\boxed{t = \frac{E}{2.3k_1(100-E)}}$ t = HRT, d E = % BOD_5 removed k_1 = reaction rate, d^{-1}	5	5	Max. 20 Min. 10	NA

NA = Not available

State	Minimum Hydraulic Residence Time (HRT)			Depth Requirements			
	Controlled Discharge and Non-discharge	Facultative Flow-Through days	Aerated days	Facultative Cell ft	Controlled Discharge ft	Aerated Cell ft	Anaerobic Cell
	A = I/(H+S-P) A = surface area, ac I = inflow, ac-ft WL = net H_2O loss S = seepage, ft H = depth > 2 ft P = precip., ft	First secondry cell max depth is 6. Other following cells may have depth 8.	k_T = 0.20/d at 20 ^{0}C k_T = 0.08/d at 0 ^{0}C $k_1 = k_{T20}(1.47)^{(T-20)}$ t = 5-10 d Warm t = 8-20 d Cold Settling Pond 5-7				
Tennessee	NA	Not specified.	$\boxed{\frac{C_e}{C_0} = \frac{1}{1 + 2.3(k_1 t)}}$ t = HRT, d C_e = eff BOD_5, mg/L C_0 = inf BOD_5, mg/L k_1 = reaction rate, d^{-1} k_1 = 1.097 @ 20 ^{0}C for complete mix. k_1 = 0.12 @ 20 ^{0}C for partial mix. $k_T = k_{20}(1.036)^{(T-20)}$	Primary 6 Greater depths considered for polishing and last ponds in series.	NA	Not less than 7	NA
Texas	NA	Based on organic loading rate	HRT in combined aerated lagoon and secondary pond system shall be a minimum of 21 days. Secondary ponds BOD_5 removal calculated by $\boxed{E = \frac{1}{1 + K(V/Q)}}$ Where: E = efficiency of CM without recycle. K = removal rate constant K = 0.5 day^{-1} V = volume, MG Q = flow rate, mgd Applies to Partial Mix and complete aerated cells.	Approx. 25 % of Inlet portion shall have a 10-12 ft depth for sludge storage and anaerobic treatment. Remainer of pond 5 to 8 ft.	NA	Secondary aerated ponds 3-5 ft	NA
Utah	NA	Exclusive of sludge build-up, 120 on winter flow at max operating depth,or 60 on summer flow and peak month I/I. HRT shall not be less than 150 at mean operating depth without chlorination. To meet bact.standards, at least 5 cells required.	$\boxed{E = \frac{1}{[1 + (2.3 k_1 t)]}}$ E = frac BOD_5 remaining t = HRT, d k_1 = reaction rate, d^{-1} k_1 = 0.12/d @ 20 ^{0}C k_1 = 0.06/d @ 1 ^{0}C 30 minimum	Primary 6 Greater depth if aeration or mixing is incorporated. Min operating depth 3 Min of 18 in for sludge.	NA	10-15	NA
Vermont	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table

NA = Not available

State	Minimum Hydraulic Residence Time (HRT)			Depth Requirements			
	Controlled Discharge and Non-discharge	Facultative Flow-Through days	Aerated days	Facultative Cell ft	Controlled Discharge ft	Aerated Cell ft	Anaerobic Cell
Virginia	NA	45 based on 4-ft operational level. Sludge storage based 20-year design life.	NA	Min operating depth 2 Max operating depth 5 excluding sludge storage.	NA	NA	NA
Washington	NA	NA	NA	NA	NA	NA	NA
West Virginia							
Wisconsin	Ten State	150	$t = \frac{E}{K(100-E)}$ t = HRT E = BOD removal, % K = reaction coef. Base e K = 0.5 at 20 ^{0}C $K_T = K_{20}(1.07)^{(T-20)}$ Min settling = 6	Max 6 Min Operting 2	Ten State	15 Minimum Operating 6	NA
Wyoming	NA	180	Primary Cell Complete Mix Not < 1.5 Partial Mix Not < 7 Secondary cells shall increase overall HRT to 30	6	NA	4-15	NA
Ten-State Standards 1997 Edition	At least180 d between 2' depth and max depth	90-120	$t = \frac{E}{2.3k_1(100-E)}$ t = HRT, d E = % BOD_5 removed k_1 = reaction rate, d^{-1} k_T = 0.12/d at 20 ^{0}C k_T = 0.06/d at 1 ^{0}C	Max 6' Primary Min 2' Greater Depths allowed in subsequent cells	Max 6' Primary Min 2' Greater Depths allowed in subsequent cells	10-15	NA
TR-16 Guides for Design of Wastewater Treatment Works 1998 Edition	180 Between 2-foot and maximum operating depth.	90-120	**Partial Mix** $t = \frac{E}{k_e(100-E)}$ t = HRT, d E = % BOD removed k_e = reaction rate, base e, d^{-1} k_T = 0.28/d at 20 ^{0}C k_T = 0.14/d at 10 ^{0}C For three cell facility suggested	3 ft minimum oprating depth. 5 ft maximum	3 ft minimum oprating depth. 5 ft maximum	**Partial Mix** 10-20	NA

NA = Not available

	Minimum Hydraulic Residence Time (HRT)			Depth Requirements			
State	**Controlled Discharge and Non-discharge**	**Facultative Flow-Through days**	**Aerated days**	**Facultative Cell ft**	**Controlled Discharge ft**	**Aerated Cell ft**	**Anaerobic Cell**
			k_e at 10^0C First cell - 0.14 Second cell - 0.06 Third cell - 0.02 **Complete Mix** 7-20				

NA = Not available

Table A-2. Sealing, Point of Discharge, N-Removed, P. Removal, Drawoff, Multi-level Required, Comments

State	Pond Bottom Sealing	Point of Discharge		Nitrogen Removal Required	Phosphorus Removal Required	Multi-level Drawoff Required	Comments
		Primary	Aerated				
Alabama							
Alaska							
Arizona							
Arkansas	Ten State	Ten State	Ten State	Ten State	Ten State	NA	
California							
Colorado	Required seepage must not exceed 1/32 in/d. If not obtained in natural soil, must use native clays, soil cement, asphalt or synthetic liners.	NA	NA	Where applicable	No	Required	

NA = Not applicable

State	Pond Bottom Sealing	Point of Discharge		Nitrogen Removal Required	Phosphorus Removal Required	Multi-level Drawoff Required	Comments
		Primary	Aerated				
Connecticut	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table
Delaware							
Florida							
Georgia	Ten State	Ten State	Ten State	Ten State	NA	Ten State	
Hawaii							
Idaho	Required Maximum seepage rate must not exceed 500 gallons per acre per day. Sensitive aquifers or near TMDL streams may require considerably lower seepage rate. After 4/15/07, rate shall be no more than 0.125 inches/day. Testing required after five years of operation.	NA	NA	Where applicable	Where applicable	NA	Controlled discharge lagoons allowed.
Minnesota 54 pg document with details for all aspects of lagoon systems.	Required	Primary Cells Influent Line terminates at midpoint of width and approx. 2/3 length from outlet	Multiple inlets required	Ten State	NA	Required	
Mississippi	Required Water loss shall not exceed 500 gpd/ac at head equal to max operating depth.	Primary cells terminate near center of cell.	Distribute flow and load in mixing zone.	NA	NA	Recommended	

NA = Not applicable

State	Pond Bottom Sealing	Point of Discharge		Nitrogen Removal Required	Phosphorus Removal Required	Multi-level Drawoff Required	Comments
		Primary	Aerated				
Missouri							
Montana	Required Max seepage 6 in/year	Midpoint of width, at approx. 10 ft from toe of dike and as far as possible from outlet structure.	Distribute load within mixing zone.	NA	NA	Must Consider	Total Retention Ponds 1 primary, 15-35 lbs/ac-d, max depth 6 ft, t = 40-80 d, 1 secondary, max depth 8 ft,
Nebraska	Required Maximum seepage rate must not exceed 1/8 in./d	Inlets to regular shaped cells terminate at center of cell. Rectangular cells inlets terminate at approx. one-third the length from upstream end of cell. Cells without outlet discharge at center of cell. Multiple inlets should be considered for large cells.	NA	See HRT	NA	Recommended	Complete retention lagoons allowed. A minimum of two cells must be provided with at least one pond having capacity to assure adequate depth. Lemna ponds considered for final pond.
Nevada	Required	Multiple inlets and outlets recommended	Multiple inlets and outlets recommended	Where applicable	NA	Recommended	

NA = Not applicable

State	Pond Bottom Sealing	Point of Discharge		Nitrogen Removal Required	Phosphorus Removal Required	Multi-level Drawoff Required	Comments
		Primary	Aerated				
New Hampshire	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table
New Jersey							
New Mexico							
New York	Required	Multiple inlets and outlets encouraged	Multiple inlets and outlets encouraged	NA	NA	Multiple outlets encouraged	Hydrograph control release lagoons permissible
North Carolina							
North Dakota	Required compacted clay, bentonite, or other approved material.	Primary Cells Essentially center of cell.	NA	NA	NA	NA	L:W, 3:1 Facultative

NA = Not applicable

State	Pond Bottom Sealing	Point of Discharge		Nitrogen Removal Required	Phosphorus Removal Required	Multi-level Drawoff Required	Comments
		Primary	Aerated				
Ohio	Ten State	Ten State	Ten State	Ten State	Ten State	Required	
Oklahoma							
Oregon	Average seepage rate less than 1/8 per day, corrected for evaporation and precipitation.	Primary cells inlets located near center of lagoon. Secondary cells inlets located at or near shoreline.	NA	NA	NA	Outlet provide for surface or subsurface withdrawals Surface skimming baffles shall be provided ahead of surface overflow structures.	Aerobic ponds 12 to 18 in. deep. Algae production main function.
Pennsylvania	Required On-site soils, bentonite, or other synthetic liners. Coefficient of permeability of sides and bottom will not exceed 1 x 10^{7} centimeters per second. Flexible membrane liners shall have a minimum thickness of 0.030 in.	At mid-point of width and at approximately two-thirds of length away from outlet structure. Multi-influent discharge points for primary cells 20 acres or larger	Distribute load within mixing zone.			Recommended for deep ponds where stratification may occur. A minimum of three discharge points are required.	Rectangular ponds with L:W 3:1 most desirable.
Rhode Island	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table
South Carolina							
South Dakota	Required Seepage rate for primary cell shall not exceed 1/16 in/d. Allowable seepage 1/8 in/d for cells in series following	Influent line should terminate at approx. 1/3 of length upstream end of cell. At approx. mid-point in cells without outlet.	To active mixing	NA	NA	Recomended	

NA = Not applicable

State	Pond Bottom Sealing	Point of Discharge		Nitrogen Removal Required	Phosphorus Removal Required	Multi-level Drawoff Required	Comments
		Primary	Aerated				
	primary.						
Tennessee	Required Earth liners, bentonite, synthetic membrane liners. Seepage rate shall not be greater than 1/4 in/d.	NA	NA	NA	NA	Multiple inlets for ponds larger than 10 ac.	Hydrograph controlled release lagoons allowed. Recirculation should be considered.
Texas	Required Clay soils meeting certain specifications are allowable. Membrane lining minimum thichness 20 mils	NA	NA	NA	NA	Required multiple inlets and outlets with baffling.	L:W of ponds 3:1 or 4:1
Utah	Required Earth liners, bentonite, synthetic membrane liners. Seepage rate shall not be greater than 1.0 x 10^{6} cm/sec.	In center of round or square cell or at third point farthest from outlet structure in rectangular cell.	At point where load is distributed within mixing zone. Multiple inlets considered in diffused air system.	NA	NA	Multiple inlets to primary cell of 20 ac	Total containment lagoons allowed. Same requirements for facultative apply with exception of discharge.

NA = Not applicable

State	Pond Bottom Sealing	Point of Discharge		Nitrogen Removal Required	Phosphorus Removal Required	Multi-level Drawoff Required	Comments
		Primary	Aerated				
Vermont	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table	TR-16 See end of table
Virginia	Required Natural soil, enhanced soil (bentonite, cement, etc., synthetic materials. Seepage rate <3 cm/yr.	Round or square ponds acceptable, but rectangular with L:W up to 10:1 most desirable. Influent and effluent shall be located as far apart as possible along flow path.	NA	NA	NA	Withdrawal points located at 0.75 ft to 2 ft below water irrespective of pond depth. Lowest draw-off shall be 12 in. off bottom.	
Washington	Required Double liner with leak detection required, and single liner with leak detection required.	NA	NA	Aeration must be provided for nitrification.	NA	NA	Total containment lagoons allowed. Design case is "wet" year (1 year in 10 recurrence interval).
West Virginia							
Wisconsin	Required Natural soil, enhanced soil (bentonite, cement, etc.,) synthetic materials. Seepage rate 1000 gal/ac-d	Round, square or rectangular allowed. Length not to exceed 3 times width. Circular lagoons discharge to center. Rectangular or square discharge to first one third of lagoon length.	NA	NA	NA	Required Multi-valved drawoff lines.	

NA = Not applicable

State	Pond Bottom Sealing	Point of Discharge		Nitrogen Removal Required	Phosphorus Removal Required	Multi-level Drawoff Required	Comments
		Primary	Aerated				
Wyoming	Must guarantee no threat to groundwater. Permeability of 10^{-7} cm/sec or less required without guarantee.	Fac Primary Cell Inlet shall terminate from outlet at least equal to greater than 2/3 the longest dimension.	Aerated inlet shall terminate in mixing zone.	Facultative t = 180 days Aerated t = 160 days	Chemical treatment required.	Required in final cell. At least one located at two-foot level.	Total containment lagoons allowed. BOD_5 loading shall not exceed 14 lb/ac-d. Rectangular cells shall have a maximum L:W of 5:1.
Ten-State Standards 1997 Edition	Required soils, bentonite or synthetic liners	Primary Cells Influent line terminates at midpoint of width and approx. 2/3 length from outlet.	Distribute load within mixing zone.	NA	NA	NA	NA
TR-16 Guides for Design of Wastewater Treatment Works 1998 Edition	Required Soils, bentonite or Synthetic Liners Leakage should be less than 500 gpd/ac. Sealed so that seepage loss is as low as possible.	NA	Distribute load within mixing zone.	NA	NA	Should be provided. In deeper ponds minimum of three withdrawal pipes at different elevations should be installed.	Screening and/or comminution should precede wastewater treatment ponds. Hydrograph release allowed. Treated effluent should be recirculated to primary cell. Anaerobic lagoons typical HRT 20-50 days.

NA = Not applicable

APPENDIX B

Summary of Pond Characteristics

APPENDIX B
Summary of Pond Characteristics

	FACULTATIVE	AEROBIC			ANAEROBIC
		Partial Mix	***Complete Mix***	***High Performance***	
Description	Earthen impoundment less than 2.5m deep. O^{2-} saturated water at surface supports aerobic biodegradation. Aerobic and anaerobic degradation processes occur mid-depth. Bottom anaerobic water supports methanogenesis. Performance depends on O_2 from algae.	Earthen impoundment in which aeration (mechanical surface mixing or submerged diffusion) is used to meet O_2 needs. No solids suspended. Performance depends on aeration.	Earthen impoundment in which mechanical mixing introduces air for BOD removal and to suspend solids. Performance depends on aeration.	Dual-power, multi-cellular systems (DPMC) designed for maximum BOD conversion efficiency.	A deep earthen basin not mixed or aerated. The organic load exceeds any naturally occurring dissolved O_2. Degradation takes place anaerobically.
Common Modifications	***Controlled Discharge –*** during winter or peak algal growth periods in summer ***Hydrograph Controlled Release -*** *discharge* when conditions in the receiving stream are suitable. ***Plastic Curtains -*** used as baffles to divide lagoon into cells. ***Floating Plastic Grids -*** **s**upporting the Growth of plants to reduce algal growth.	***Plastic curtains*** - with floats, anchored to bottom dividing lagoons into multiple cells to improve hydraulic conditions. ***Submerged diffusers*** - suspended from flexible floating booms which move in a cyclic pattern during aeration activity. Treats a larger volume with	*High Performance BIOLAC™ Nitrogen Removal Nitrification and de-nitrification.*	None known at this time.	***Placement -*** in front of facultative lagoon as part of design or retrofitted to existing system.

		each aeration line. ***Effluent recirculation*** -within the system to enhance oxygen levels.			
	FACULTATIVE	**AEROBIC**			**ANAEROBIC**
		Partial Mix	***Complete Mix***	***High Performance***	
Performance	**BOD: to** <30mg/L 95% of the time. **TSS:** to <100 mg/L. **NH_3:** up to 90% removal in summer. **P:** up to 50% removal. **Pathogen and fecal coliform removal:** varies with temperature and detention time.	**BOD: to** <30 mg/L 95% of the time with settling at end of system. **TSS:** to < 60 mg/L. **NH_3:** nitrified during summer.	Not available.	**TSS: to** < 15mg/L. **NH_3:** 90%removal.	**BOD:** reduced by 60%; less in cold climates.
Costs	**See Ch. 8**	**See Ch. 8**	**See Ch. 8**	**See Ch. 8**	**See Ch. 8**

	FACULTATIVE	AEROBIC			ANAEROBIC
		Partial Mix	*Complete Mix*	*High Performance*	
Applicability	Raw municipal wastewater effluent from *Primary treatment* *Trickling filters* *Aerated ponds* *Anaerobic ponds* *Biodegradable industrial wastewater*	Municipal and industrial wastewaters of low to medium strength.	Municipal and industrial applications where space is limited. Raw, screened or primary settled municipal wastewater. Biodegradable industrial wastewaters.	Screened municipal and industrial wastewater in areas where space is limited.	Pretreatment of municipal and industrial wastewater with high organic loading.
Advantages	Removes BOD, TSS, bacteria and NH_3. Low energy requirements. Easy to operate.	Smaller plant footprint than facultative ponds. Discharge acceptable under all climatic conditions.	Small footprint. Discharge acceptable under all conditions. No ice formation in cold weather.	Small footprint Removes BOD, TSS and bacteria when used with a settling basin Effective at converting NH_3 to NO_3^-.	Treats high organic loadings. Produces methane for energy recovery. Produces less sludge than other processes. Low energy requirements.
Disadvantages	Higher sludge accumulation in cold climates; removal required. Mosquitoes, other insect vectors, and burrowing animals may be a problem. Odors can occur with spring and fall pond turnover. Larger footprint. Difficult to control or predict ammonia levels in winter.	Requires energy input. Not as effective at removing N and P as facultative ponds. Ice formation. Mosquitoes and other vectors. Sludge removal required. Routine maintenance and cleaning required to maintain design aeration rates.	Agitation must be sufficient to suspend all solids. High energy requirement for aeration and solids suspension. Increased solids disposal. Settling basin needed to facilitate solids separation.	Solids removal is greater than other options. High energy requirements. Relatively little experience with this type of system.	Large footprint. Odors. Long retention times. Process may not be effective in colder climates.

<table>
<tr><th rowspan="2"></th><th rowspan="2">FACULTATIVE</th><th colspan="3">AEROBIC</th><th rowspan="2">ANAEROBIC</th></tr>
<tr><th>Partial Mix</th><th>Complete Mix</th><th>High Performance</th></tr>
<tr>
<td>Design Considerations and Criteria</td>
<td>Systems with at least three cell in series are recommended.

Inlet and outlet structure design should maximize volume to avoid short circuiting.

<u>Typical criteria:</u>
Loading Rate
22-67kg BOD_5/ha-d.
Detention Time
25-180 d.
Depth 1.5m – 2.5m.
Surface Area 4-60 ha.

Average organic loading rate and detention time relative to ambient temperature:
<table>
<tr><th>T, ^oC</th><th>BOD, kg/ha/d</th><th>t_{det},d</th></tr>
<tr><td>>15</td><td>18-36</td><td>?</td></tr>
<tr><td>0-10</td><td>9-18</td><td>?</td></tr>
<tr><td><0</td><td>4.5-9</td><td>120-180</td></tr>
</table>
Maximum loading rate for the 1st cell in multi-cell systems relative to temperature:
<table>
<tr><th>T, ^oC</th><th>BOD, kg/ha/d</th></tr>
<tr><td>>15</td><td>40</td></tr>
<tr><td><0</td><td>16</td></tr>
</table>

Lining may be required.</td>
<td>1.8 - 6 m (6 to 20 ft), 3 m (10 ft).

Submerged diffusion 3.7 to 4 kg O_2/kW-hour (6 to 6.5 lbs O_2/hphour).

Mechanical surface aerators 1.5 to 2.1 kg O_2/kW-hour (2.5 to 3.5 lbs O_2/hp-hour).

System should have at least three lined cells in series depending on soil conditions.

Detention Times 10 – 30d [20 days most common].

The design of aerated lagoons for BOD removal is based on first-order kinetics and the complete mix hydraulics model. Even though the system is not completely mixed.

Ponds should be rectangular with a 3:1 or 4:1 length to width ratio</td>
<td>Three cell systems are recommended.

Agitation must be sufficient to suspend all solids.

Detention time: 1.5 < 3d.

The design for BOD removal is based on first-order kinetics and the complete mix hydraulics model.</td>
<td>$\geq$ 6W/m^3 needed for primary basin, 1.8W/m^3 for settling basin.

Detention Time <1.5d.

Not appropriate for $CBOD_5$<100mg/L</td>
<td>BOD loading rate: 0.04 -0.30 kg/m^3/d (2.5- 18.7 lb /10^3 ft^3/d).

Detention time: 1-50d.

Depth: 2.4-6m [Deeper is better.]

Surface area: 0.2-0.8 ha.

Minimum freeboard: 0.9m.

Liners recommended to prevent seepage.

Surface runoff should be diverted from lagoon surface.</td>
</tr>
</table>

	FACULTATIVE	AEROBIC			ANAEROBIC
		Partial Mix	*Complete Mix*		
Design Models and Equations [See Appendix C for example calculations]	**Areal Loading Rate Method –** Simple to use when impoundment will not be mixed. **Gloyna Model –** Assumes a BOD_5 removal efficiency of 89-90%. **Complete Mix Kinetics –** Marias and Shaw model (also assumes first order degradation kinetics). [NOTE: This model is not widely accepted as a complete mix kinetic model; assumptions have not proven to be valid for facultative ponds.] **Plug Flow** **Intermediate Flow (between complete mix and plug flow) –** Wehner-Wilhlem Equation accounts for both biodegradation kinetics and dispersion.	Use **complete mix kinetic** model.		Use **complete mix kinetics** model.	Design based on volumetric loading rate, water temperature and hydraulic detention time. See **Advanced Integrated Pond System** Design (Oswald 1999).

APPENDIX C

Design Examples

APPENDIX C

DESIGN EXAMPLES

FROM CHAPTER 3 DESIGN OF MUNICIPAL WASTEWATER POND SYSTEMS

Example 3-1. Design of Anaerobic Pond Based on Volume and Per Capita (Oswald, 1996)

Design Flow Rate =	947 m^3/d
Influent Ultimate BOD_5 (C_o) =	400 mg/L
Effluent Ultimate BOD_5 (C_e) =	50 mg/L
Sewered Population =	6000 people
Maximum Bottom Temperature in Local Waters =	20 °C
Temperature of Pond Water at Bottom of Pond =	10 °C

1. Calculate the BOD_5 Loading

BOD_5 Loading = Influent BOD_5 x Flow Rate/1000 378.8 kg/d

2. Design the Anaerobic Pond (Fermentation Pits)

Except for systems with flows less than 200m^3/day, always use two ponds so that one will be available for removing sludge when pond is filled. Surface area of anaerobic pond should be limited to 1000 m^2 and made as deep as possible to avoid turnover with oxygen intrusion. Minimum pit depth should be 4 m.

Number of Anaerobic Ponds in Parallel = minimum of two ponds = 2
BOD_5 Loading on Single Pond = 189.4 kg/d

First Size Pond on Basis of Load per Unit Volume

Load per Unit Volume (varies with temperature of water) 0.189 kg/m^3/d

Volume in One Pond =	1002.7 m^3
HRT in Ponds =	2.12 d
Pond Depth = minimum of four meters =	4 m
Pond Surface Area (assuming vertical walls) =	250.7 m^2
Maximum Pond Surface Area = 1000 m^2; No. of Ponds =	0.25
Round to Next Largest Number of Ponds =	1
Overflow Rate in Ponds = total surface area/total flow rate =	1.89 m/day

Overflow rates of less than 1.5 m/day should retain parasite eggs and other particles as small as 20 μ, which includes all but the smallest parasite eggs (ova). Size of pond should be increased to reduce overflow rate to 1.5 m/day.

Check Pond Volume per Capita

Total Volume in Ponds = total BOD_5 loading/loading rate	2005 m^3
Pond Volume/Capita = total volume/population	0.33 m^3/cap

Pond Volume/Capita should be greater than 0.0566 m^3/person as used in conventional separate digesters. When pit volume/capita exceeds that amount, fermentation can go to completion with only grit and refractory organics left to accumulate.

Example 3-2. Anaerobic Pond Design Based on Volumetric Loading or Detention Times (Crites et al., 2006)

Based on Volumetric Loading or HRT(WHO, 1987).
Design Based on Volumetric Loading, HRT and Climate Temperature.

Temperature, °C	Detention Time, day	BOD_5 Reduction, %
10	5	0 - 10
10 - 15	4 - 5	30 - 40
15 - 20	2 - 3	40 - 50
20 - 25	1 - 2	40 - 60
25 - 30	1 - 2	60 - 80

Climates with temperatures exceeding 22 °C:

Volumetric Loading $BOD_5/m^3/d$	up to 300 g
HRT approximately	5 days
Depth	2.5 - 5 m

Cold Climates : 50 percent estimated reduction in BOD_5

Volumetric Loading $BOD_5/m^3/d$	as low as 40g
HRT approximately	50 days

Design

Input

Flow, m^3/day	18925
Influent BOD_5, mg/L	250
Temperature, °C	10
Depth, m	3
Length to Width Ratio,	1
Volumetric Loading, $BOD_5/m^3/d$	60
HRT, d	5
Slope	3

Output-Volumetric Loading

Volume, m^3	78854
Length, m	171
Width, m	171

Output-Detention Time

Volume, m^3	94625
Length, m	187
Width, m	187
Detention Time, days	5

Example 3-3. Design of Facultative Pond with Frequently Used Formulations

Wehner-Wilhelm Equation

The Wehner-Wilhelm Equation is used when designing for conditions between ideal plug flow and complete mix.

$$\frac{C_e}{C_0} = \frac{4\, a e^{1/(2D)}}{(1+a)^2 (e^{a/2D}) - (1-a)^2 (e^{-a/2D})}$$

where:

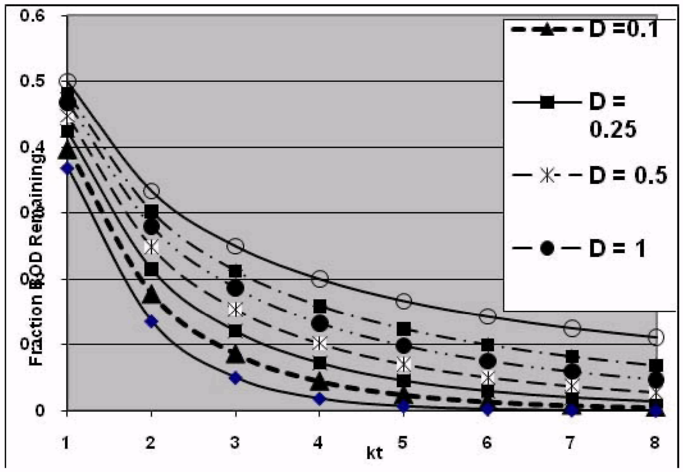
C_O = influent BOD concentration, mg/L
C_e = effluent BOD concentration, mg/L
e = base of natural logarithms, 2.7183
$a = (1 + 4ktD)^{0.5}$
k = 1st order reaction rate constant/d
t = HRT, d
D = dimensionless dispersion number
$D = H/vL = Ht/L^2$
H = axial dispersion coefficient, area per unit time
v = fluid velocity, length per unit time
L = length of travel path of a typical particle

Dispersion numbers measured in wastewater ponds range from 0.1 to 2.0 with most values less than 1.0. The selection of a value for D can dramatically affect the detention time required to produce a given quality effluent. The selection of a design value for k can have an equal effect. A modified form of the chart prepared by Thirumurthi (1974) is shown to facilitate solving for D:

$$D = \frac{0.184[tv(W+2d)]^{0.489}(W)^{1.511}}{(Ld)^{1.489}}$$

where:

D = dimensionless dispersion number
t = HRT, d
v = kinematic viscosity, m^2/d
d = liquid depth of pond, m
W = width of pond, m
L = length of pond, m



Wehner-Wilhelm Equation (modified from Thirumuthi, 1974

The variation of the reaction rate constant k with the water temperature is determined by the equation:

$$k_T = k_{20}(1.09)^{T-20}$$

where:

k_T = reaction rate at water temperature T/d

k_{20} = reaction rate at 20 °C = 0.15/d

T = operating water temperature, °C

Design a facultative pond system using the (1) Wehner-Wilhelm Equation surface loading method; (2) the complete mix equation developed in South Africa; (3) and the plug flow equation for the following environmental conditions and wastewater characteristics.

Flow rate = Q =	3785 m^3/d (1MGD)
Influent BOD_5, C_0 =	200 mg/L
Required effluent BOD_5, C_e =	30 mg/L
Operating water temperature =	5 °C
Reaction Rate k_T at 20 °C =	0.15/ d

Calculate k_t = reaction rate at water temperature T 1/day.

$k_t = k_{20} {}^{x}(1.09)^{T-20}$

$k_t = 0.04118$

First Iteration: Solve for "a" first.

t_1 = Assumed HRT =	53.9 d
D = Assumed dimensionless dispersion # =	0.1

$a = (1+4\ k_t\ Dt)$^0.5
a_1= 1.37399

Solve the Wehner-Wilhelm equation to determine if the two sides are equal.

Calculate the dimensions of the pond.

L to W =	3
v = Kinematic Viscosity =	0.1312 m^2/day
t = Optimum HRT (final iteration)	53.9 days
d = Liquid depth of pond =	2.45 m (8.0379 ft
Volume =	204012 m^3 (53.9 MG)

Divide the flow into streams.

Number of Streams =	1
Volume in one stream =	204012 m^3

Divide volume into 3 equal volumes

Volume in one pond is =	68004 $m^{3\,(}$18.0 MG)
Surface Area of each is =	2.78 m^2 (6.9 acres)
Theoretical HRT in each pond is =	53.9 days

Surface Area = L x W

W =	96.2 m^2 (315.6 $ft^{2)}$
L =	288.6 m^2 (946.7 $ft^{2)}$

Approximately measured HRT is a value of 1/2 the theoretical value t_d = 26.95 days.

The following equation was developed by Polprasert and Bhattarai (1985) to improve D value accuracy for the Wehner-Wilhelm Equation based on a measured HRT.

With measured HRT (assumed to be 1/2) and dimensions of one cell, the accurate dispersion number is

t_d = 26.95 d
$D = [0.184\ ((t \times v\ (W+2d))^{0.489} \times W^{1..511}] / (L \times d)^{1.489}$
D = 0.185

With theoretical detention time, the dispersion number is

t_d = 53.9 d
D = 0.2597

The dimensions of each cell using theoretical HRT and initial dispersion number

L to W = 3
W = 96.19 m (315.6 ft)
L = 288.57 m (946.7 ft)

Calculate the effluent BOD_5 concentration using the theoretical HRT as the Wehner-Wilhelm Equation was developed based on the theoretical value. Total HRT is used because the equation represents the entire system.

$D = 0.2597$
$a_3 = 1.374$
$C_e = [4\, a \times e^{1/(2D)}] / [\, (1 + a)^2\, (e^{a/2D}) - (1 - a)^2\, (e^{-a/2D})\,] / C_o$
$C_e = 30$ mg/L

Organic Loading Method

Depth =	2.45 m (8.04 ft)			
Organic Load = BOD_5 x Q/1000 =	757 kg/d (1669 lbs/d)			
Organic Loading Rate = kg/ha (lbs/ac)/d =	27 (60)	18 (40)	14 (30)	5 (10)
Area Required = ha (ac) =	11 (28)	17(42)	23 (56)	68 (167)
Volume = m^3 =	275774	413660	551547	1654641

Area or volume divided into three or four cells in series.

Complete Mix Model

$$\frac{C_n}{C_o} = \left(\frac{1}{1 + k_c t_n}\right)^n \qquad t = \frac{n}{k}\left[\left(\frac{C_o}{C_n}\right)^{\frac{1}{n}} - 1\right]$$

Flow rate = Q =	3785 m^3/d (1.00 MGD)
Influent BOD_5 =	200 mg/L
Effluent BOD_5 =	30 mg/L
Influent SS =	150 mg/L
No. of cells in series =	3
Water temperature =	5 °C
Reaction rate at 35 °C	0.5/d
Temp. corr. coef.	1.085/ d
K_t = rate @ t	0.043/d
HRT =	61.17d
Volume =	231533 m^3 (61.16 MG)
Depth =	2.45 m (8 ft)
Surface Area =	9.45 ha (23.4 ac)

Gloyna Method

$$V = 0.035Q\, (BOD_5)\, (1.099)^{LIGHT\,(35-T)/250}$$

where:

Q = Flow Rate =	3785 m^3/d (0.999360 MG/D
BOD_5 =	200 mg/L
LIGHT =	200 Langleys
Temp. coef. =	1.099
Temperature =	5 °C
Volume =	255334 m^3 (67.45 MG)

Predicted Effluent BOD_5 = 80 to 90 percent reduction = 20 - 40 mg/L
Total volume will be divided into three or four equal cells.

Plug Flow Model

$$\frac{C_e}{C_0} = \exp[-kpt] \qquad t = \ln\left(\frac{C_e}{C_o}\right)\frac{1}{Kp}$$

where:

C_o = Influent BOD_5 =	200 mg/L
C_e = Effluent BOD_5 =	30 mg/L
k_{p20} = Plug flow reaction rate at 20 $^{\circ}$C =	0.07 /d
HRT =	98.7 d
Θ = Temperature Correction Factor =	1.09
T = Water temperature =	50 °C
kp_T = Plug flow reaction rate at $T\,^{\circ}$C =	0.01922 /d

With Influent and Effluent specified calculate HRT.

t =	98.7 d

With Influent and hydraulic detention time specified calculate Effluent.

C_e =	30 mg/L
Volume = Q x t =	373646 m^3 (98 MG)
Surface Area = V/depth =	152508 m^2 (37.7ac)

Summary of Results

Method	HRT d	Volume m^3	Surface area m^2
Wehner-Wilhelm	53.9	204012	83270
Surface Area*	145.7	551547	225124
Complete mix	61.2	231533	94503
Gloyna	67.5	255334	104218
Plug flow	98.7	373646	152508

*Values based on surface loading rate of (34 kg/ha/d (30 lbs/ac/d). At 66 kg/ha/d (60 lbs/ac/d), the results would be close to the others but a reliable effluent BOD_5 of 30 mg/L might not be as attainable.

Example 3-4. Detention Times in Partial Mix Aerated Ponds
Compare detention times for the same BOD_5 removal levels in partial mix aerated ponds having one to
five cells.
Assume

C_o = 200 mg/L
k = 0.28/d
T_w = 20 °C

1. Solve the equation for a single cell system:

$$t = \frac{n}{k}\left[\left(\frac{C_0}{C_n}\right)^{\frac{1}{n}} - 1\right]$$

$$t = \frac{1}{0.28}\left[\left(\frac{200}{30}\right)^{\frac{1}{1}} - 1\right]$$

$$t = 20.2\,d$$

2. Similarly, when:

n = 2	t = 11 days
n = 3	t = 9.4 days
n = 4	t = 8.7 days
n = 5	t = 8.2 days

3. Continuing to increase n will result in the HRT being equal to the HRT in a plug flow reactor. It can be seen from the tabulation above that the advantages diminish after the third or fourth cell.

Example 3-5. Design a Four-Cell Partial Mix Aerated Pond having Two Trains for BOD_5 Removal
Design a four-cell partial mix aerated pond with two trains to remove BOD_5 for the following environmental conditions and wastewater characteristics:

Q =	1136 m^3/d (0.3 MGD)
C_o =	220 mg/L
C_e from fourth cell =	30mg/L
k_{20} = 0.276 d	
Temperature =	8 °C (winter), = 25 °C (summer)
Elevation =	50 m (164 ft)
Depth =	4 m (13.1 ft)

Solution:

Flow Rate = Q =	568 m^3/d
Influent BOD_5 =	220 mg/L

Influent TSS =	200 mg/L
Total N =	30 mg/L
Total P =	10 mg/L
Reaction rate at 20 °C =	0.276/d
Influent temperature °C =	15 °C
Summer air temp. °C = T_a	25 °C
Winter air temp: °C = T_a	8 °C
Temperature correction coef. =	1.09
Surface Elevation =	50 m
Minimum DO =	2 mg/L
Depth =	4 m
Length to Width Ratio =	2
Side slope =	3

1. Start solution by assuming winter pond temperature and determine volume of Cell 1 in the pond system.

Assume influent temperature	12.06 °C
Correct reaction rate for temperature	$k_T = k_{20}(1.036)^{(T-20)}$
	$k_T = 0.210\ d^{-1}$
HRT in Cell 1 =	3.60 d
Effluent BOD_5 in Cell 1 =	$C_0/(1 + kt)$ 125.69 mg/L
Volume in Cell 1=	2044.80 m^3

2. Calculate dimensions of Cell 1 at water surface and the surface area.

Depth =	4 m
Width =	24.51 m
Length =	49.02 m
Surface area in Cell 1 =	1201.61 m^2 (0.134 ac)

3. Check pond temperature using cell area calculated above and equation shown below.

Cell 1 $T_w = (AfT_a + QT_i)/(Af + Q)$ 11.4 °C

If calculated T_w differs from assumed water temperature, iteration is necessary.

Add a freeboard	0.90 m
Dimensions at top of dike in Cell 1	
W top of dike =	29.91 m
L top of dike =	54.42 m

4. For second cell

Influent temperature	11.4 °C
Correct reaction rate for temperature:	$k_T = k_{20}(1.09)^{(T-20)}$

k_T = 0.20/d

Influent BOD_5 in Cell 2 =	125.69 mg/L
HRT in Cell 2 =	3.50 d
Effluent BOD_5 Cell 2 =	73.39 mg/L
Volume in Cell 2 =	1988.00 m^3

Calculate dimensions of Cell 2 at water surface and the surface area.

Depth =	4.00 m
Width =	24.28 m
Length =	48.56 m
Area =	1179.11 m^2 (0.134 ac)
Cell 2 $T_w = (AfT_a + QT_i)/(Af + Q)$	9.67 °C
Add a freeboard	0.9 m

Dimensions at top of dike in Cell 2

W top of dike =	29.68 m
L top of dike =	53.96 m

5. For third cell

Influent temperature	9.7 °C
	k_T = 0.19/d
Influent BOD_5 to Cell 3 =	73.39 mg/L
HRT in Cell 3 =	3 d
Effluent BOD_5 in Cell 3 =	46.61 mg/L
Volume in Cell 3 =	1704.00 m^3

Calculate dimensions of Cell 3 at water surface and the surface area.

Depth =	4.00 m
Width =	23.07 m
Length =	46.14 m
Area =	1064.56 m^2 (0.134 ac)
Cell 3 $T_w = (AfT_a + QT_i)/(Af + Q)$	8.86 °C
Add a freeboard	0.90 m

Dimensions at top of dike in Cell 3

W top of dike =	28.47 m
L top of dike =	51.54 m

6. For fourth cell influent temperature	8.86^{0}C
	k_T = 0.19/d
Influent BOD_5 to Cell 4 =	46.61 mg/L
HRT in Cell 4 =	3 d
Effluent BOD_5 in Cell 4 =	29.91 mg/L
Volume in Cell 4 =	1704.00 m^3

Calculate dimensions of Cell 4 at water surface and the surface area.

Depth =	4.00 m
Width =	23.07 m
Length =	46.14 m
Area =	1064.56 m^2(0.134 ac)
Cell 4 $T_w = (AfT_a + QT_i)/(Af + Q)$	8.44 °C
Add a freeboard	0.90 m
Dimensions at top of dike	
W top of dike =	28.47 m
L top of dike =	51.54 m

7. Determine the oxygen requirements for pond system based on organic loading and Water Temperature. Maximum oxygen requirements will occur during the summer months.

T_w Cell 1 = $(AfT_a + QT_i)/(Af + Q)$	20.1 °C
T_w Cell 2 =	22.6 °C
T_w Cell 3 =	23.8 °C
T_w Cell 4 =	24.4 °C

Organic load (OL) in the influent wastewater

OL on Cell 1 = C_o x Q	5.21 kg/hr
(Calculate effluent BOD_5 from first cell using equations below at T_w for summer.)	
$k_{Tw} = k_{20}$ x (temp. coef.)$^{(Tw-20)}$	0.28/d
$C_1 = C_o/[(kt) + 1]$	110.08 mg/L
Winter =	125.69 mg/L
OL on Cell 2 = Q x C_1	2.61 kg/hr
$k_{Tw} = k_{20}$ x (temp. coef.)$^{(Tw-20)}$	0.30/d
$C_2 = C_1/[(kt) + 1]$	53.45 mg/L
Winter =	73.39 mg/L
OL on Cell 3 = C_2 x Q	1.26 kg/hr
$k_{Tw} = k_{20}$ x (temp. coef.)$^{(Tw-20)}$	0.32 /d
Winter =	46.61 mg/L
OL on Cell 4 = C_3 x Q	0.65 kg/hr
$k_{Tw} = k_{20}$ x (temp. coef.)$^{(Tw-20)}$	0.32/d
$C_4 = C_3/[(kt) + 1]$	13.97 mg/L
Winter =	29.91 mg/L

DO is assumed to be a multiple of organic loading. Multiplication factor (MF) = 1.50

DO in Cell 1 = OL1 x MF =	7.81 kg/hr
DO in Cell 2 = OL2 x MF =	3.91
DO in Cell 3 = OL3 x MF =	1.90
DO in Cell 4 = OL4 x MF =	0.97

8. Use following equation to calculate equivalent O_2 transfer.

$N = N_{DO}/a[(C_{sw}-C_L)/C_s](\text{temp factor})^{(Tw-20)})$

N_{OD} = DO in various cells

$C_{sw} = b \times C_{ss} \times P$

$b = 0.90$

P = ratio of barometric pressure at pond site to pressure at sea level = 0.80

Cell 1 Tap water O_2 sat. value C_{ss} =	9.15 mg/L
Cell 2 =	8.74 mg/L
Cell 3 =	8.56 mg/L
Cell 4 =	8.46 mg/L

Cell 1 C_{sw} =	6.59 mg/L
Cell 2 C_{sw} =	6.29 mg/L
Cell 3 C_{sw} =	6.16 mg/L
Cell 4 C_{sw} =	6.09 mg/L
a = O_2 transfer in wastewater/ O_2 transfer in tap water	0.90
C_L = min. O_2 conc. to be maintained in wastewater	2.00 mg/L
C_s = O_2 sat. value of tap water at 20 °C and 1 atm.	9.17 mg/L
Temp. factor =	1.025

$N1$ =	17.29 kg/hr
$N2$ =	8.70
$N3$ =	4.23
$N4$ =	2.18

9. Evaluate surface and diffused air aeration equipment to satisfy oxygen requirement only.

Power for surface aerators (approx.)	1.90 kg O_2/kWh
	1.40
Power for diffused air (approx.)	2.70
	2.00

Total power for surface aeration

Cell 1	9.10 kW	12.35 hp
Cell 2	4.58	6.21
Cell 3	2.23	2.99
Cell 4	1.15	1.54

Total power for diffused aeration

Cell 1	6.40 kW	8.64 hp
Cell 2	3.22	4.35
Cell 3	1.57	2.12
Cell 4	0.81	1.09

These surface and diffused aerator power requirements must be corrected for gearing and blower efficiency.

Gearing efficiency	0.90
Blower efficiency	0.90

Total power req. corrected for efficiency (surface aerators)

Cell 1	10.11 kW	13.48 hp
Cell 2	5.09	4.31
Cell 3	2.48	1.20
Cell 4	1.27	0.31
Total Power - Surface aerators	18.95	19.30

Power Cost/ kWhr: $ 0.06
Total Power Costs for Surface Aerators ($/yr) 9958.02

Cell 1 - Diffused aeration	7.11 kW	9.49 hp
Cell 2	3.58	3.03
Cell 3	1.74	0.85
Cell 4	0.90	0.22
Total Power	13.33	13.58

Power Cost/ kWhr: $ 0.06
Total Power Costs for Diffused Aerators ($/yr) 7007.49/yr

These power requirements are approximate values and are used for the preliminary selection of equipment.

Example 3-6. Detention Times in Complete Mix Aerated Ponds having One to Five Cells
Compare detention times for the same BOD_5 removal levels in complete mix aerated ponds having **one to five cells**. Assume

C_o = 200 mg/L
k = 2.5/d
T_w = 20 °C

Solution
1. Solve the following equation for a single cell system:

$$t = \frac{n}{k}\left[\left(\frac{C_0}{C_n}\right)^{\frac{1}{n}} - 1\right]$$

$$t = \frac{1}{2.5}\left[\left(\frac{200}{30}\right)^{\frac{1}{1}} - 1\right]$$

$$t = 2.7\,d$$

2. Similarly:
when

n = 2	t = 1.04 days
n = 3	t = 0.35 days
n = 4	t = 0.24 days

3. Continuing to increase *n* will result in the detention time being equal to the detention time in a plug flow reactor. It can be seen from the tabulation above that the advantages diminish after the third cell. This advantage is lost because of the need for a hydraulic residence of time of approximately one and one-half days for the biomass to develop.

Example 3-7. Design of a Four-Cell Complete Mix Aerated Pond having Two Trains for BOD_5 Removal

Design a four-cell complete mix aerated pond with two trains to remove BOD_5 for the following environmental conditions and wastewater characteristics:

Q =	1136 m^3/d (0.3 MGD)
C_o =	220 mg/L
C_e from fourth cell =	10 mg/L
k_{20} = 2.5/d	
Air temperature (winter) =	8 °C,
(summer) =	25 °C
Elevation =	50 m
DO =	2 mg/L in all cells
Depth =	4 m (13.1 ft).

Solution:

Flow Rate = Q =	568 m^3/d
Influent BOD_5 =	220 mg/L
Influent TSS =	200 mg/L
Total N =	30 mg/L
Total P =	10 mg/L
Reaction Rate at 20 °C =	2.5 d^{-1}
Influent temperature °C =	15 °C
Winter air temp: °C = T_a =	8 °C
Summer temp. °C = T_a	25 °C
f = units conversion factor =	0.50
Temperature correc. coef. =	1.09
Surface Elevation =	50 m
Minimum DO Conc. =	2 mg/L
Depth =	4 m
Length to Width Ratio =	2
Side slope =	3

1. Start solution by assuming winter pond temperature and determine volume of a Cell 1 in the pond system.

Assume water temperature: 12.7 °C

Correct reaction rate for temperature: $k_T = k_{20}(1.09)^{(T\text{-}20)}$

$k_T = 1.34/\text{d}$

HRT in Cell 1 = 1d

Effluent BOD_5 in Cell 1 = 94.13 mg/L

Volume in Cell 1 = 568 m^3

2. Calculate dimensions of Cell 1 at water surface and the surface area.

Depth = 4 m

Width = 16.48 m

Length = 32.97 m

Surface area in Cell 1 = 543.40 m^2 (0.134 ac)

3. Check pond temperature using cell area calculated above and equation shown below:

Cell 1 $T_w = (AfT_a + QT_i)/(Af + Q)$ 12.74 °C

If calculated T_w differs from assumed water temperature, iteration is necessary.

Add a freeboard 0.90 m

Dimensions at top of dike in Cell 1

W top of dike = 21.88 m

L top of dike = 38.37 m

4. For Cell 2, influent water temperature = 12.74 °C

Correct reaction rate for temperature: $k_T = k_{20}(1.09)^{(T\text{-}20)}$

$k_T = 1.34/\text{d}$

Influent BOD_5 = 94.13 mg/L

HRT = 1 d

Effluent BOD_5 = 40.28 mg/L

Volume = 568 m^3

Calculate dimensions at water surface and the surface area.

Depth = 4 m

Width = 16.48 m

Length = 32.97 m

Area = 543.40 m^2 (0.134 ac)

Cell 2 $T_w = (AfT_a + QT_i)/(Af + Q)$ 11.20 °C

Add a freeboard 0.90 m

Dimensions at top of dike 2

W top of dike = 21.88 m

L top of dike = 38.37 m

5. For Cell 3, influent temperature = 11.20 °C

k_T = 1.17/d

Influent BOD_5 = 40.28 mg/L

HRT = 1 d

Effluent BOD_5 = 18.55 mg/L
Volume = 568 m^3

Calculate dimensions of Cell 3 at water surface and the surface area.
Depth = 4 m
Width = 16.48 m
Length = 32.97 m
Area = 543.40 m^2 (0.134 ac)

Cell 3 $T_w = (AfT_a + QT_i)/(Af + Q)$ 10.17 °C

Add a freeboard 0.90 m

Dimensions at top of dike
W top of dike = 21.88 m
L top of dike = 38.37 m

6. For Cell 4, influent temperature = 10.17 °C
k_T = 1.07/d
Influent BOD_5 = 18.55 mg/L
HRT = 1 d
Effluent BOD_5 = 8.96 mg/L
Volume = 568 m^3

Calculate dimensions of Cell 4 at water surface and the surface area.
Depth = 4.00 m
Width = 16.48 m
Length = 32.97 m
Area = 543.40 m^2 (0.134 ac)

Cell 4 $T_w = (AfT_a + QT_i)/(Af + Q)$ 9.47 °C

Add a freeboard 0.90 m
Dimensions at top of dike
W top of dike = 21.88 m
L top of dike = 38.37 m

7. Determine the oxygen requirements for pond system based on organic loading and water temperature. Maximum oxygen requirements will occur during the summer months. Use Equation 3.14 to estimate pond temperature during the summer.
T_w Cell 1 = $(AfT_a + QT_i)/(Af + Q)$ 18.2 °C
T_w Cell 2 = $(AfT_a + QT_i)/(Af + Q)$ 20.4 °C
T_w Cell 3 = $(AfT_a + QT_i)/(Af + Q)$ 21.9 °C
T_w Cell 4 = $(AfT_a + QT_i)/(Af + Q)$ 22.9 °C

OL in the influent wastewater

OL on Cell 1 = C_o x Q 5.21 kg/hr

Calculate effluent BOD_5 from first cell using equations below at T_w for summer.

$k_{Tw} = k_{20}$ x (temp. coef.)$^{(Tw\text{-}20)}$ 2.15 d^{-1}

$C_1 = C_o/[(kt) + 1]$ 69.90 mg/L

Winter = 94.13 mg/L

OL on Cell 2 = Q x C_1 1.65 kg/hr

$k_{Tw} = k_{20}$ x (temp. coef.)$^{(Tw\text{-}20)}$ 2.59/d

$C_2 = C_1/[(kt) + 1]$ 19.45 mg/L

Winter = 40.28 mg/L

OL on Cell 3 = C_2 x Q 0.46 kg/hr

$k_{Tw} = k_{20}$ x (temp. coef.)$^{(Tw\text{-}20)}$ 2.95/d

$C_3 = C_2/[(kt) + 1]$ 4.93 mg/L

Winter = 18.55 mg/L

OL on Cell 4 = C_3 x Q 0.12 kg/hr

$k_{Tw} = k_{20}$ x (temp. coef.)$^{(Tw\text{-}20)}$ 3.21/d

$C_4 = C_3/[(kt) + 1]$ 1.17 mg/L

Winter = 8.96 mg/L

DO is assumed to be a multiple of organic loading

Multiplying factor (MF) 1.50

DO in Cell 1 = OL1 x MF 7.81 kg/hr

DO in Cell 2 = OL2 x MF 2.48

DO in Cell 3 = OL3 x MF 0.69

DO in Cell 4 = OL4 x MF 0.18

8. Use following equation to calculate equivalent O_2 transfer.

$N = N_{OD}/(a[(C_{sw}-C_L)/C_s](\text{temp factor})^{(Tw\text{-}20)})$

N_{OD} = DO in various cells

$C_{sw} = b$ x C_{ss} x P

$b = 0.90$

P = ratio of barometric pressure at pond site to pressure at sea level 0.80

Cell 1 Tap water O_2 sat. value C_{ss} = 9.49 mg/L

Cell 2 = 9.10 mg/L

Cell 3 = 8.85 mg/L

Cell 4 = 8.69 mg/L

Cell 1 C_{sw} = 6.84 mg/L

Cell 2 C_{sw} = 6.55 mg/L

Cell 3 C_{sw} = 6.37 mg/L

Cell 4 C_{sw} = 6.26 mg/L

$a = O_2$ transfer in WW/O_2 transfer in tap water	0.90
C_L = min. O_2 conc. to be maintained in wastewater	2.00 mg/L
$C_s = O_2$ sat. value of tap water at 20 °C and 1 atm.	9.17 mg/L
Temp. factor	1.025

$N1$ = 17.19 kg/hr
$N2$ = 5.50
$N3$ = 1.54
$N4$ = 0.39

9. Evaluate surface and diffused air aeration equipment to satisfy O_2 requirement only.

Power req. for surface aerators		1.90 kg O_2/kWh
		(1.40 kg O_2/hp-h)
Power req. for diffused air		2.70 kg O_2/kWh
		(2.00 kg O_2/hp-h)
Total power for surface aeration		
Cell 1	9.05 kW	12.28 hp
Cell 2	2.89	3.93
Cell 3	0.81	1.10
Cell 4	0.21	0.28
Total power for diffused aeration		
Cell 1	6.37 kW	8.60 hp
Cell 2	2.04	2.75
Cell 3	0.57	0.77
Cell 4	0.14	0.19

These surface and diffused aerator power requirements must be corrected for gearing and blower efficiency.

Gearing efficiency	0.90
Blower efficiency	0.90

Total power req. corrected for efficiency

Cell 1 - Surface aerators	10.05 kW	13.48 hp
Cell 2 -	3.21	4.31
Cell 3 -	0.90	1.20
Cell 4 -	0.23	0.31
Total Power -	14.39	19.30

Power Cost/kWhr:	0.06
Total Power Costs for Surface Aerators:	$7564.74 /yr

Cell 1 - Diffused aeration	7.07 kW	9.49 hp
Cell 2 -	2.26	3.03
Cell 3 -	0.63	0.85
Cell 4 -	0.16	0.22
Total Power -	10.13 kW	13.58 hp

Power Cost/kWhr:	0.06
Total Power Costs for Diffused Aeration:	$5323.33/yr

These power requirements are approximate values and should be used for the preliminary selection of equipment.

10. Evaluation of power requirements for maintaining a complete mix reactor.
Power required to maintain solids suspension = 6.00 kW/1000 m^3 (30.48 hp/MG)

Total power required in all Cells = 3.41 kW 4.57 hp

11. Total power required in system will be the sum of the maximum power required in each cell as measured above.

Assuming that complete mixing is to occur in all cells, use the first set shown below. An alternative is to use the power calculated for each cell to satisfy O_2 demand or a mixture of complete mix and O_2 requirements.

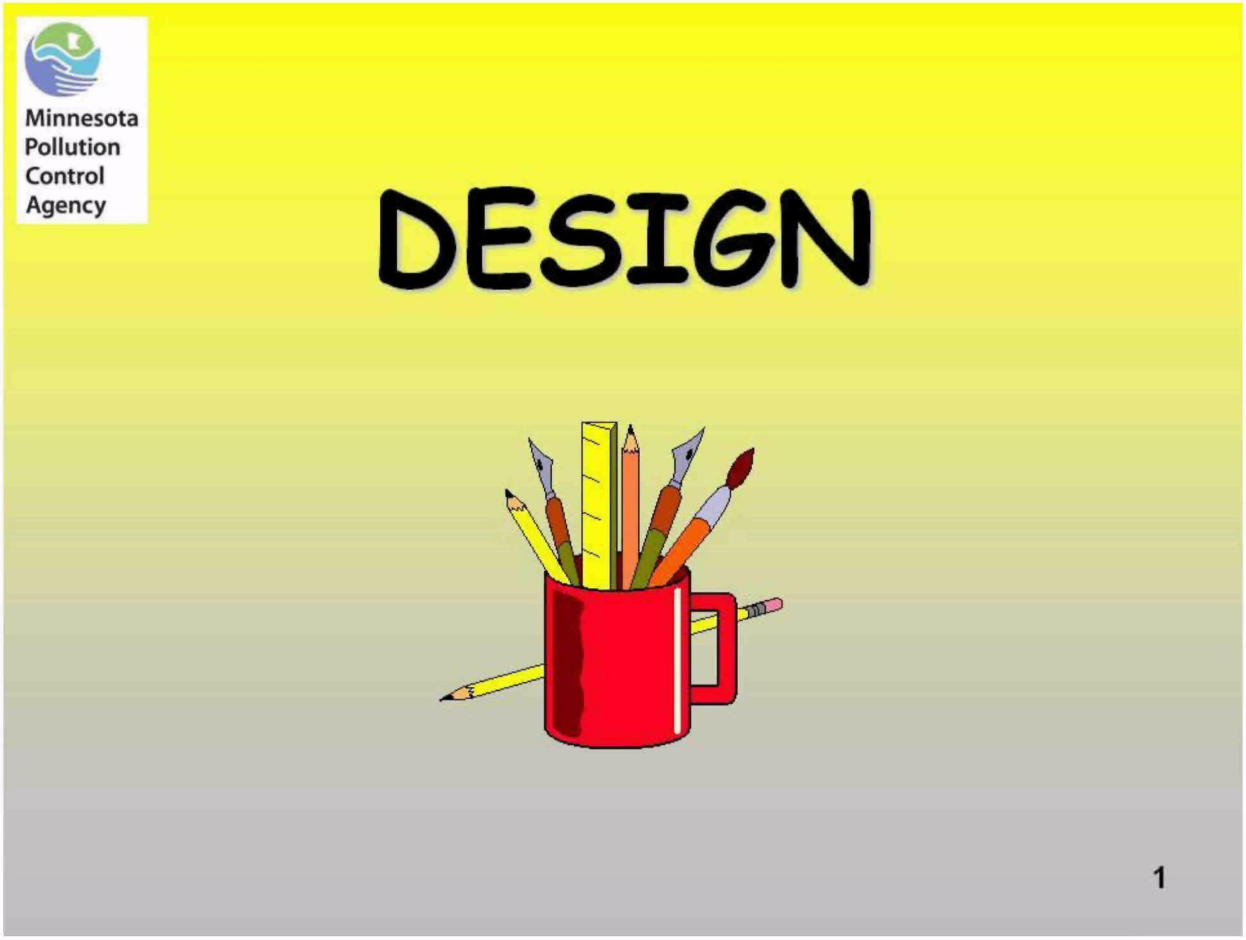
Power Required for Complete Mix in All Cells

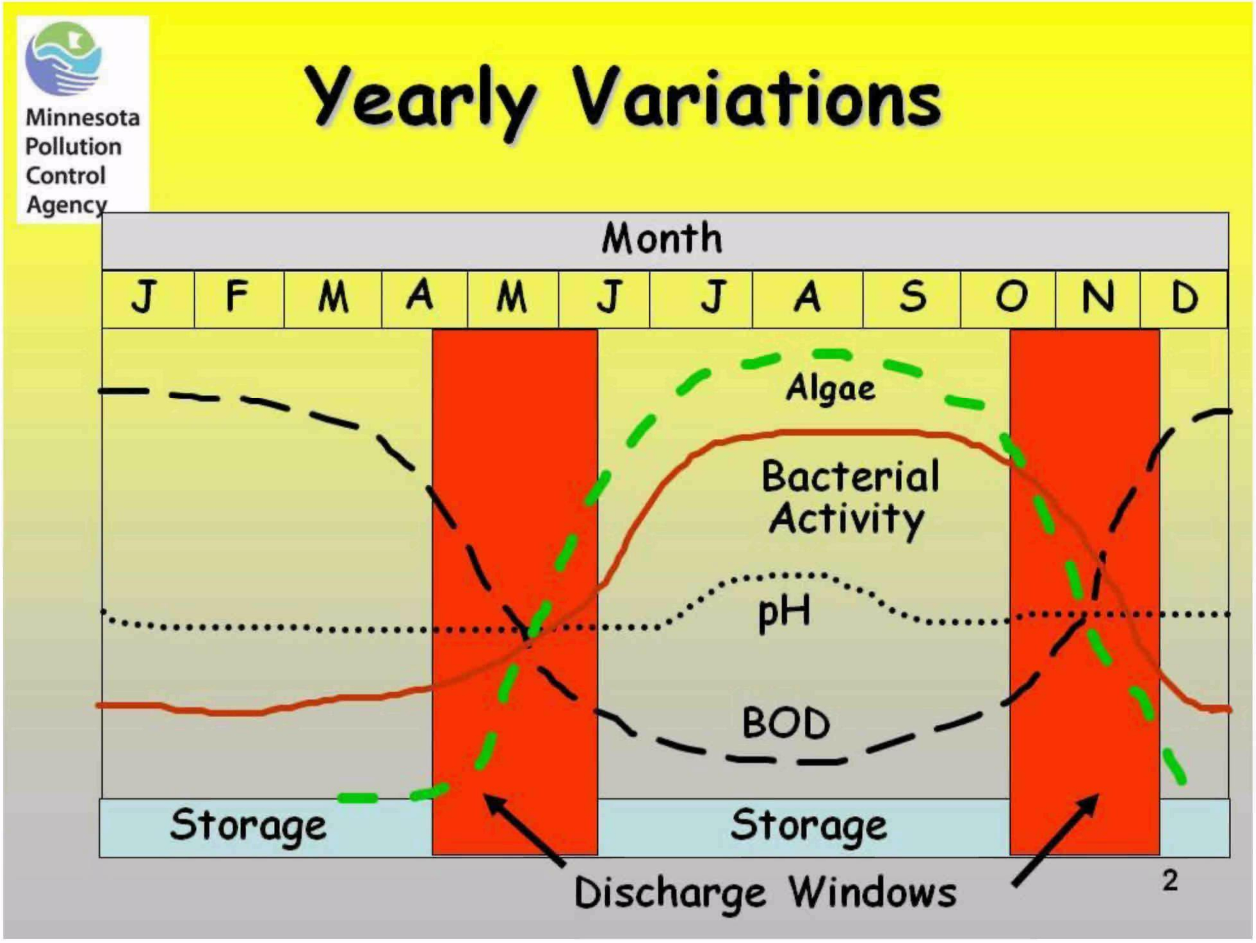
All Cells =	3.41 kW	
Total =	13.63 kW	
Power Costs =		$7164.98/year

Power Requirements for Each Cell Based on BOD_5 removal

Cell 1 =	10.05 kW	13.48 hp	
Cell 2 =	3.21	4.31	
Cell 3 =	0.90	1.20	
Cell 4 =	0.23	0.31	
Total =	14.39	19.30	
Power Costs =			$7564.74/year

The State of Minnesota Pollution Control Agency has prepared a graphic presentation on designing ponds that provides a user-friendly overview of the entire process. We present it here for everyone's enlightenment, but especially for public utility managers, who may find this version provides some insights into all the elements that need to be addressed when a pond system is being considered for wastewater treatment.



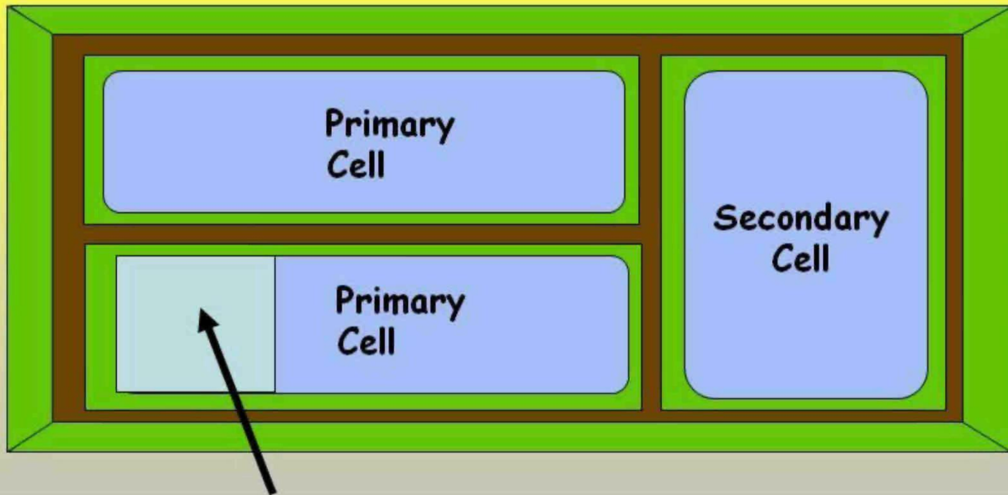

Minnesota Pollution Control Agency
Yearly Variations
Month
J
F
M
A
M
J
J
A
S
O
N
D
Algae
Bacterial Activity
pH
BOD
Storage
Storage
Discharge Windows
2

Basic Design Concepts

- How much “food” – organic load
- How much water – hydraulic load

4

Area/Loading (Non-Aerated Pond)



22 Pounds of BOD/Day/Acre
or
approximately 100 people/acre

5

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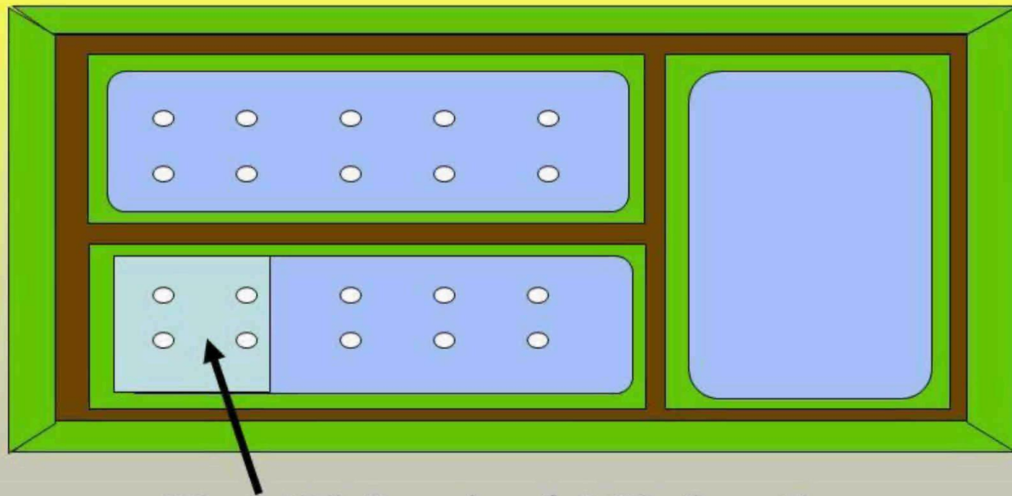
Area/Loading (Aerated Pond)



20 - 400 Pounds of BOD/Day/Acre
or
approximately 75 - 2350 people per acre

6

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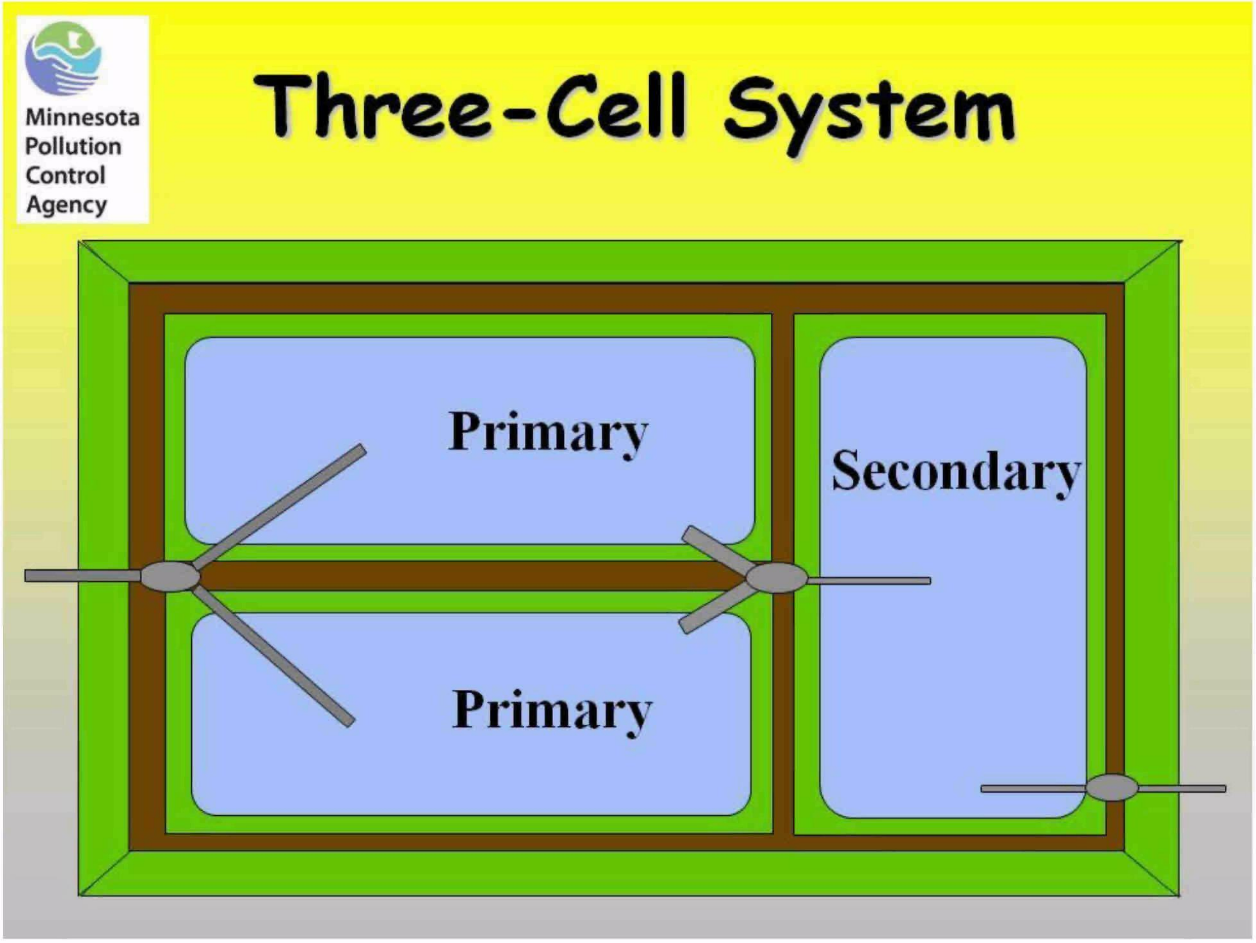
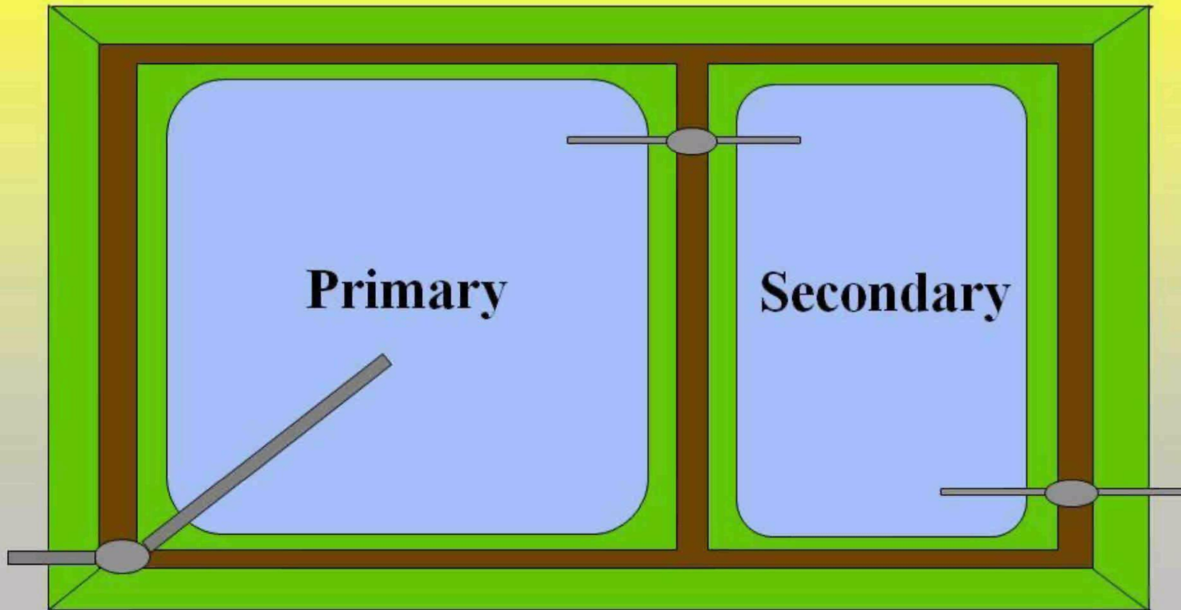
Detention Times

- Non - Aerated
  minimum of 180 - 210 days
  (210 days for spray irrigation)
- Aerated
  minimum of 25 - 35 days

7


Minnesota
Pollution
Control
Agency
Three-Cell System
Primary
Primary
Secondary

Two-Cell System



9

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Agency

Secondary Pond Size

- Volume of the secondary MUST BE at least 1/3 the volume of the entire system

  -Two cell system
    Primary is twice the size of the secondary
  -Three cell system or larger
    All cells are equal size

- Why ?

10

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Secondary Volume Vs. Discharge Time

Secondary Volume (As a % of total volume)	Actual Flow (% of design)			
	25%	50%	75%	100%
20%	34	62	90	115
25%	14	40	66	92
33%	12	36	60	66

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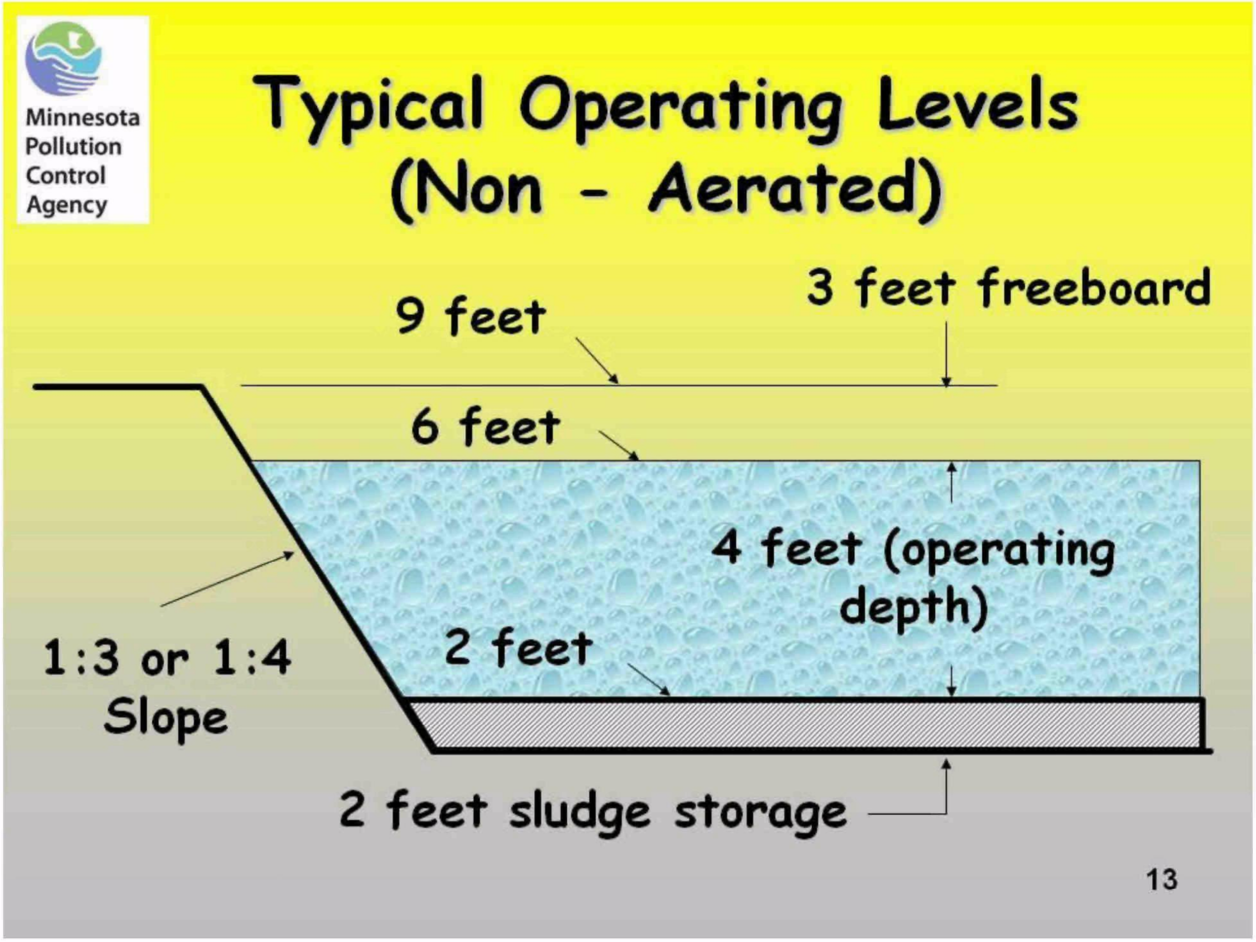
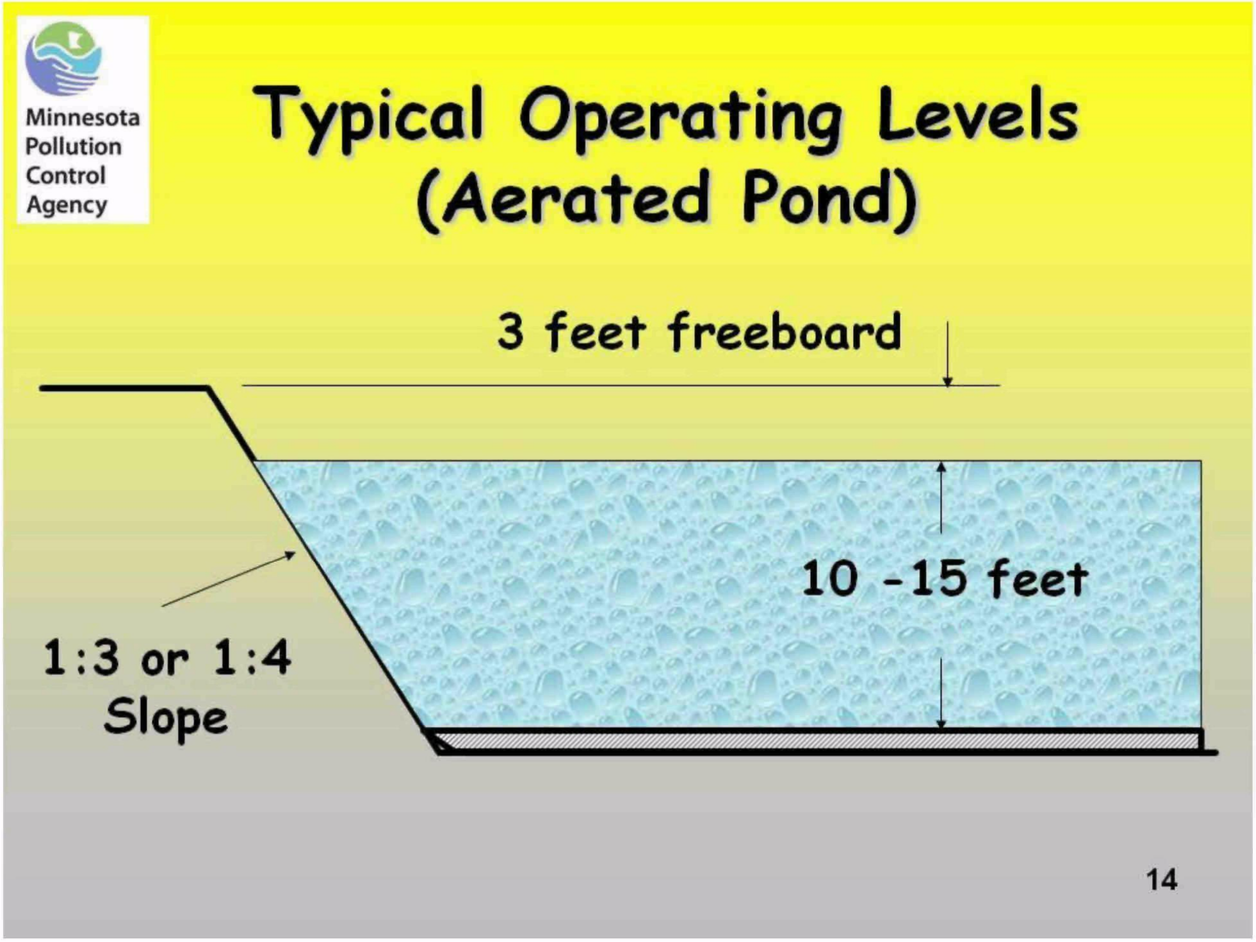
Total Days Needed To Discharge

	Number of Discharges Needed* 1	2	3	4	5
Transfer	0	8	8	8	8
Settle (after transfer)	0	4	4	4	4
Test Results	6	6	6	6	6
Discharge	8	8	8	8	8
Days	14	26	26	26	26
Total Days	14	40	66	92	118

* With four feet of elevation difference or a pump of sufficient size

12


Minnesota
Pollution
Control
Agency
Typical Operating Levels
(Non - Aerated)
9 feet
3 feet freeboard
6 feet
4 feet (operating
depth)
1:3 or 1:4
Slope
2 feet
2 feet sludge storage
13


Minnesota
Pollution
Control
Agency
Typical Operating Levels
(Aerated Pond)
3 feet freeboard
10 -15 feet
1:3 or 1:4
Slope
14

ALLOWABLE SEEPAGE CRITERIA

- Prior to 1975
  3500 gallons per acre per day
- 1975 to present
  500 gallons per acre per day

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Clay Liner

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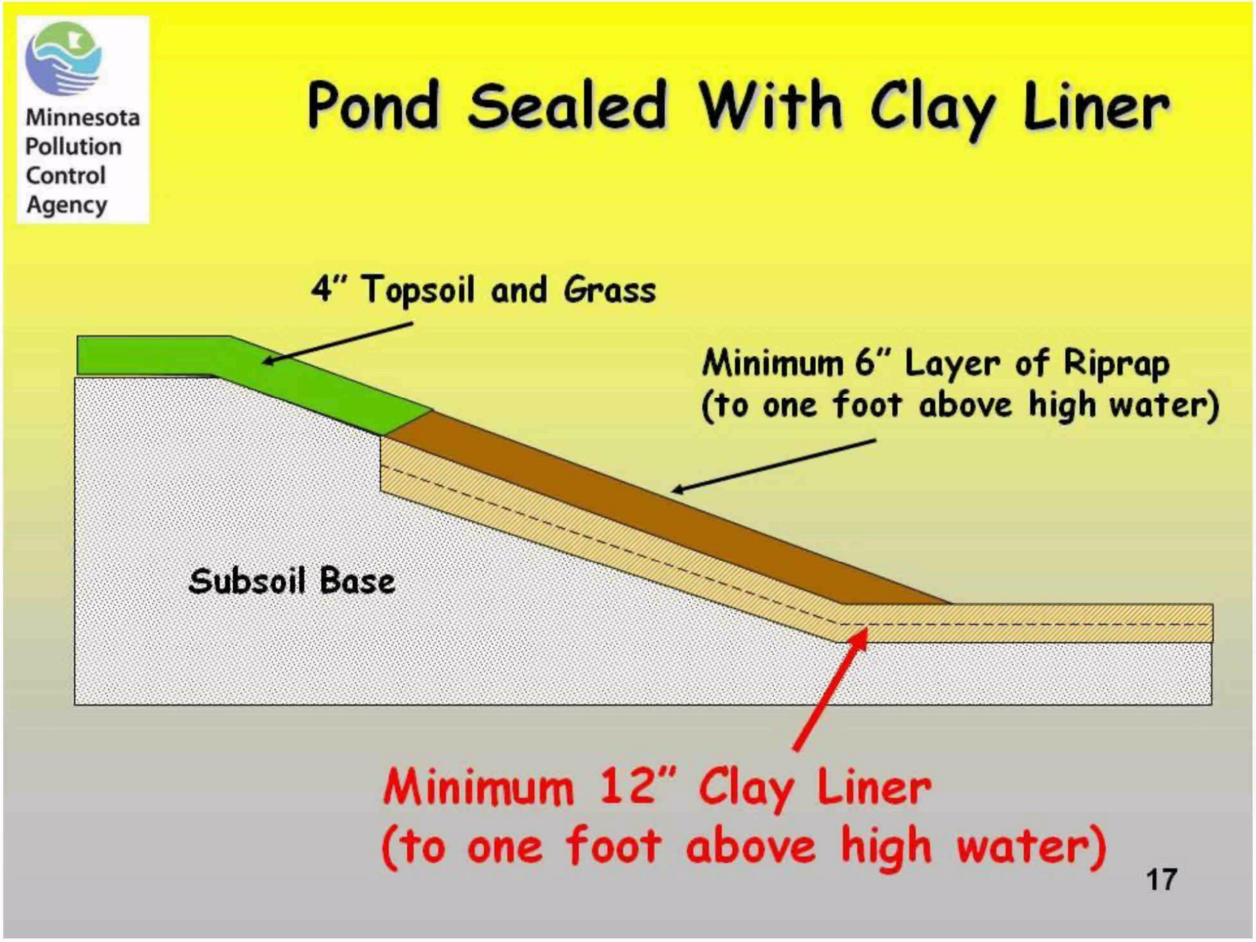
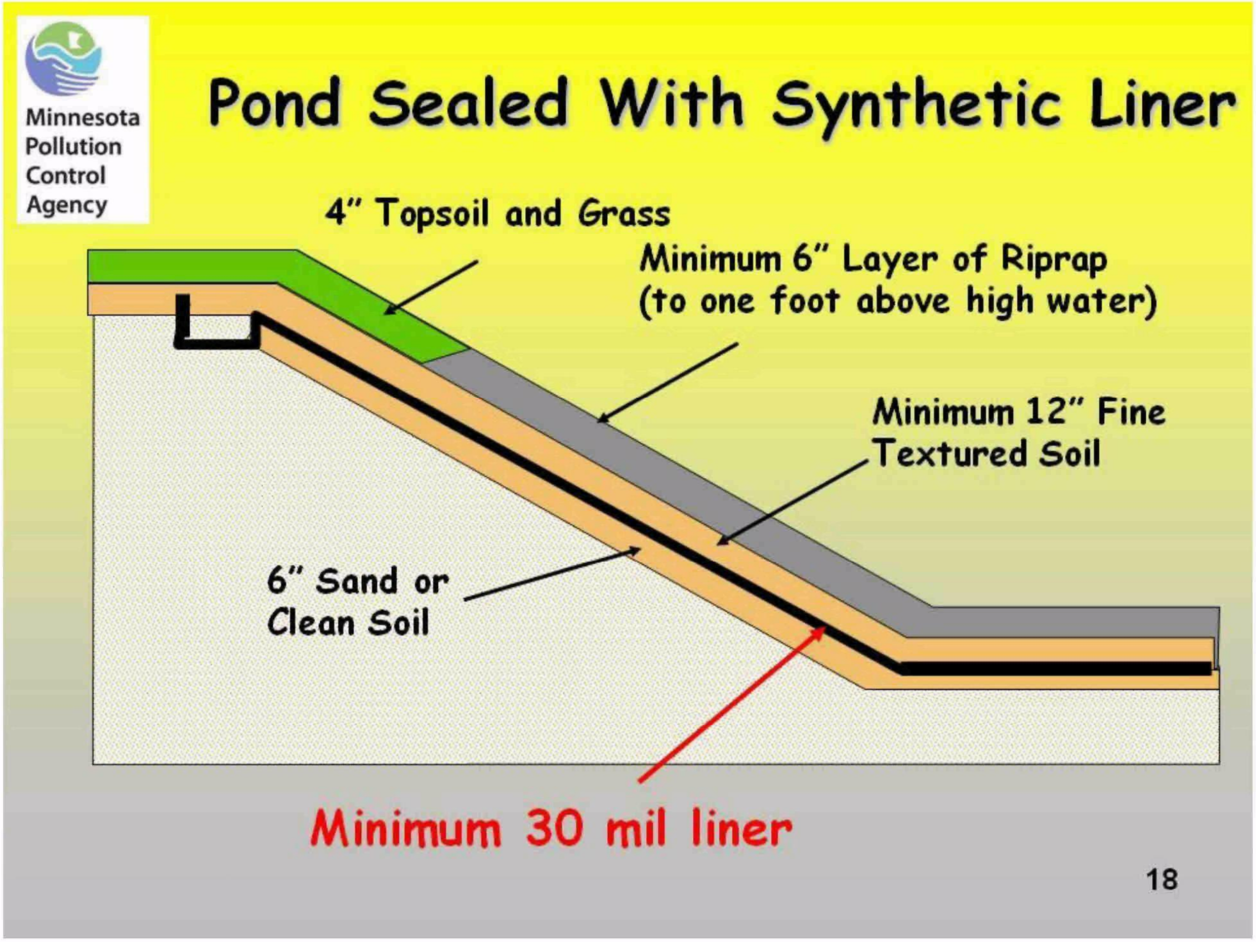

Minnesota Pollution Control Agency
Pond Sealed With Clay Liner
4" Topsoil and Grass
Minimum 6" Layer of Riprap
(to one foot above high water)
Subsoil Base
Minimum 12" Clay Liner
(to one foot above high water)
17


Minnesota Pollution Control Agency
Pond Sealed With Synthetic Liner
4" Topsoil and Grass
Minimum 6" Layer of Riprap
(to one foot above high water)
Minimum 12" Fine Textured Soil
6" Sand or Clean Soil
Minimum 30 mil liner
18

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Installation of Synthetic Liner

19

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Joining of Liner Sheets

20

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Liner Cover Material

21

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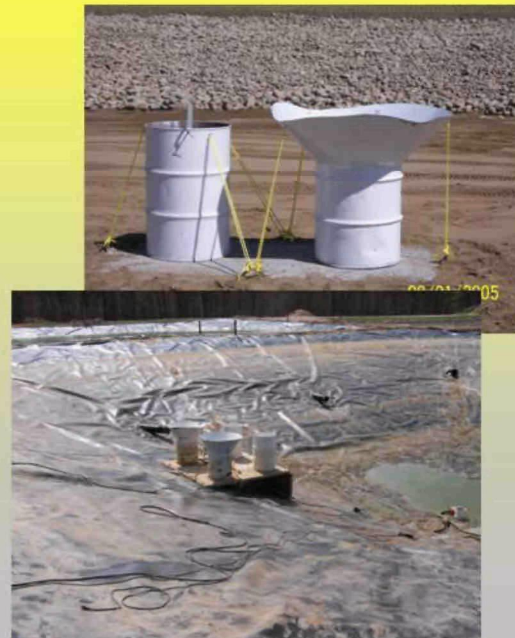
Conduct Water Balance

- To check for actual seepage rate
- Install barrels in pond
- Take depth measurements in barrels:
  - pond depth
  - rainfall
  - evaporation
  - four consecutive weeks
- Do calculations



22

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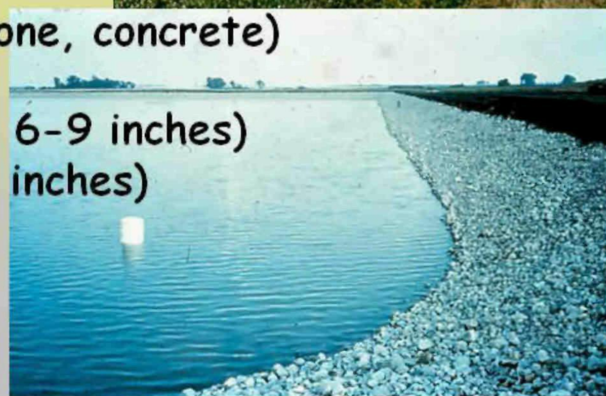
Erosion Protection

- Grass or Rock ?
- Grass (inner dikes)
  - -Not recommended
- Rock
  - -Durable (no sandstone, concrete)
  - -Clean (no fines)
  - -Thickness (at least 6-9 inches)
  - -Size (majority 3-9 inches)



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Control Structures

- Mechanism to regulate/transfer water between pond cells
- Also provides a place to measure depth
- Slide gates and/or valves
- Shut off valve (outside of structure)

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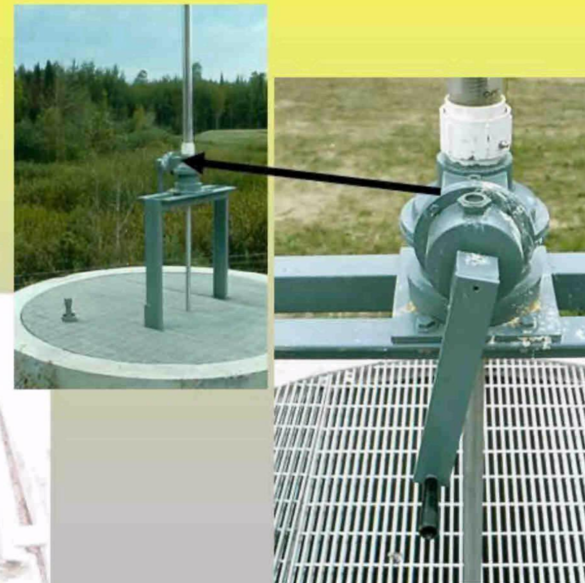
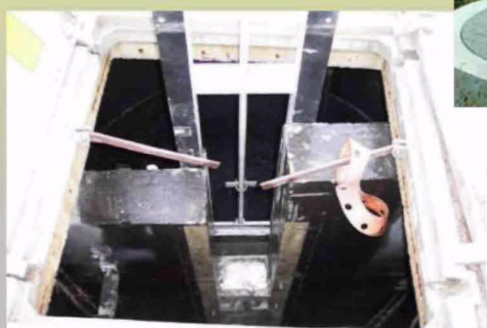
Control Structures

Telescoping Valve or Downward Sluice Gates





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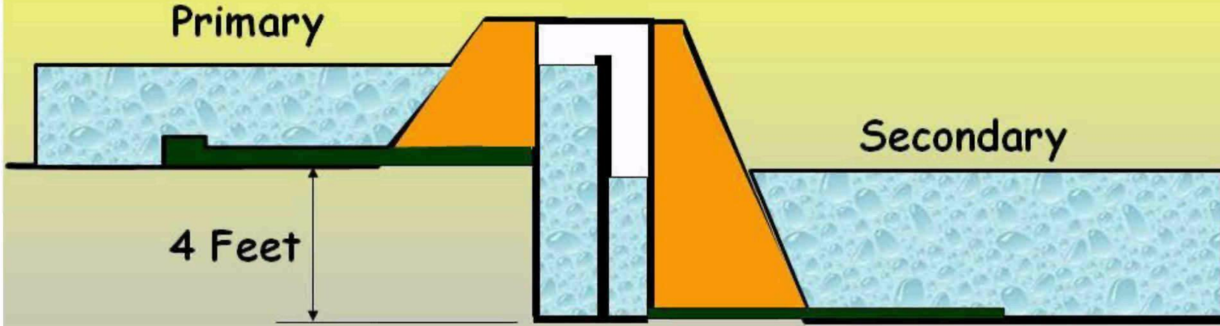
Recommended Elevation Difference



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Influent Line
Horizontal Vs. Vertical ?

- To avoid plugging problems
- Force main directly into pond ?

Solids buildup because of vertical inlet

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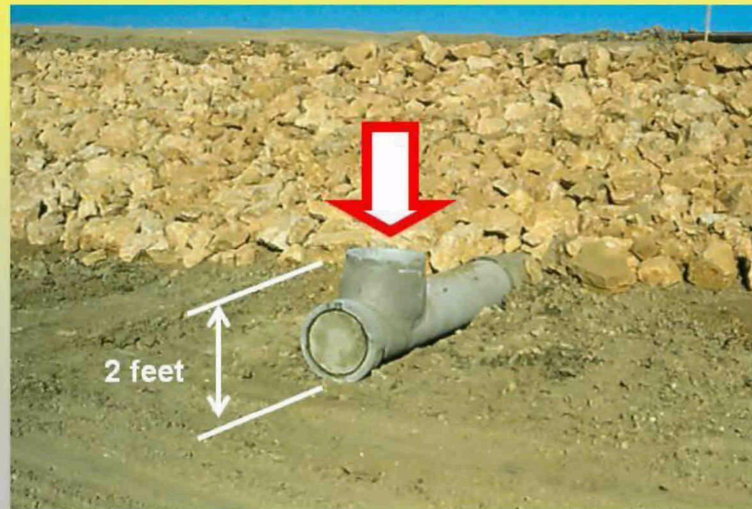
Transfer Piping

- Two foot riser
- Allows for transfer from above; not bottom
- Keeps sludge/nutrients in bottom of pond



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Entrance Road, Gate, Fences, Signs

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Flow Measurement

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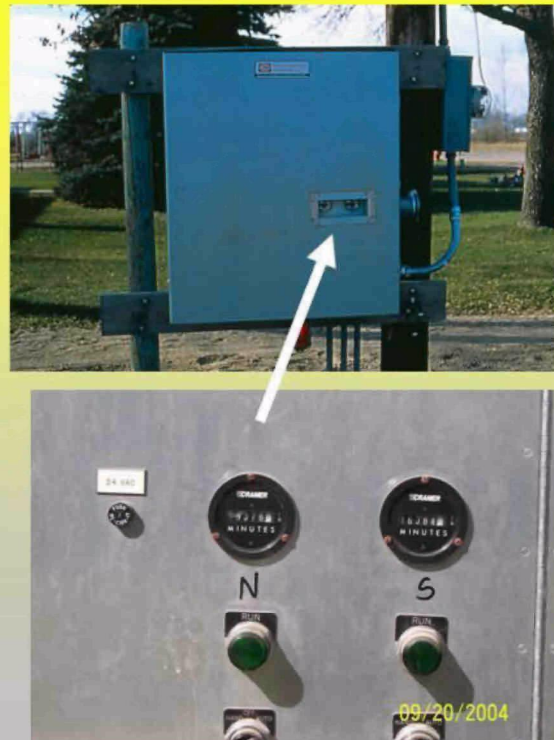
- Very important !
- Permit requirement

  -daily flow
- Most have running time meters

  -must have three time meters; both pumps run together
- Must calibrate pumps

  -at least once/year



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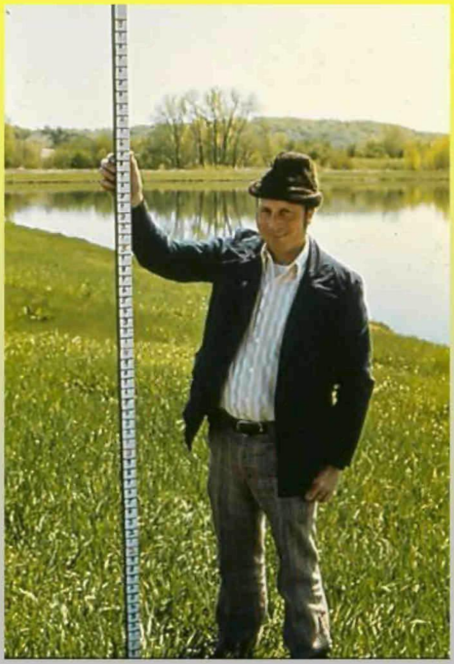
Depth Measurement

- Permit requirement
  - weekly
  - nearest inch
- Measure actual pond depth in control structure



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Location

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Recommended (not a requirement) !!

- 1/2 mile from the nearest city
- 1/4 mile from the nearest resident

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FROM CHAPTER 6
NUTRIENT REMOVAL

Example C-6-1. *NH_3* Conversion in a Partial Mix Pond
Estimate the expected NH_3 conversion in a partial mix aerated pond receiving adequate DO and alkalinity to nitrify an NH_3 concentration of 20 mg/L at a water temperature of 10 °C. Determine the effluent concentration at a detention time of 30 days and at a desired effluent concentration of 10 mg/L.

Temperature =	10 °C
Influent $NH_4^+ = C_0$ =	20 mg/L
Θ = temp correction factor =	1.04
Reaction Rate = k_{20} =	0.0107/d
HRT =	30 d
Effluent NH_3 =	10 mg/L

Solution:

1. Correct reaction rate for temperature:
   $k_T = k_{20}(\Theta)^{(T-20)}$ = 0.005079/d

2. Determine the effluent concentration with known HRT:
   $C_e/C_0 = e^{-kt}$
   C_e = 17.17 mg/L

3. Determine the detention time required to achieve effluent concentration:
   $t = (Ln\ (C_e/C_0)\ /\text{-}k$ = 136.5 d

Example C-6-2. Design for Benthal Stabilization of Waste Solids

Input Data: Insert Design Values in Shaded Fields		
BOD_5	1000 mg/L	
Flow Rate = Q =	136.3 m^3/d	0.036 mgd
Temperature = T =	35 °C	
Solids Retention Time =	20 d	
Nonbiodegradable Solids = Xi =	100 mg/L	
Decay Factor = F_1	0.427 (see Tables for Decay Factors)	
Growth Yield = Y =	0.5 g VSS/g BOD_5	
Annual Average Stabilization Rate = BM	35.64 g VSS/m^2/d	
Aerator Performance = N =	1/25 kg O_2/kWh	
Unit Rate of Benthal Oxygen Demand = B_{O2}	60 g O_2/m^2day	
Solids Fraction = X	0.02	
Solution:		

1. Calculate daily loading rate of biomass:		
$R_{xa} = QY(S_o + X_{so})F_1 =$	29100.05 g VSS/d	
2. Calculate surface area required for stabilization:		
$A = R_{xa}/B_M =$	816 m^2	8788 t^2
3. Calculate aeration power in terms of sludge-water interface area:		
$P/A_o = 4.16 \times 10^{-5} \times B_{02}/N =$	0.001997 A_o m^3	
4. Calculate volume of water column so that aeration intensity will not permit solids to settle by gravity = 1.7 W/m^3 (8.5 hp/MG);	1	
$V_w/A_o = 1000P/1.7 =$	1.997A_o m^3	
5. Estimate volume of sludge during annual cycle:		
$Vs = 365 \times Q \times X/(X \times 10^6) =$	249 m^3	0.065712 MG
6. Calculate required volume of basin:		
$V = V_s + V_w$		
Select a basin and operating depth that will provide a $V_{required}$ using the following equation:		
Pond Depth =	4.62	
$W =$	10 m	
$L =$	10 m	
Side Slope =	0 Horizontal To Vertical	
Volume =	$V = [LW + (L - 2\,sd)(W + 2sd) + 4\,(L - sd)(W - sd)]\,{}^{d}/_{6}$	
V provided =	462 m^3	0.122 MG
Surface Area =	100 m^2	0.025 ac
V required =	118 m^3	0.118462 MG
7. Substitute values for length and width until V provided equals or exceeds V required.		
8. For additional years of operation, use the following table. Substitute years and depths in shaded fields.		

Table C-6-1. Pond Volume and Area vs. Sludge Depth over Time

Years of Operation	Volume Required m^3	Area Required for Sludge Depth						
		Sludge Depth, m						
		1	2	3	4	5	6	
5	2242.138	2242.138	1121.069	747.3792	560.5344	448.4275	373.6896	m^2
		0.55	0.28	0.18	0.14	0.11	0.09	ac
7	3138.993	3138.993	1569.496	1046.331	787.7481	627.7985	523.1654	m^2
		0.78	0.39	0.26	0.19	0.16	0.13	ac

10	4484.275	4484.275	2242.138	1494.758	1121.069	896.855	747.3792	m^2
		1.11	0.55	0.37	0.28	0.22	0.18	ac
15	6726.413	6726.413	3363.206	2242.138	1681.603	1345.283	1121.069	m^2
		1.66	0.83	0.55	0.42	0.33	0.28	ac
20	8968.55	8968.55	4484.275	2989.517	2242.138	1793.71	1494.758	m^2
		222	1.11	0.74	0.55	0.44	0.37	ac

Table C-6-2. Annual Average Stabilization Rates

Month	**Air Temp (ave.)**	**Sludge Temp.***	**B_1** L/m^2/d *(g)***	**B_2*** L/m^2/d *(g)***	**B_1+ B_2 L/m^2/d *(g)***
Jan	0.6	0.1	29.5 *(7.81)*	0.3 *(0.07)*	29.9 *(7.88)*
Feb	4	2	33.6 *(8.88)*	0.5 *(0.13)*	34.1 *(9.01)*
Mar	8.1	6.1	44.3 *(11.71)*	1.9 *(0.51)*	46.2 *(12.23)*
Apr	12	10	57.7 *(15.25)*	7.0 *(1.86)*	64.7 *(17.11)*
May	16.2	14.2	76.6 *(20.26)*	28.0 *(7.40)*	104.6 *(27.67)*
June	20.5	18.5	102.6 *(27.10)*	115.5 *(30.51)*	218.1 *(57.62)*
July	23.7	21.7	127.4 *(33.66)*	331.3 *(87.52)*	458.7 *(121.17)*
Aug	23.1	21.1	122.3 *(32.32)*	271.9 *(71.83)*	394.2 *(104.14)*
Sept	18	16	86.6 *(22.89)*	50.7 *(13.39)*	137.3 *(36.28)*
Oct	11.7	9.7	56.5 *(14.94)*	6.4 *(1.68)*	62.9 *(16.63)*
Nov	5.6	3.6	37.4 *(9.89)*	0.9 *(0.23)*	38.3 *(10.12)*
Dec	1.1	0.1	29.6 *(7.81)*	0.3 *(0.07)*	29.9 *(7.88)*
Average			55.8 *(17.71)*	56.6 *(17.93)*	112.4 *(35.64)*

*Sludge temperature assumed to be 2°C lower than average air temperature.
**B_1 = aerobic stabilization rate
***B_2 = anaerobic stabilization rate

FROM CHAPTER 7
UPGRADING POND EFFLUENTS

Example C-7-1. Intermittent Sand Filter Design

DESIGN DATA AND ASSUMPTIONS

Design Flow = Q = 379 m^3/d (0.1 mgd)
HRT = 0.29 m^3/m^3/d (0.310 mg/ac/d)
Minimum Number of filters = 2
Assum ptions:

Design to minimize operation and maintenance
Gravity flow
Topography and location satisfactory
Adequate land is available at reasonable cost
Filter sand is locally available
Filters are considered plugged when, by the end of the infiltration period, the water from the previous dose has not dropped below the filter surface.

DESIGN

Determine dimensions of filters
Areas of each filter = Q/HLR
Area = 1306.9 m^2 (0.32 ac)
L:W = 2:1
W = 25.56 m (83.9 ft)
L = 51.1 m (167.7 ft)
Minimum of 2 filters required

INFLUENT DISTRIBUTION SYSTEM

Design Assumptions:

Dosing siphon will be used to gravity feed filters.
Electronically activated valves may be used.
Loading sequence will be designed to deliver one-half the daily flow rate to one filter unit/d in two equal doses.
More frequent dosing is acceptable.
Pipe sizes are selected to avoid clogging and to make cleaning convenient.
Hydraulics do not control the rate of treatment.

DOSING BASIN SIZING

No. of dosings/d = 2
Q = Design Q/# of dosings = 189.5 m^3/d (6,692 ft^3)
Volume = 189.5 m^3 (6,692 ft^3)

Install overflow pipe to filters.
Distribution manifold from dosing siphon is designed to minimize velocity of water entering the filter.
Use 25 cm (10 in) diameter pipe in this design.
Each of the outlets from the manifold will be spaced 30.5 cm (10 ft) from each end and 6.4 m (21 ft) on centers on the long side of the filter.

Manifold outlets will discharge onto 91.4 cm by 91.4 cm (3 ft by 3 ft) splash pads constructed of gravel 3.75 cm (1.5 in) in diameter.

FILTER CONTAINMENT

Filter may be contained in a reinforced concrete structure or a synthetic liner to prevent ground water contamination. Slopes of filter bottom are dependent on slope of drainpipe configuration. Use slope of 0.025 percent slope with lateral collection line 4.6 m (15 ft) on center.
Lateral collecting pipe (18 cm [6 in]) and 24 cm (8 in) collection manifolds will provide adequate hydraulic capacity and ease of maintenance.
Minimum freeboard required for filters must be adequate to receive one dosing x safety factor.
Safety Factor = 3.
Water depth assuming no passage = 0.435 m (1.47 ft).

Example C-7-2. Design of a Controlled Discharge Pond Using a Minimum Discharge Period Criterion

In areas of high evaporation rates or high rainfall, the volume of the pond should be adjusted to compensate for the water loss or gain. In this example, it is assumed that rainfall is equal to evaporation, producing no net change in volume. This example illustrates the design of a controlled discharge pond using a minimum discharge period criterion.

Design Conditions:

Minimum discharge period = 30 d

Q = Design Flow Rate = 1893 m^3/d (0.5 mgd)
C_o = Influent BOD_5 = 150 mg/L
C_e = Effluent BOD_5 = 30 mg/L
k_{p20} = reaction rate for plug flow at 20°C = 0.1/d
T_w = water temperature during the critical period of the year = 2 °C

Requirements: Size a controlled discharge wastewater pond system to treat the wastewater and specify the following parameters:

Detention time, t
Volume, V
Surface area, A
Depth, d
Length, L
Width, W

Solution: t = 365 d (minimum discharge period) = 365 d – 30 d = 335 d

Discharge can occur when the effluent quality satisfies standards or the receiving stream flow rate is adequate to receive the effluent. Discharge periods more frequent than once a year can be scheduled, but the performance of the system should be evaluated for the shorter hydraulic residence times. The methods used to design facultative ponds can be used to estimate the performance of a controlled discharge pond.

Raw wastewater is not added to the pond being emptied. Raw wastewater inlets and effluent withdrawal ports are provided in each cell of the system. The cells are connected in series to facilitate operation and flexibility. Three cells are used in this example.

An effective depth (d') of 1.5 m (5 ft) and a total depth (d) of 2 m (6.6 ft) is used. This depth allows for adequate light penetration to sustain photosynthetic oxygen production, providing an aerobic environment through much of the pond contents. The aerobic environment enhances treatment and reduces odor problems. Also, to control odors during discharge periods, the pond is emptied to a minimum depth of 0.5 m (1.5 ft). Additional volume must be provided to compensate for this minimum withdrawal depth.

$$A/cell = \frac{effective\ volume}{n(effective\ depth)} = \frac{Qt}{3d'} \frac{1893\ m^3 (335\ d)}{3(1.5\ m)} = 140{,}900\ m^2\ (35\ ac)$$

This area is used to calculate the total volume for the pond total depth:
$d = 1.5$ m + 0.5 m = 2 m (6.6 ft)

V/cell = (A/cell)(d) = (140,900 m^2)(2 m) = 281,800 m^3 (74.4 x 106 gal)

Significant volumes of wastewater may be lost through seepage if the pond bottom is not sealed. For this example, seepage rates are considered minimal. The length to width ratio of the cells in a controlled discharge pond has less effect on the performance of the system than in flow-through systems. Dimensions for the cells are selected to avoid short-circuiting during discharge or interbasin transfer. A length to width ratio of 2 to 1 was selected for this example.

Dimensions of Ponds

The dimensions of each pond with side slopes of 4 to 1 and a length to width ratio of 2 to 1 can be calculated using the following formula:

$$V = (L \times W) + (L - 2sd)(W - 2sd) + 4\ (L - sd)(W - sd)d/6 \qquad (7\text{-}1)$$

where:

- V_1 = volume of pond #1 = 281,800 m^3
- L = length of pond at water surface, m
- W = width of pond at water surface, m
- s = horizontal slope factor, e.g., 4 to 1 slope, $s = 4$
- d = depth of pond = 2 m (6.6 ft)

$$\frac{(L \times L/2) + (L - 2 \times 4 \times 2)(L/2 - 2 \times 4 \times 2)}{4\ (L - 4 \times 2)(L/2 - 4 \times 2)} = (281{,}800)(6/2)$$

$$3L^3 - 72L + 512 = 845{,}400$$

$$L^2 - 24L = 281{,}630$$

Solve the quadratic equation by completing the square:

$L^2 - 24L + 144 = 281{,}630 + 144$
$(L - 12)^2 = 281{,}774$
$L - 12 = 530.8$

$L = 542.8$ m (1780 ft)
$W = 542.8/2 = 271.4$ m (890 ft)

A freeboard of 0.6 m (2 ft) should be provided. The dimensions of each pond at the top of the inside of the dike will be 547.6 by 276.2 m. The three ponds shall be interconnected by piping for parallel and series operation.

Effluent Quality Prediction
In a pond with an HRT of over 300 days, it is obvious that an effluent with a BOD_5 concentration of less than 30 mg/L can be achieved. However, if it becomes necessary to discharge at shorter intervals, some method of estimating the effluent quality is needed. Controlled discharge ponds are basically facultative ponds, and the effluent quality can be predicted using the plug flow model used to design a facultative pond in Appendix C.

$$\frac{C_e}{C_0} = e^{-k_p t}$$

where:

C_e = effluent BOD_5 concentration, mg/L
C_o = influent BOD_5 concentration, mg/L
e = base of natural logarithms, 2.7183
k_p = plug flow first-order reaction rate/d
t = hydraulic residence time, d
k_{pT} = reaction rate at minimum operating water temperature/d
k_{p20} = reaction rate at 20 °C = 0.1/d
T_w = minimum operating water temperature, °C

Assume that it becomes necessary to discharge from the ponds after a mean HRT of 100 days when the mean water temperature during the period is 2 °C. What would be the concentration of BOD_5 in the effluent?

$k_{pT} = 0.1(1.09)^{(2\text{-}20)}$
$k_{pT} = 0.021$/d
$C_e/150 = e^{-0.021(100)}$
$C_e = 18$ mg/L

The BOD_5 concentration of 18 mg/L in the effluent will easily satisfy the standard of 30 mg/L. TSS concentrations will have to be monitored on-site to ensure that the standards for discharge are met. The guidelines presented at the beginning of this example must be followed in operating the controlled discharge pond system.

Summary:

$V = 845{,}400\ m^3$
$A = (542.8)(271.4)(3) = 441{,}950\ m^2$
$t = 335\ d$

Example C-7-3. Complete Retention Ponds

Figure C-7-1 presents data from NOAA National Weather Service (2004) for estimating evaporation and precipitation in southern Arizona. The air temperature and wind speed data represent mean values over a 54- and 61-year period, respectively. The precipitation data are the mean of the 5 wettest years over a 60-year period. The pan evaporation data represent the year with the lowest evaporation for a 10-year period. These values generally represent the worst case, thus providing for a conservative design.

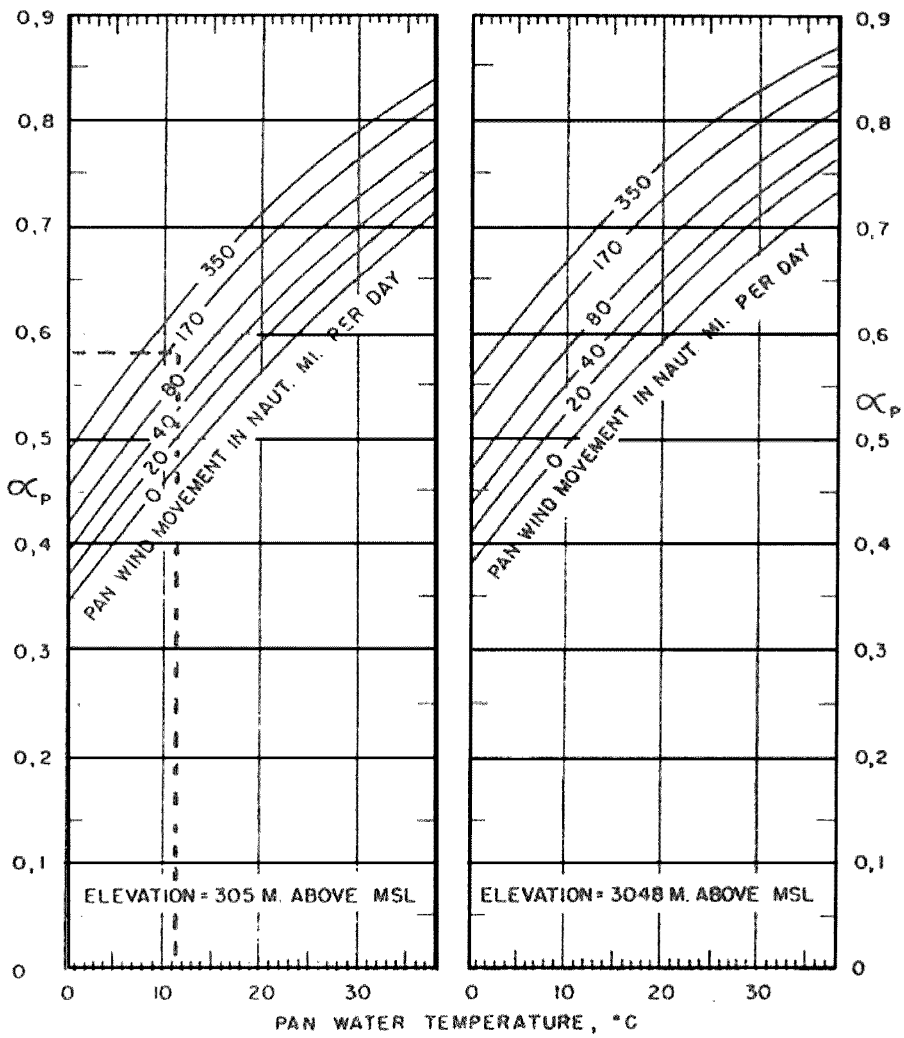

Figure C-7-1. Portion of advected energy (into a Class A Pan) utilized for evaporation in metric units (NOAA National Weather Service, 2004).

The difference between the surface water temperature and the air temperature is assumed to be 1 °C. The selection of this value can have a significant effect on the evaporation losses as shown in Figure C-7-2; therefore, the value must be selected to reflect local conditions.

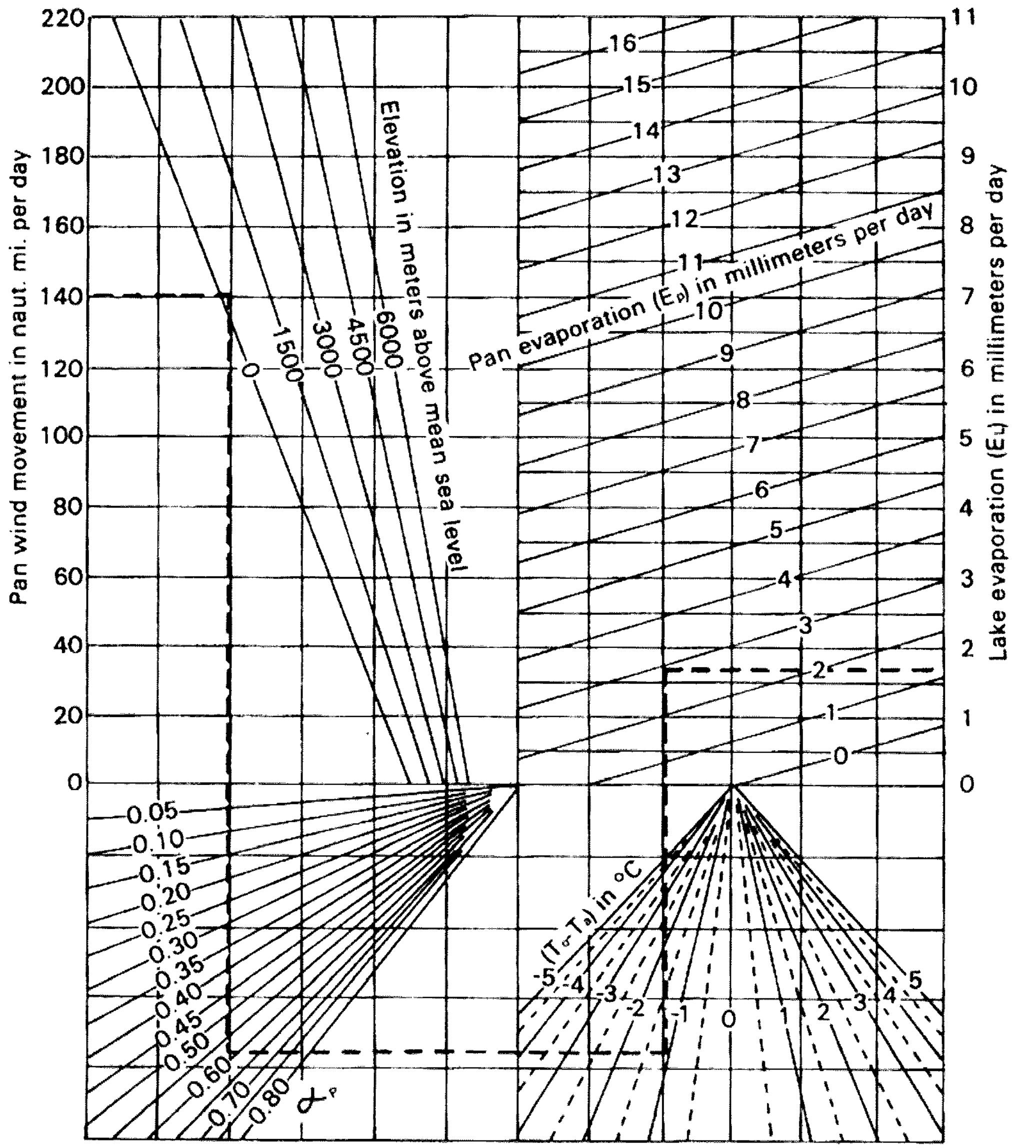

Figure C-7-2. Shallow lake evaporation as a function of Class A Pan evaporation and heat transfer through the Pan in metric units per day (NOAA National Weather Service, 2004).

Surface water temperature = T_o= air temperature (T_a) minus 1°C

T_o - T_a = -1°C

Q = 950 m^3/d (0.25 mgd)

Influent BOD_5 = 150 mg/L

Seepage = 0.76 mm/d (0.2 in/wk).

Seepage is prohibited in some areas. State agency wastewater facility standards may require the pond bottom be sealed with an impervious liner, reducing seepage to zero. Elevation = 300 m (980 ft) above mean sea level (MSL).

Requirements:

Size a complete retention wastewater pond with no overflow for the given geographic area. Specify the following:

$$\text{Area, } A = \frac{Q\ (365\ \text{d/yr})}{d - (\text{Annual Precip.} - \text{Annual Evap.} - \text{Annual Seepage})}$$

Surface area, A
Depth, d
Length, L
Width, W

Solution
The design procedure consists of the following steps:

1. Using the data in Table C-7-1 with Figure C-7-1 (elevation = 305 m) and Figure C-7-2, determine the mean monthly evaporation from the pond. The calculation of pond evaporation is shown on the figures by dashed lines. The results are presented in Table C-7-2.

2. Using the data presented in Tables C-7-1 and C-7-2, calculate the area required for an assumed mean depth for one year of operation under design conditions. The mean depth (d) may range from 0.1 - 1.5 m (0.3 - 5.0 ft). The mean depth is usually near 1 m (3 ft).

3. Use the A value determined in step 2 to calculate the stage of the pond at the end of each month of operation during the design year.

4. Calculate the monthly stage of the pond under average conditions. If the pond is designed so that it never overflows, the average yearly evaporation and seepage must exceed the inflow and precipitation entering the pond.

5. Repeat steps 2 and 3 until a satisfactory pond depth is obtained.

Table C-7-1. Climatological Data (National Climatic Data Center, 2004).

CLIMATOLOGICAL DATA FOR CALCULATING POND EVAPORATION AND PRECIPITATION						
Month (Days in Month)	Mean Precipitation	Air Temp.	Wind Speed	Minimum 10-Year Pan Evaporation		Mean 10-Year Pan Evaporation
	mm/month	˚C	*Kts*, day[a]	mm/month	mm/day	mm/month
Jan (31)	12.3	12.4	140.4	87.5	2.82	1050
Feb (28)	12.1	14.9	148.7	130.5	4.66	177.5
Mar (31)	10.1	17.7	154.2	198.2	6.39	220.0
Apr (30)	4.2	21.0	157.0	238.1	7.94	271.4
May (31)	2.9	24.6	154.2	332.0	10.71	365.2
Jun (30)	2.2	29.3	134.9	374.4	12.48	423.1

CLIMATOLOGICAL DATA FOR CALCULATING POND EVAPORATION AND PRECIPITATION						
Month (Days in Month)	Mean Precipitation	Air Temp.	Wind Speed	Minimum 10-Year Pan Evaporation		Mean 10-Year Pan Evaporation
	mm/month	˚C	*Kts*, day[a]	mm/month	mm/day	mm/month
Jul (31)	6.8	32.8	140.4	416.0	13.42	449.3
Aug (31)	15.9	32.4	134.9	347.8	11.22	389.5
Sep (30)	10.6	29.1	115.6	278.5	9.28	323.1
Oct (31)	7.8	22.6	110.1	210.4	6.82	219.9
Nov (30)	8.3	16.8	126.6	137.4	4.58	163.5
Dec (31)	14.6	12.9	143.2	95.2	3.07	131.4
Total	**107.8**			**2847.0**		**3238.9**

[a]*Kts* = knots = total of nautical miles/hr of wind per day.

Use A = 142,259 m^2 (35 ac) to calculate the stage of the pond at the end of each month of operation. Table C-7-2 contains a summary of the results of this procedure for the design year of operation, assuming the pond is empty at the beginning of the year.

An examination of the pond stage results in Table C-7-3 shows that the maximum depth of water in the pond during the design year (conservative design data) would be 0.62 m (2 ft) plus the depth at the beginning of the design year. The pond stage under average conditions is shown in Table C-7-4. Average evaporation and seepage are within 5 percent of inflow and precipitation. Assuming that several average years would occur in sequence, there would be a small accumulation of water in the pond. Because of the imprecise methods available to predict the sequence of occurrence of the design year, maximum and average years, the pond surface area of 142,259 m^2 is large enough to prevent overflow of the pond.

The depth of complete retention ponds is limited only by ground water conditions, and evaporation rates and cost. Generally maximum depths range from 1 to 3 m (3 to 10 ft) with a freeboard of 0.6 m (2 ft). The maximum depth required will depend on the time of year that filling of the pond begins and the initial depth of water in the pond. It is not possible to predict accurately the water stage in the pond, therefore, it is necessary to exercise good judgment based upon the constraints at particular locations. Estimates beyond the average- and design-year conditions can be made by analyzing historical data for the site, but this is still no guarantee of accuracy.

The water depth in the pond after one year of operation under design conditions will be equal to the mean depth plus the depth of water at the beginning of the year. During certain months of the year, the depth may exceed the mean depth when filling of the pond occurs at the beginning of the wet season. Three different starting dates are shown in Table C-7-3 to illustrate this point. Using the abovementioned procedure, it is possible for the design engineer to estimate the stage of the pond under as many conditions as considered necessary.

A maximum depth of 1.2 m (4 ft) would be adequate to avoid overflow from the pond by providing storage for 5 average years (Table C-7-3) and a design year in sequence. It is unlikely that 5 average years of evaporation would preceed the design year. The pond L and W values are calculated from the surface area A. No restrictions are imposed on the length to width ratio. Also, the need to divide the pond volume to enhance hydraulic characteristics is eliminated. The most economical design consists of a single pond, provided the system can be isolated enough to avoid complaints about odors when solids decompose on exposed slopes.

$$A = LW$$
$$L = W = A^{(1/2)} = 142{,}259 \text{ m}^{(1/2)} = 377 \text{ m } (1{,}237 \text{ ft})$$

Summary:

A = 142259 m^2 (35 ac)
W = 377 m (1237 ft)
V = 170800 m^2 (45 MG)
d = 1.2 m (3.9 ft)

Table C-7-2. Calculated Pond Evaporation Data for Design and Average Conditions

Month	αp	Design Pond Evaporation		Average Pond Evaporation
		mm/day	mm/month	mm/month
January	0.58	1.7	53	61
February	0.62	3.0	84	110
March	0.64	4.1	127	141
April	0.66	5.2	156	174
May	0.71	7.2	223	259
June	0.74	8.4	252	313
July	0.77	9.0	279	323
August	0.77	7.4	229	300
September	0.73	6.2	186	235
October	0.58	4.4	136	127
November	0.62	2.9	87	101
December	0.58	1.8	56	76
TOTAL			1868	2220

Table C-7-3. Volume and Stage of Pond at Monthly Intervals for Design Conditions and A = 142,259 m^2/Month

	No. Days in Month	Inflow + Precipitation[a] m^3	Evaporation +Seepage[b] m^3	Storage Volume[c] m^3	Pond Stage[d] Depth (m)
Starting Date 1					
September	30	29888	29704	184	0
October	31	30436	22699	7921	0.06
November	30	29561	15620	21862	0.15
December	31	31403	11318	41947	0.29
January	31	31076	10891	62131	0.44
February	28	28209	14977	75363	0.53
March	31	30763	21419	84708	0.60
April	30	28977	25436	88249	0.62
May	31	29739	35075	82912	0.58
June	30	28693	39093	72513	0.51
July	31	30293	43042	59764	0.42
August	31	31588	35929	55423	0.39
Total		360626	305202		
Starting Date 2					
January	31	31076	10891	20184	0.14
February	28	28209	14977	33417	0.23
March	31	30763	21419	42761	0.30
April	30	28977	25436	46303	0.33
May	31	29739	35075	40966	0.29
June	30	28693	39093	30566	0.21
July	31	30293	43042	17817	0.13
August	31	31588	35929	13476	0.09
September	30	29888	29704	13661	0.10
October	31	30436	22699	21397	0.15
November	30	29561	15620	35338	0.25
December	31	31403	11318	55423	0.39
Starting Date 3					
December	31	31403	11318	20085	0.14
January	31	31076	10891	40269	0.28
February	28	28209	14977	53502	0.38
March	31	30763	21419	62846	0.44
April	30	28977	25436	66387	0.47
May	31	29739	35075	61051	0.43
June	30	28693	39093	50651	0.36
July	31	30293	43042	37902	0.27
August	31	31588	35929	33561	0.24
September	30	29888	29704	33746	0.24
October	31	30436	22699	41482	0.29
November	30	29561	15620	55423	0.39

[a] Inflow = *Q*(days/mo): precip. = (monthly precip)(*A*)
[b] Seepage = 0.00076 m/d (days/mo)(*A*); evaporation = monthly evap.(*A*)
[c] Storage *V* = cum. sum of (inflow + precip.) – (evap. + seepage)
[d] Pond Stage = storage *V*/*A*

Table C-7-4. Volume and Stage of Pond at Monthly Intervals for Average Conditions and A = 142,259 m^2

Month Average Year	No. Days in Month	Inflow + Precipitation (m^3)	Evaporation + Seepage (m^3)	Storage Volume (m^3)	Pond Stage Depth (m)
September	30	29888	36674	6786	0
October	31	30436	21561	8875	0.06
November	30	29561	17612	20824	0.15
December	31	31403	14192	38035	0.27
January	31	31076	12385	56726	0.40
February	28	28209	18690	66245	0.47
March	31	30763	23410	73598	0.52
April	30	28977	28708	73868	0.52
May	31	29739	40197	63409	0.45
June	30	28693	47771	44332	0.31
July	31	30293	50582	24044	0.17
August	31	31588	45887	9744	0.07
Total		360626	357668		

FROM CHAPTER 8 COST ESTIMATES: Spread Sheets for Estimating the Cost of Intermittent Sand and Rock Filters, Table 8-1a, b and c; Intermittent Sand Filter Cost Estimation Procedure, Table 8-2a, b and c; and Intermittent Rock Filter Cost Estimation Procedure. ***Table 8-1a. Spreadsheet for estimating costs. (Input Design Values are shaded)***

Inside and outside Slope of dikes =	***Width of top of dike =***	8 ft
3H to 1V= 3		
Design Flow Rate (DFR) =	0.3 mgd	
Design Loading Rate =	0.3 mg/ac/d	
Surface Area		
Required/filter =	1 ac	
At least one filter out of service for cleaning =		
Total number of filters needed	3	
Surface area of sand required/filter =	1 ac	
	43560 sq ft	
L to W = 2 to 1 =	2	
Wwater surface =	147.58 ft	
Lwater surface =	295.16 ft	
Sand depth	3 ft	
Volume of sand/filter	119,050 cu ft	
	4,409 cu yd	
Depth of underdrain graded gravel =	1 ft	
Volume of dike		
Soil/Dike-ft =	191.2 ft^3/ft of dike	
Width to center of dike =	168.14 ft	
Length to center of dike =	315.72 ft	

Table 8-1a (cont.)

If common walls are used, reduce number of widths or lengths (Input Design Values are shaded)

Number of common dike widths	2	*Dike widths to be reduced =*	336.29 ft
Number of common dike lengths=	0	*Dike lengths to be reduced =*	0.00 ft

Total lengths to be reduced =	336.29 ft
Corrected total lengths of dikes/filter =	631.4 ft
Total Volume of dikes/filter =	120,725.78 cu ft
	4471.33 cu yd
Freeboard Volume 3 X DFR =	2.76
Wbottom =	129.6 ft
Lbottom =	277.2 ft
Surface Area of bottom of filter sand =	35914.6537 3sq ft
Volume of underdrain graded gravel=	34,706 cu ft
	1,285 cu yd
Lateral Under Drain Piping Diameter =	10 in
Center Drain Pipe Diameter =	12 in
Length of Center Drain Pipe=	227 ft
Spacing of Laterals	20 ft OC
Number of Laterals	14
Length of Laterals	1796 ft
Pump or dosing siphon sump	1
Valves/filter	2
Synthetic or soil liner/ft of dike	327.92
Total surface area of liner/filter	207,062

Table 8-1a (cont.)

Cost Summary *Input values are shaded*					
Item	**No. Require d per filter**	**Quantit y**	**Unit**	**Unit Cost**	**Total Cost**
Filter Sand	1	4409	cu yd	$15.00	$66,138.88
Under drain Gravel	1	1285	cu yd	$10.00	$12,854.23
Dike Earth Work	1	4471.3	cu yd	$2.00	$8,942.65
Lateral Pipe 10 in	1	1796	ft	$10.00	$17,957.33
Main Drain 12 in	1	277	ft	$17.00	$4,711.74
Liner and protection	1	207,062	sq ft	$1.75	$362,358.45
Valves 12 in	2	2	per	$1,000.00	$2,000.00
Pump or dosing siphon with sump	1	1	per	$5,000.00	$5,000.00
Total cost for one filter =					$479,963.00
Total cost for required number of filters =					$1,439,890.00

Table 8-1b.

Updated Cost for Intermittent Sand Filters from 1983 EPA Design Manual						
Year	**Location**	**Design Flow**	**Loading Rate**	**ENR CC Index**	**ENR CC Index for 2006 Kansas City**	**Capital Cost**
1975	Huntington, UT	0.300	300,000	2508.98	8704.67	418,745
1975	Ailey, GA	0.080	600,000	2508.98	8704.67	62,093
1975	Moriarty, NM	0.200	300,000	2508.98	8704.67	94,033
1978	White Bird, ID	0.030	400,000	3039.64	8704.67	21,160
1976	Mt. Shasta, CA	0.700	700,000	2687.10	8704.67	512,315

Table 8-1c.

Location	**Capital Corrected Costs 2006 US$**	**\$/mgd Year of Construction**	**\$/mgd Corrected for 2006**
Huntington, UT	4,452,796	1,395,817	4,842,655
Ailey, GA	215,426	776,163	2,692823
Moriarty, NM	326,239	470,165	1,631,193
White Bird, ID	60,596	705,333	2,019,875
Mt. Shasta, CA	1,659,608	731,879	2,370,869

Table 8-2 a,b,c. Intermittent Rock Filter Cost Estimation Procedure

Table 8-2a. Spreadsheet for estimating costs. ***(Input Design Values are shaded)***

Design Flow Rate (DFR) = 10688 ft^3/d	**0.08 mgd**	
Design Loading Rate =	**0.4 ft^3/ft^3 Rock**	
Volume of Rock Required at WS =	**26720 ft^3**	
Top of Dike =	**8 ft**	
Inside and Outside Slopes of Dikes 3H to 1V =	**3**	
Total number of filters needed =	**1**	
Water Depth =	**6 ft**	

Trial and Error calculation of L and W must be completed before proceeding

Wwater surface=	**83.90 ft**	
Lwater surface =	**83.90 ft**	
When E56 and F56 agree, dimensions are correct	**7039**	7054ft^2 =
	Water Surface Area	
Rock Depth above Water Surface	**1.00 ft**	
Design Flow Rate (DFR) =	**0.3 mgd**	
Wwater surface	**89.90 ft**	
Lrock surface =	**89.90 ft**	
Rock Depth =	7.00 ft	
Volume of rock/filter =	34,259 cu ft	
	1,269 cu yd	
Volume of Dike Soil/Dike-ft =	203.0 ft^3/ft of dike	
Width to Center of Dike =	93.90 ft	
Length to Center of Dike =	93.90 ft	
Total Length of Dikes =	375.6 ft	
Total Volume of Dikes =	76,246.80 cu ft	
	2,823.96 yd^3	

Table 8.2a (con't)

Lbottom =	47.90 ft
Wbottom =	47.90 ft
Surface Area of Bottom of Filter=	2,294.41 ft^2
Central Influent Pipe Diameter =	**12 in**
Length of Center Drain Pipe=	48 ft
Length of Two Effluent Channels=	95.8 ft
Pump or transfer structure =	**1**
Valves/filter =	**2**
Synthetic or Soil Liner/ft of Dike=	52.27 ft^2
Total surface area of liner/filter=	21,928 ft^2

Table 8-2b.

CONSTRUCTION COST SUMMARY (INPUT VALUES ARE SHADED)					
Item	**No. Required per filter**	**Quantity**	**Unit**	**Unit Cost**	**Total Cost**
Rock	1	1,269	cu yd	$10.00	$2,688.69
Dike earthwork	1	2,824	cu yd	$2.00	$5,647.91
Central influent pipe 12#	1	48	ft	$10.00	$479.00
Effluent trough and weir	1	96	ft	$17.00	$1,628.60
Liner and protection	1	21,928	sq ft	$1.75	$38,373.53
Valves 12 in	2	2	per	$1,000.00	$2,000.00
Pump with transfer structure	1	1	per	$5,000.00	$5,000.00
Total cost for one filter =					$65,818
Total cost for required number of filters =					$65,818

Table 8-2b (cont.)

Construction Cost Summary (Input values are shaped)					
Item	**No. Required per filter**	**Quantity**	**Unit**	**Unit Cost**	**Total Cost**
Rock	1	1,269	cu yd	$10.00	$2,688.69
Dike earthwork	1	2,824	cu yd	$2.00	$5,647.91
Central influent pipe 12#	1	48	ft	$10.00	$479.00
Effluent trough and weir	1	96	ft	$17.00	$1,628.60
Liner and protection	1	21,928	sq ft	$1.75	$38,373.53
Valves 12 in	2	2	per	$1,000.00	$2,000.00
Pump with transfer structure	1	1	per	$5,000.00	$5,000.00
Total cost for one filter =					$65,818
Total cost for required number of filters =					$65,818

Table 8-2c.

UPDATED COST FOR ROCK FILTERS FROM 1983 EPA DESIGN MANUAL						
Year	**Location**	**Design Flow Rate, mgd**	**Loading Rate ft^3/ft^3 of rock**	**ENR CC Index**	**ENR CC Index for 2006 Kansas City**	**Capital Cost Year of Const. (US$)**
1974	Wardell, MO	0.080	0.40*	2308.25	8704.67	12,900.00
1974	Delta, CA	0.080	0.40*	2308.25	8704.67	15,230.00
1976	California, MO	0.360	0.40	2308.25	8704.67	57,843.00
1975	Luxemburg, WI	0.400	0.40	2687.10	8704.67	46,708.00
	Veneta, OR	0.220	0.27	2508.98	8704.67	41,720.00
		Capital Costs Connected to 2006 (US$)		**$/mgd Year of Construction**	**$/mgd 2006**	
	Wardell, MO	$48,647.35		161,250	608,092	65,818.00
	Delta, CA	$57,434.04		190,375	717,926	65,818.00
	California, MO	$218,132.45		160,675	605,923	165,032.00
	Luxemburg, WI	$151,307.26		116,770	378,268	178,659.00
	Veneta, OR	$144,743.61		189,636	657,926	154,190.00

APPENDIX D

Case Studies

CASE STUDIES

These studies are presented to provide a sense of the range of challenges that wastewater pond systems designers and operators have faced over the years and some of the solutions that have been put in place. We include examples of systems from different parts of the country, which must comply with similar regulations though they live in different environmental conditions.

New Hampshire

Rockland

New Hampshire treatment ponds generally operate with a permit to discharge effluent to ambient water during the winter months (November 1 through April 30) and spraying on irrigation fields during the summer. Ponds designed to meet BOD/TSS are increasingly required to meet NH_3 limits in the winter. Studies measured the base level of NH_3 coming into the ponds and results suggested that changing the discharge schedule would reduce the number of NH_3 limit violations.

Kansas

The Kansas Department of Health and Environment published its Surface Water Nutrient Reduction Plan in December 2004. Referring to a study it conducted in 2002, the KDHE reaffirmed its support of wastewater treatment ponds as the only feasible treatment technology for many small Kansas towns and attested to their effectiveness in removing nutrients (T*N* by 65% and T*P* by 55%).

California

Los Banos

A small city (population 40,000) in the Central Valley of California was a candidate for a study using solar-powered water circulators (Solarbee®) to evaluate effectiveness and potential savings in energy from this source if new water quality standards are added to its permit. The study provided support for the effectiveness of the treatment system. Another study examined the impact of the release of effluent on agricultural fields over time.

Arkansas

The Wastewater Treatment Ponds in Arkadelphia, AK have been in operation since 1968. In 1994, with the addition of a small Lem-Tech duckweed system after the last pond, the system consistently meets discharge limits year round.

New Hampshire

Meeting Ammonia Requirements by Reviewing Nutrient Values against Discharge Operation Schedules

Summary

A wastewater treatment pond system consisting of two facultative ponds (2.6 MG) and one storage pond (18 MG) serving a county jail and nursing home were constructed in 1990 and designed to meet BOD_5/TSS of 30/30 mg/L. In 1996, NH_3 limits of 6.1 mg/L monthly average with a 12.2 daily maximum. The storage pond was permitted to discharge treated effluent to a small stream from October to April. Total Kjeldahl *N* (TK*N*) and NH_3 in the influent increased from 28 mg/L to 45 and 8 mg/L to 21, respectively from 1996 to 2010. Water conservation and use of kitchen disposals is thought to be the reason for the increase. An operational decision to change the timing of the initiation of discharge from January to November brought the facility into compliance for NH_3.

Introduction

Rockingham County Complex, Brentwood, New Hampshire operates a three-cell aerated pond system dedicated to serving a county jail and nursing home. The ponds were constructed in 1990 and were designed to meet typical secondary treatment standards of 30/30 mg/l for BOD_5 and TSS. The plant was originally designed to treat a flow of 0.67 ML/d (0.178 mg/D and a BOD_5 load of 215 kg/d (475 lbs/d). Current flow and loadings are 0.26 ML/d (0.07 mg/d) and 73 kg/d (160 lbs)/d BOD_5 (Table 1). No expansions are anticipated and with water conservation measures enacted over the years, it is unlikely design conditions will be met in the foreseeable future.

Table 1. Discharge Requirements, Rockingham County Complex NH

Design flow	Act. flow	BOD/ TSS (mg/L)	Discharge season	NH_3 limit (mg/L)
0.67 ML/d 0.178 mg/d	0.26 ML/ d 0.07 mg/ d	30/30	October 1 to April 30	6.1 /mo. ave. 12.2 max/d

The County has an NPDES permit that allows discharge to a very small brook from October 1st through April 30th at an implied flow of 0.085 MGD. They also have a groundwater discharge permit allowing spray irrigation from May 1st through October 31st. The majority of biological treatment takes place in the first two ponds, each having a volume of 2.6 MG. Treated flow is then transferred to an 18 MG storage pond, where it can be held until discharged to the brook or spray irrigated. Due to the design of the valving and piping arrangements, the operator is limited to holding and treating an entire week's worth of flow in the first two ponds during the week, and then transferring that volume of water to the storage pond during the weekend.

In 1997, the County's NPDES permit was reissued with NH_3 limits to the brook from October 1st through April 30th of 6.1 mg/l as a monthly average and 12.2 mg/l for a max day. This presented an immediate problem as the system was not designed to remove NH_3. Eliminating discharge to the brook altogether would require building another large holding pond and expanding the spray irrigation sites, neither of which was deemed feasible at the time.

Results

An initial one year study performed in '96 and '97 showed that the system was capable of producing winter effluent concentrations on the order of 5 mg/l TKN and 3.3 mg/l NH_3 at water temperatures < 3°C. Summer NH_3 levels would go as low as 0.2 mg/l. Influent TKN at the time averaged around 28 mg/l and NH_3 8 mg/l. Biological nitrification was determined to be the primary method for NH_3 reduction as demonstrated by the production of NO_3^-. Substantial nitrification occurred in the first pond and was brought to completion in the second pond. This continued throughout the summer and well into the fall. The unusually low winter effluent concentrations are thought to be primarily due to dilution in the storage pond.

By October 31st, the end of spray season, the storage pond contained a volume of 4.8 MG of fully nitrified effluent. The second pond continued to support nitrification well into December, until the temperatures decreased to the level of nitrification inhibition and NH_3 concentrations increased. As a more NH_3 rich water is transferred to the holding pond, the NH_3 concentration in the pond and, ultimately, the final effluent, gradually increases to a concentration potentially exceeding 6.1 mg/l. That level was not reached during the initial study.

The plant performed well for the first several seasons under the new permit limits. Beginning in the winter of 2001, however, and lasting through 2005, winter monthly average violations were experienced on a regular basis. January, usually a good month, averaged around 4.5 mg/l NH_3. February, March and April averages ranged from 6 to 11 mg/l. There were no violations during the winters of '06 and '07. The violations resumed, however, in '08 and '09. Another study was undertaken to try to determine the cause.

The study showed that the flow and BOD_5 loadings remained the same but that the N load had increased considerably. Influent TKN now averages 45 mg/l, up from 28 mg/l, and influent NH_3 increased from an average of 8 mg/l in 1996 to 21 mg/l in 2009. It is believed that water conservation measures, in conjunction with the heavy use of garbage disposals in the kitchen area, have led to the increased N loading of the system (Table 2).

Table 2. TKN and NH_3 in Pond Influent, 1996 and 2010

Influent	TKN (mg/L)	NH_3 (mg/L)
1996	28	8
2010	45	21

The new study showed that the ponds did continue to nitrify, but that the process was now confined to the second pond, and at a slower rate and beginning much later in the summer. As a result, the ponds were unable to handle the increased N load within the system's detention time and resulted in the passing of NH_3 to the storage pond and effluent. Dissolved oxygen levels were also found to be too low in the first pond (often < 0.2 mg/l). Insufficient aeration in the first pond could lead to the passing of BOD_5 to the second pond, potentially delaying the onset of nitrification. BOD_5 must be removed before nitrification can proceed. A review of operational data pointed out the fact that after spray season ends, the operators held all flow for the entire months of November and December and then discharge to the brook in January, February and March, precisely when NH_3 concentrations would be expected to be the highest.

During the first year of operating under this plan, discharge to the brook for the months of November and December 2009 resulted in average *NH_3* concentrations of 1.77 and 1.29 mg/l, respectively. From January 2010 through April, all flow was held until the start of spray season. Supplemental aeration in the first pond has not yet been implemented.

Discussion

The major recommendation of this study was to maximize discharge to the brook, within permit limits, during the months when the *NH_3* was expected to be low, mainly November, December and the first half of January, and rely on holding and spray irrigation for the remainder of the year. This plan, being weather dependent, requires careful planning by the operator to maximize the storage volume of the holding pond to ensure there is adequate room for storage from mid-January to the start of spray season. The study also recommended adding supplemental aeration to the first pond in order to maximize BOD_5 removal there, which, in theory, should allow nitrification to proceed faster and be of longer duration in the second pond.

Conclusion

This case study illustrates the benefits of system-wide monitoring, close evaluation of flows and loadings, and assessing plant operations to determine the potential for a pond to nitrify. It is unlikely that a continuous flow through pond would meet limits of < 6 mg/l in a cold climate without further enhancements, but in an under-loaded pond where the detention time and discharge periods may be manipulated, this may be possible. Further study of those variables may be warranted.

Report by Wes Ripple, New Hampshire Department of Environmental Services. The plant operator is Mark Pettengill (603-679-5335).

The State of Kansas

The Case for Ponds in Anticipation of More Stringent Nutrient Limits for Wastewater Treatment System Effluents

Given overall good, consistent treatment and low cost, the Kansas Department of Health and Environment has encouraged communities to build ponds for wastewater treatment. As a result, nearly 80% of all municipal wastewater treatment in the State of Kansas is provided by wastewater pond systems.

In 1994, the Kansas Department of Health and Environment adopted water quality standards that were significantly increased in scope and stringency. Language was indicating that wastewater treatment ponds would be able to meet these standards was not approved by US EPA Region 5. In 1999, the KDHE adopted revised standards that eliminated the reference to ponds and spelled out how ponds would be addressed in the NPDES permitting process. This included a study of pond performance.

Effluent samples from eighteen facilities built in accordance with KDHE's Minimum Standards of Design for Water Pollution Control Facilities were analyzed, including BOD_5, $SBOD_5$, $CBOD_5$, NH_3, TK*N*, NO_3, T*P*, dissolved *P*, fecal coliform and *p*H. Overall, the data indicated that pond systems provide consistently good treatment for $CBOD_5$, *N* and bacteria. Increase in total BOD_5 in late summer, correlating to increase of NH_3 and organic *N*, is thought to be due to increasing anaerobic conditions in the sediment, leading to microorganism die-off. KDHE has been evaluating maintenance options to reduce solids in pond effluent.

Similarly, ponds are shown to provide good quality year-round disinfection of wastewater. During the recreation season, best fit curves indicated that fecal coliform will be <200 MPN/100 mL 50% of the time, with 100% of the samples <1700 MPN/100 mL. In the winter, 55% of the samples will be <200 MPN/100 mL and 90% will be <2000 MPN/100 mL. An increase in fecal coliform seen in the late summer correlates fairly well with the increase in *N*.

The issue of greatest concern for the viability of the pond systems in Kansas is the adoption of nutrient criteria by the EPA. The Agency's approach is to develop criteria by ecoregion; Kansas is located in five of those regions. The EPA Region 7 Regional Technical Advisory Group's task is to identify rivers impacted by nutrients, collect water quality data from those rivers, select the upper 25^{th} percentile of the nutrient values as ecoregion reference conditions. Where insufficient data exist, the lower 25^{th} percentile of the available data from all sites will be used. RTAG recommended criteria for all lakes and reservoirs in Kansas, Iowa, Nebraska and Missouri are 0.70 mg/L (T*N*); 35 µg/L (T*P*); and 8 µg/L (chl α). These criteria, it is believed, will be of concern to all types of wastewater treatment facilities, not only pond systems.

The KDHE published the Surface Water Nutrient Reduction Plan in December 2004. It reaffirms its support of wastewater treatment ponds as the only feasible treatment technology for many small Kansas towns and attests to their effectiveness in removing nutrients (T*N* by 65% and T*P* by 55%).

The information for this case study is taken from Tate, M.B. et al. 2002 and the Kansas Department of Health and Environment, 2004.

Los Banos, CA

Looking at potential for reducing energy costs and long-term impacts of irrigating with pond effluent

Los Banos is community of 40,000 people located in the Central Valley of California. The wastewater treatment facility consists of 234 ha (354 ac) of treatment and storage ponds (167.4 ha/ 354 ac) and spray fields (67.2 ha/ 166 ac). Several studies have been initiated to understand baseline conditions before plans are developed for expansion. Unfavorable economic conditions have slowed the rate of growth, which affected at least one of the studies.

Pacific Gas & Electric, working with the California Wastewater Process Optimization Program (CalPOP), asked the City of Los Banos if it would participate in a study to evaluate and document potential energy savings using Solarbee® technology. Previous projects typically involved the replacement of standard mechanical mixing and aeration systems with Solarbee® units; the Los Banos project involved the introduction of the technology as an alternative to introducing standard mechanical mixing and aeration systems to a large-scale facultative pond treatment system. While the potential savings is speculative, the deployment of the Solarbee® aerators in one treatment and one storage pond did demonstrate their effectiveness in changing the hydrodynamics (reducing stratification of temperature and dissolved oxygen) and provided water quality information.

Table 3. Los Banos Wastewater Treatment System Process Features

Treatment Process Characteristics	**Value**	
Average Influent Flow	3.5 mg/d	
Average Influent BOD	535 mg/L	
Assumed Influent NH_3	25 mg/L	
Average Recycled Flow (treatment pond effluent) to Plant Headworks	19.2 mg/d	
Average Recycled BOD (treatment pond effluent) to Plant Headworks	70 mg/L	
Combined (Influent + recycled) BOD_5 to Ponds	140 mg/L	
Volume/Surface Area	**Cm(K)/Ha**	**(MG/Ac)**
Treatment Pond 1&2	471/34.4	(124.5 / 85)
Treatment Pond 5	651/28.3	(172 / 70)
Treatment Pond 6	738/28. 3	(195 / 70)
Storage Pond 3	307/12.9	(81 / 42)
Storage Pond 4	609/36.4	(161 /90)
Storage Pond 7	723/27.1	(190.5 / 67)

The interpretation of water quality results was complicated by the fact that the system process includes constant effluent recycling and redistribution to ponds, as well as differences in pond depths, pond internal loadings, and detention times. Project analysis indicated that the Solarbee®s operated according to their design parameters and met specifications. Specifically, the water column characteristics of temperature, DO, *p*H, and conductivity showed that the ponds with Solarbee®s were better mixed, less stratified, cooler, and had significantly better O_2 profiles than the control ponds. It was concluded that the installation of Solarbee® aerators would be a reasonable alternative to the mechanical aeration.

Table 3 provides a facility process summary that presents applicable information collected as part of the pre-project analysis for the treatment facility. Based on influent data from January 2007 to December 2007, the current plant influent loading is 3.5 mgd flow with a yearly average BOD_5 concentration is 535 mg/I. However, as is common in plants with food processing influent flows, there is a sustained peak of 600 mg/I influent BOD_5 for two months during the year.

Ponds 1 through 7 were monitored for DO, BOD_5, TSS, EC and temperature. Supplemental testing included $CBOD_5$, NH_3, TKN, $NO_2^=$ and NO_3^- (the results are not presented here).

While the comparison of $CBOD_5$ and BOD_5 between ponds gave limited information, the ratio of BOD_5 to $CBOD_5$, an indicator of the N and non-organic O_2 demand portion, was fairly consistent in all ponds. In addition, the facility is achieving $CBOD_5$ levels that are non-detect in Ponds 6 and 7 (Table 4). ("The non-detect CBOD results are outstanding for any pond-based system anywhere in the country. Most of the credit goes to a well-maintained and well-operated system operated by dedicated staff…")

Table 4. $CBOD_5$ (mg/L) in Ponds with Solarbee® Aerators vs. Control

	Treatment Ponds		Storage Ponds	
	Solar Bee®	Control	Solar Bee®	Control
	Pond 1	Pond 2	Pond 6	Pond 7
Ave	32	26	15	13
Min	22	13	2.5*	2.5*
Max	60	35	35	24

*One/half the detection limit. Actual value is non-detect.

Another study was conducted recently to look at the effect of spraying fields with Los Banos WWTP water (data not provided). The consultant evaluated water level and EC data from new and previously existing monitoring wells around the WWTP, as well as from a network of shallow piezometers maintained by the Central California Irrigation District. Results of this salinity study indicate that (1) unlined irrigation canals are likely influencing shallow groundwater quality, masking the regional salinity gradient, and (2) evapo-concentration may be locally concentrating salts in groundwater. The study also included installation of additional monitoring wells and quarterly sampling of these wells. It is anticipated that these data will be used to demonstrate that the City's wastewater management practices are not adversely impacting the groundwater.

This case study was prepared from *Solar-Powered Circulator Energy Assessment Project, Emerging Technologies Program, Pacific Gas & Electric (2008)* Quantum Energy Services & Technologies, Inc.; *Six Months Report Reviewing Phase One Solarbee® Pilot Program at the City of Los Banos Wastewater Treatment Facility* (2008) (*http://www.Solarbee.com*) and *Salinity Study for Wastewater Treatment Facilities Expansion in City of Los Banos* EKI Consultants (*http://www.ekiconsult.com*)

Arkadelphia, Arkansas

The City of Arkadelphia, located on the Ouachita River, with a current population of 11,000, put in a wastewater treatment pond system in 1968. Arkadelphia's wastewater treatment facility consists of 164 acres of oxidation ponds, with the final eleven acres in aquaculture (Lemna Process, see smallest pond in Figure 1). Discharge NPDES limits to the Ouachita River are 30 mg/L BOD_5 and 90 mg/L TSS. Average flow through the system is 1.9 MGD with current capability of treating 3.0 MGD.

Sludge was removed from the first pond in 1980. In 1994, a duck weed pond was added to the treatment train to provide consistent TSS, especially in the summer. The operators were advised that they would have to harvest the *Lemna*, using a harvester to break up the clumps of vegetation. In fact, they have never had to use the harvester, as the Lemna breaks up in the fall and decomposes, without causing a significant build up of sludge. The *Lemna* pond is partitioned into a grid system by a series of plastic enclosures. The infrastructure, including plastic sheeting, stainless steel pins, has not needed to be replaced in sixteen years of operation. An added bonus is the number of species of birds visiting the *Lemna* pond to eat the insects that are found there.

Report based on interviews and website information (www.cityorarkadelphia.com).

Figure 1. Arkadelphia Wastewater Treatment Ponds with Lemna Process (smallest pond).

APPENDIX E

Troubleshooting

APPENDIX E

TROUBLESHOOTING

Table 9-5. Common Problems in Wastewater Treatment Pond Operation (Richard and Bowman 1991)

Problem	Possible Causes	Possible Solutions
Odors	Organic overload	Increase aeration capacity.
	Poor aeration or mixing	After aerator run time, change or supplement type of aeration.
	Previous ice-covered ponds	Increase aerator run time, change type of aerator to eliminate ice over.
	Duckweed growth	Increase aerator run time, chemical treatment (Diquat), physical removal (harvest), add ducks or geese.
	Excess weed growth on pond banks harboring flies, and mosquitoes, trapping grease and organics.	Physical removal by pulling, mowing, burning or chemical treatment (Diquat). In winter, lower pond level and allow ice to freeze around weed stem. Increase water level.
Poor BOD Removal	Organic overloading	Increase aeration capacity.
	Short Circuiting	Improve inlet-outlet conditions, add baffles, add recirculation to improve mixing, add or improve aeration of ponds.
	Ice-covered ponds	Change or add aerator.
	Recent reduction in pond temperature	Increase hydraulic detention time.
	Algal bloom	Increase mechanical mixing; add physical shade (Aquashade, Photoblue), floating cover such as swimming pool cover, Styrofoam sheets or balls, duckweed cover. Addition of algal predator such as daphnia. Add chemicals (copper sulfate). Addition of constructed wetlands to polish effluent.
High Effluent TSS	Algal bloom	See algal bloom solution above.
	Excess pond mixing or short circuiting	See short circuiting solution above.
	Spring or Fall turnover	Add different types of aeration to eliminate stratification or add supplemental aeration, add recirculation.
	Excessive solids buildup in bottoms of ponds	Physically remove solids by pump or sludge barge; proper sludge disposal in conformance with State and Federal regulations.
Poor Fecal Coliform Removal	Chlorine residual too low or poor chlorine contact chamber design	Increase chorine feed rate, provide 40:1 1:w ratio, provide a minimum 30 minute detention time at peak flow.
	Increase in chlorine demanding substance in effluent (H_2S, NO_2)	Remove solids from chlorine contact chamber, increase chlorine feed rate.
	Water fowl contamination	Increase chlorine feed rate.
High pH	Algal stripping of carbon dioxide and bicarbonate alkalinity	See algal bloom solution above.
Low pH	Accumulation of organic sludge stuck in the “acid phase”	Physically remove sludge by pumping or sludge dredge.
	Extensive nitrification	Increase aerator run time, add recirculation.
	Organic overloading	Increase aeration capacity, add recirculation.
	Excessive daphnia growth	Increase aeration capacity or run time.

Table 9-6. Troubleshooting Test and Probable Causes (Richard and Bowman, 1991)

Probable Cause		Test
BOD_5 - high C BOD_5 – high TSS – moderate Filtered BOD_5 - high	Low DO	DO/DO Profile Microscopic Exam TSS/BOD_5 Ratio
BOD_5 - high C BOD_5 – low TSS – high pH - high	Low DO at night (Algae overgrowth or nitrification)	Filtered BOD DO Profile (early morning) Effluent Ammonia Test TSS/BOD_5 Ratio Microscopic Exam
BOD_5 – high C BOD_5 – high TSS – moderate Filtered BOD - high	Low DO Short circuiting Sludge Buildup (Soluble organics released from sludges)	DO/DO Profile Microscopic Exam TSS/BOD_5 Ratio

Table 9-7. Troubleshooting Tables (USEPA, 1977)
How to Control Water Weeds (USEPA, 1977)

Indicators/Observations	Probable Cause	Solutions
Weeds provide for burrowing animals cause short circuiting problems, stop wave action so that scum can collect and make a nice home for mosquitoes, and odors develop in the still area. Duckweed stops sunlight penetration and prevents wind action thus reducing the oxygen in the pond. Root penetration causes leaks in pond seal.	Poor circulation, maintenance insufficient water depth.	• Pull weeds by hand if new growth. • Mow weeds with a sickle bar mower. • Lower water level to expose weeds, then burn with gas burner. (Check with local fire department prior to burning.) • Allow the surface to freeze at a low water level, raise the water level and the floating ice will pull the weeds as it rises. (Large clumps of roots will leave holes in pond bottom; best results are obtained when weeds are young.) • Increase water depth to above tops of weeds. Use riprap. Caution: If weeds get started in the riprap they will be difficult to remove but can be sprayed with EPA approved herbicides. • To control duckweed, use rakes or push a board with a boat, then physically remove duckweed from pond.

Table 9-7 (Cont.)

How to Control Burrowing Animals (USEPA 1977)

Indicators/Observations	Probable Cause	Solutions
Burrowing animals must be controlled because of the damage they do to dikes. Rodents such as muskrats and nutria dig partially submerged tunnels into dikes. If the water level is raised, they will burrow further and may go on out the top thus weakening the dike.	Bank conditions that attract animals. High population in area adjacent to ponds.	• Remove food supply such as cattails and burr reed from ponds and adjacent areas. • Muskrats prefer a partially submerged tunnel, if the water level is raised it will extend the tunnel upward and if lowered sufficiently, it may abandon the tunnel completely. They may be discouraged by raising and lowering the level 6-8 inches over several weeks. • If problem persists, check with local game commission officer for approved methods of removal, such as live trapping, etc.

How to Control Dike Vegetation (Modified USEPA 1977)

Indicators/Observations	Probable Cause	Solutions
High weed growth, brush, trees and other vegetation provide nesting places for animals. This can cause weakening of the dike and presents an unsightly appearance. Also may reduce wind action on the pond.	Poor maintenance.	• Periodic mowing is the best method. Sow dikes with a mixture of fescue and blue grasses on the shore and short native grasses elsewhere. It is desirable to select a grass that will form a good sod and drive out tall weeds by binding the soil and "out compete" undesirable growth. • Spray with approved weed control chemicals. Note: Be sure to check with authorities. Some states do not allow chemical usage. All others require that chemicals be bio-degradable. • Some small animals, such as sheep, have been used. May increase fecal coliform, especially to the discharge cell. Not recommended with pond systems utilizing synthetic pond liner. Practice "rotation grazing" to prevent destroying individual species of grasses. An example schedule for rotation grazing in a 3-pond system would be: Graze each pond area for 2 months over a 6-month grazing season.

How to Control Scum (USEPA 1977)

Indicators/Observations	Probable Cause	Solutions
It is necessary to control sum formations to prevent odor problems and to eliminate breeding spots for mosquitoes. Also, sizeable floating rafts will reduce sunlight.	Pond bottom is turning over with sludge floating to the surface. Poor circulation and wind action. High amounts of grease and oil in influent will also cause scum.	• Use rakes, a portable pump to get a water jet or motor boats to break up scum formations. Broken scum usually sinks. Any remaining scum should be skimmed and disposed of by burial or hauled to landfill with approval of regulatory agency.

Table 9-7 (Cont.)

How to Control Odors (Modified USEPA 1977)

Indicators/Observations	Probable Cause	Solutions
Low pH (less than 6.5) and dissolved oxygen (less than 1 mg/L). Foul odors develop when algae die off.	Blue-green algae is an indicator of incomplete treatment, overloading and/or poor nutrient balance.	• Refer to Common causes of pond effluent noncompliance. • Apply chemical such as sodium nitrate. Application rate: 5-15 percent of sodium nitrate per pound of BOD on a pound for pound basis. Or apply 200 pounds sodium nitrate per million gallons. See literature for commercial products. Repeat at a reduced rate on succeeding days. Or use 100 pounds sodium nitrate per acre (112kg/ha) for first day, then 50 pounds per acre (56 kg/ha) per day thereafter if odors persist. Apply in the wake of a motor boat. • Install supplementary aeration such as floating aerators, caged aerators, or diffused aeration to provide mixing and oxygen. Daily trips over the pond area in a motor boat also helps. Note: Stirring the pond may cause odors to be worse for short periods but will reduce total length of odorous period. • Recirculate pond effluent to the pond influent to provide additional oxygen and to distribute the solids concentration. Recirculate on a 1 to 6 ratio. • Eliminate septic or high-strength industrial wastes.

How to Control Insects (Modified USEPA 1977)

Indicators/Observations	Probable Cause	Solutions
Insects present in area and larvae or insects present in pond water.	Poor circulation and maintenance.	• Keep pond clear of weeds and allow wave action on bank to prevent mosquitoes from hatching out. • Keep pond free from scum. • As for stocking fish, note that Gambusia do not eat mosquito larvae any faster than other small fish species. • Spray with EPA approved larvacide as a last resort. Check with state regulatory officials for approved chemicals.

Table 9-7 (Cont.)

How to Control Blue-Green Algae (Modified USEPA 1977)

Indicators/Observations	Probable Cause	Solutions
Low pH (less than 6.5) and dissolved oxygen (less than 1mg/L). Foul odors develop when algae die off.	Blue-green algae is an indicator of incomplete treatment, overloading and/or poor nutrient balance.	• Refer to common causes of pond effluent noncompliance. WARNING!: Prior to using copper sulfate, see explanation below. • Apply 3 applications of a solution of copper sulfate. ➢ If the total alkalinity is above 50 mg/L apply 1200 kg/m^3 (10 lbs/MG) of copper sulfate per million gallons in cell. ➢ If alkalinity is below 50 mg/L reduce the amount of copper sulfate to 600 kg/m^3 (5 lbs/MG. **Note:** Some states do not approve the use of copper sulfate since in concentrations greater than 1 mg/L it is toxic to certain organisms and fish. • Break up algal blooms by motor boat or a portable pump and hose. Motor boat motors should be air cooled as algae may plug up water cooled motors. **Important:** In the past copper sulfate has been used to control algae. It is recommended that the operator check with the regulatory agency to determine if a copper parameter must be added to the discharge permit. It should also be noted that prolonged use of copper sulfate may cause a buildup of copper in the benthic sludges making it difficult to dispose of the sludges when pond cleaning becomes necessary**.**

How to Obtain Best Algae Removal In The Effluent (Modified USEPA 1977)

Indicators/Observations	Probable Cause	Solutions
Most of the suspended solids present in a pond effluent are due to algae. Because many single-celled algae are motile and are also very small they are difficult to remove.	Weather or temperature conditions that favor particular population of algae.	• Draw off effluent from below the surface by use of a good baffling arrangement or variable depth draw off. • Use multiple ponds in series. • Check other chapters in the manual for latest algae control methods. • In some cases, alum dosages of 20mg/L have been used in final cells used for intermittent discharge to improve effluent quality. Dosages at or below this level are not toxic.

Table 9-7 (Cont.)

How to Correct Lightly Loaded Ponds (USEPA 1977)

Indicators/Observations	Probable Cause	Solutions
Lightly loaded ponds may produce filamentous algae and moss which limits sunlight penetration. These forms also tend to clog pond outlets.	Overdesign, low seasonal flow.	• Correct by increasing the loading by reducing the number of cells in use. • Use series operation.

How to Correct Overloading (Modified USEPA 1977)

Indicators/Observations	Probable Cause	Solutions
Overloading which results in incomplete treatment of the waste. Overloading problems can be detected by offensive odors, a yellow green or gray color. Lab tests showing low pH, DO, and excessive BOD loading per unit should also be considered.	Short circuiting, industrial wastes, poor design, infiltration, new construction (service area expansion), inadequate treatment and weather conditions.	• Bypass the cell and let it rest. • Use parallel operation. • Apply recirculation of pond effluent. • Look at possible short-circuiting. • Install supplementary aeration equipment.

How to Correct A Decreasing Trend In pH (USEPA 1977)

Indicators/Observations	Probable Cause	Solutions
pH controls the environment of algae types, as an example, the green chlorella needs a pH from 9.0 to 8.4 pH should be on the alkaline side, preferable about 8.0 to 8.4 Both pH and DO will vary throughout the day with lowest reading at sunrise and highest reading in the afternoon. Measure pH same time each day and plot on a graph.	A decreasing pH is followed by a drop in DO as the green algae die off. This is most often caused by overloading, long periods of adverse weather or higher animals, such as Daphnia, feeding on the algae.	• Bypass the cell and let it rest. • Use parallel operation. • Apply recirculation of pond effluent. • Check for possible short circuiting. • Install supplementary aeration equipment if problem is persistent and due to overloading. • Look for possible toxic or external causes of algae die-off and correct at source.

Table 9-7 (Cont.)

How to Correct A Low Dissolved Oxygen (DO) (Modified USEPA 1977)

Indicators/Observations	Probable Cause	Solutions
A low, continued downward trend in DO is indicative of possible impending anaerobic conditions and the cause of unpleasant odors. Treatment becomes less efficient.	Poor light penetration, low detention time, high BOD loading or toxic industrial wastes. (Daytime DO should drop below 1.0 mg/L during warm months.)	• Increase aerator running time. • Remove weeds such as duckweed if covering greater than 40 percent of the pond. • Reduce organic loading to primary cell(s) by going to parallel operation. • Add supplemental aeration (surface aerators, diffusers and/or daily operation of a motor boat). • Add recirculation by using a portable pump to return final effluent to the head works. • Apply sodium nitrate (see How to Control Odors for rate). Determine if overload is due to industrial source and remove it.

How to Correct Short Circuiting (Modified USEPA 1977)

Indicators/Observations	Probable Cause	Solutions
Odor problems low DO in part of the pond, anaerobic conditions and low pH found by checking values from various parts of the pond and noting on a plan of the pond. Difference of 100 percent to 200 percent may indicate short circuiting. After recording the reading for each location, the areas that are not receiving good circulation become evident. These areas are characterized by a low DO and pH.	Poor wind action due to trees or poor arrangement of inlet and outlet locations. May also be due to shape of pond, weed growth or irregular bottom.	• Cut trees and growth at least 150 m (500 ft) away from pond if in direction of prevailing wind. • Install baffling around inlet location to improve distribution. • Add recirculation to improve mixing. • Provide new inlet-outlet locations including multiple inlets and manifolds. • Clean out weeds. • Fill in irregular bottoms. • Add directional surface mixers or aerators to mix and retard flow.

Table 9-7 (Cont.)

How to Correct Anaerobic Conditions (USEPA 1977)

Indicators/Observations	Probable Cause	Solutions
Facultative pond that turned anaerobic resulting in high BOD, suspended solids and scum in the effluent in continuous discharge ponds. Unpleasant odors, the present of filamentous bacteria and yellowish-green or gray color and placid surface indicate anaerobic conditions.	Overloading, short circuiting, poor operation or toxic discharge.	• Change from a series to parallel operation to divide load. Helpful if conditions exist at a certain time each year and are not persistent. • Add supplemental aeration if pond is continuously overloaded. • Change inlets and outlets to eliminate short circuiting. See How to Correct Short Circuiting. • Add recirculation (temporary use portable pumps) to provide oxygen and mixing. • In some cases temporary help can be obtained by adding sodium nitrate at rates described elsewhere in this manual. • Eliminate sources of toxic discharges.

How to Correct Problems In Aerated Ponds (USEPA 1977)

Indicators/Observations	Probable Cause	Solutions
Fluctuating DO, fin pin floc in final cell effluent, frothing and foaming, ice interfering with operation.	Shock loading, over-aeration, industrial wastes, floating ice.	• Control aeration system by using time clock to allow operation during high load periods, monitor DO to set up schedule for even operation, holding approximately 1 mg/L or more. • Vary operation of aeration system to obtain solids that flocculate or “clump” together in the secondary cell but are not torn apart by excessive aeration. • Locate industrial wastes that may cause foaming or frothing and eliminate or pretreat wastes. Examples are slaughter house, milk or some vegetable wastes. • Operate units continuously during cold weather to prevent freezing damage or remove completely if not a type that will prevent freeze-up.

Table 9-7 (Cont.)

How to Correct A High BOD In The Effluent (Modified USEPA 1977)

Indicators/Observations	Probable Cause	Solutions
High BOD concentrations that are in violation of NPDES or other regulatory agency permit requirements Visible dead algae.	Short detention times, poor inlet and outlet placement, high organic or hydraulic loads and possible toxic compounds.	• Refer to Common Causes of Lagoon Effluent Noncompliance. • Check for collection system infiltration and eliminate at source. • Use portable pumps to recirculate the water. • Add new inlet and outlet locations. • Reduce loads due to industrial sources if above design level. • Prevent toxic discharges.

How to Correct Problems In Anaerobic Ponds (USEPA 1977)

Indicators/Observations	Probable Cause	Solutions
Odors Hydrogen sulfide, (rotten egg) odors or other disagreeable conditions due to sludge in septic condition.	Lack of cover over water surface and insufficient load to have complete activity which eventually forms scum blanket.	• Use straw cast over the surface or polystyrene plans as a temporary cover until a good surface sludge blanket has formed.
Low pH pH below 6.5 accompanied by odors are the result of acid bacteria working in the anaerobic condition.	Acid formers working faster than methane formers in an acid condition.	• The pH can be raised by adding a lime slurry of 580 kg dehydrated lime/200L water (100 lbs/ 50 gal) at a dosage rate of 12 g/10,000 L (1 lb/ 10,000 gal) in the pond. The slurry should be mixed while being added. The best place to put the lime is in at the entrance to the lagoon so that it is well mixed as it enters the pond.

APPENDIX F-1 & F-2

Study Guides for Pond Operators

STUDY GUIDE

INTRODUCTION
TO
STABILIZATION PONDS
AND
AERATED LAGOONS

SUBCLASS D

WISCONSIN DEPARTMENT OF NATURAL RESOURCES
BUREAU OF SCIENCE SERVICES
OPERATOR CERTIFICATION PROGRAM
http://dnr.wi.gov/org/es/science/opcert/

NOVEMBER 1998 EDITION *

* Note – As of Jan 2010, this study guide contains objectives plus key knowledges.

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Mike LaRose - Rice Lake
Mike Magee - Rice Lake
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Janet Hopke, Northern Region, Spooner
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Ron Wilhelm, Madison
Tom Kroehn, Madison
Tom Mickelson, Madison

PREFACE

This operator's study guide represents the results of an ambitious program. Operators of wastewater facilities, regulators, educators and local officials, jointly prepared the objectives and exam questions for this subclass.

The objectives in this study guide have been organized into modules, and within each module they are grouped by major concepts.

NOTE: As of January 2010, this study guide also includes key knowledges.

HOW TO USE THESE OBJECTIVES WITH REFERENCES

In preparation for the exams, you should:

1. Read all of the key knowledges for each objective.
2. Use the resources listed at the end of the study guide for additional information.
3. Review all key knowledges until you fully understand them and know them by memory.

IT IS ADVISABLE THAT THE OPERATOR TAKE CLASSROOM OR ONLINE TRAINING IN THIS PROCESS BEFORE ATTEMPTING THE CERTIFICATION EXAM.

Choosing A Test Date

Before you choose a test date, consider the training opportunities available in your area. A listing of training opportunities and exam dates is available on the DNR Operator Certification home page http://dnr.wi.gov/org/es/science/opcert/ It can also be found in the annual DNR "Certified Operator" or by contacting your DNR regional operator certification coordinator.

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INTRODUCTION TO THE OPERATION OF PONDS AND LAGOONS

MODULE A: PRINCIPLE, STRUCTURE AND FUNCTION

CONCEPT: PRINCIPLE OF PONDS

1. Explain the reasons for using Ponds to treat wastewater.

   PONDS HAVE HISTORICALLY BEEN USED TO PROVIDE DETENTION TIME FOR WASTEWATER TO ALLOW IT TO BE STABILIZED THROUGH NATURAL PROCESSES. WASTEWATER IS TREATED BY THE ACTION OF BACTERIA (BOTH AEROBIC AND ANAEROBIC), OTHER MICRO AND MACRO ORGANISMS, ALGAE, AND BY THE PHYSICAL PROCESS OF GRAVITY SETTLING. WHEN PROPERLY DESIGNED, PONDS ARE CAPABLE OF PROVIDING THE EQUIVALENT OF SECONDARY TREATMENT FOR BOTH BOD AND SUSPENDED SOLIDS.

2. Discuss the advantages and disadvantages of Pond systems as compared to bio-mechanical systems for wastewater treatment.

ADVANTAGES

* LOW CONSTRUCTION COST
* LOW OPERATIONAL COST
* LOW ENERGY USAGE
* CAN ACCEPT SURGE LOADINGS
* LOW CHEMICAL USAGE
* FEWER MECHANICAL PROBLEMS
* EASY OPERATION
* NO CONTINUOUS SLUDGE HANDLING

DISADVANTAGES

* LARGE LAND REQUIREMENTS
* POSSIBLE GROUNDWATER CONTAMINATION FROM LEAKAGE
* CLIMATIC CONDITIONS AFFECT TREATMENT
* POSSIBLE SUSPENDED SOLIDS PROBLEMS (ALGAE)
* POSSIBLE SPRING ODOR PROBLEMS (AFTER ICE-OUT)
* ANIMAL PROBLEMS (MUSKRATS, TURTLES,ETC.)
* VEGETATION PROBLEMS (ROOTED WEEDS,DUCKWEED,ALGAE)
* LOCALIZED SLUDGE PROBLEMS (DEPOSITION NEAR INLET)

3. Describe the following types of ponds:

   A. Areobic.
   B. Anaerobic.
   C. Aerated.
   D. Facultative.

A. <u>AEROBIC</u>: AN AEROBIC POND SYSTEM WOULD HAVE OXYGEN DISTRIBUTED THROUGHOUT THE ENTIRE AREA. THIS WOULD BE SIMILAR TO A CLEAN LAKE WITH ANAEROBIC CONDITIONS OCCURRING ONLY IN BOTTOM SED-IMENTS. THIS CONDITION WOULD PROBABLY ONLY OCCUR IN A TREATMENT SYSTEM UPON INITIAL START-UP WHEN THE POND WOULD BE FILLED WITH A CLEAR WATER SOURCE, OR WHEN COMPLETELY MIXED WITH SUPPLEMENTAL AIR.

B. <u>ANAEROBIC</u>: AN ANAEROBIC POND WOULD BE DEVOID OF ALL OXYGEN THROUGHOUT THE ENTIRE AREA. THIS TYPE OF POND SYSTEM WOULD ONLY BE USED IN SPECIAL APPLICATIONS, USUALLY FOR TREATING CERTAIN INDUSTRIAL WASTES. IF A NORMAL POND SYSTEM IS TOTALLY ANAEROBIC, IT IS ORGANICALLY OVERLOADED. THE ONLY EXCEPTION WOULD BE UNDER ICE COVER FOR A FILL AND DRAW TYPE FACILITY.

C. <u>AERATED</u>: AN AERATED POND SYSTEM WOULD HAVE SUPPLEMENTAL AIR SOURCES TO PROVIDE DISSOLVED OXYGEN. THIS IS USUALLY ACCOMPLISHED WITH SURFACE MECHANICAL AERATORS AND MIXERS, OR BY VARIOUS FORMS OF DIFFUSERS SUPPLIED WITH COMPRESSED AIR FROM MECHANICAL BLOWERS OR COMPRESSORS. FOR EQUAL SIZED PONDS, THE AERATED POND WOULD PROVIDE THE BEST TREATMENT DUE TO THE MECHANICAL ADDITION OF OXYGEN, AND FOR A GIVEN ORGANIC LOADING, WOULD REQUIRE THE LEAST AMOUNT OF LAND AREA.

D. <u>FACULTATIVE</u>: MOST STABILIZATION POND FACILITIES ARE OF THIS TYPE. THE POND CONTAINS AN AEROBIC SURFACE ZONE, AN ANAEROBIC BOTTOM ZONE, AND A TRANSITIONAL (FACULTATIVE) ZONE IN BETWEEN. THIS ALLOWS AEROBIC ORGANISMS TO FUNCTION IN THE UPPER AREA, ANAEROBIC ORGANISMS IN THE LOWER AND SLUDGE AREA, AND FACULTATIVE ORGANISMS IN THE MIDDLE AREA. A FACULATIVE ORGANISM CAN USE DISSOLVED OXYGEN OR COMBINED OXYGEN, BECAUSE THEY CAN ADAPT TO CHANGING CONDITIONS. THEY CAN CONTINUE DECOMPOSITION WHEN THE SYSTEM CHANGES FROM AEROBIC TO ANAEROBIC, ORFROM ANAEROBIC TO AEROBIC.

4. Discuss the relationship between bacteria and algae in a Pond system.

IN ANY WASTEWATER POND, TREATMENT IS ACCOMPLISHED BY A COMPLEX COMMUNITY OF ORGANISMS. THEY WORK IN AN INTERACTION WITH EACH OTHER WHICH IS MUTUALLY BENEFICIAL. ALGAE, LIKE ALL GREEN GROWING MATTER, USES NUTRIENTS AND CARBON DIOXIDE

IN THE PRESENCE OF SUNLIGHT TO PRODUCE OXYGEN IN A PROCESS CALLED PHOTOSYNTHESIS. THE OXYGEN PRODUCED IS USED BY BACTERIA TO ASSIMILATE ORGANIC MATTER, BREAKING IT DOWN INTO SIMPLER MATERIALS AND RELEASING CARBON DIOXIDE TO BE USED BY THE ALGAE.

CONCEPT: STRUCTURE AND FUNCTION

5. Draw line diagrams of three Ponds in Series and in Parallel operation.

SERIES:

```
INCOMING --->  POND   --->  POND   --->  POND   --->DISCHARGE
WASTE          #1           #2           #3          OF TREATED
STREAM                                                 EFFLUENT
```

PARALLEL:

```
POND#1
INCOMING                                             DISCHARGE
WASTE  --->              ---> POND#3 -------->       OF TREATED
STREAM                                                 EFFLUENT
POND#2
```

6. Explain the function of each part of the following parts of a Pond system:

A. Dikes.
B. Pond Seal.
C. Inlet and Outlet Water Control.
D. Flow Meter/Weirs.
E. Headworks/Screening.
F. Rip Rap.

A. DIKES - THE POND SIDES WHICH GIVE THE POND IT'S DEPTH AND STRUCTURE.

B. POND SEAL - A CLAY OR SYNTHETIC LINER THAT KEEPS WASTEWATER FROM PERCOLATING INTO THE GROUNDWATER.

C. WATER CONTROL STRUCTURES:

1. INLET - THE PIPING ARRANGEMENT THROUGH WHICH WASTEWATER IS INTRODUCED INTO THE POND.
2. OUTLET - THE STRUCTURE TO MAINTAIN THE SELECTED POND WATER LEVEL AND ALLOW TREATED WASTE TO FLOW OUT.

D. FLOW METER/WEIRS - DEVICES TO MEASURE INCOMING OR DISCHARGED WASTEWATER FLOW RATES.

E. HEADWORKS/SCREENING - SOMETIMES PROVIDED TO REMOVE RAGS AND LARGE OBJECTS.

F. RIP RAP - ROCK OR STONE PLACED AT NORMAL POND OPERATING LEVELS TO PREVENT EROSION OF THE DIKES THAT COULD OCCUR FROM WIND ACTIONS.

7. Describe two common kinds of pond water level control structures.

A. SUBMERGED PIPE OUTLET WITH WATER LEVEL CONTROL BOARDS BOARDS ARE REMOVED OR ADDED TO RAISE OR LOWER THE POND LEVEL (USUALLY IN A MANHOLE).

B. TELESCOPING VALVE - A TELESCOPING PIPE SECTION THAT CAN BE RAISED OR LOWERED TO CONTROL WATER LEVELS (USUALLY IN A MANHOLE-LIKE STRUCTURE).

8. State two important functions of an Aeration System.

A. IT ADDS DISSOLVED OXYGEN TO THE POND CONTENTS.
B. IT MIXES THE POND CONTENTS.

9. Describe the function of each of the following components of a Pond Aeration System:

A. Compressors/Blowers.
B. Airlines.
C. Diffusers.
D. Mechanical Aeration.

A. COMPRESSORS/BLOWERS: USED TO PROVIDE LOW PRESSURE AIR USED IN THE POND AERATION SYSTEM.

B. AIRLINES: A PIPING SYSTEM USED TO CONVEY COMPRESSED AIR TO THE POINTS OF APPLICATION.

C. DIFFUSERS: VARIOUS TYPES OF EQUIPMENT USUALLY LOCATED NEAR THE POND BOTTOM. USED TO FORM BUBBLES IN THE POND LIQUID TO ENTRAIN OXYGEN AND PROVIDE MIXING.

D. MECHANICAL AERATION: SEVERAL DIFFERENT TYPES OF EQUIPMENT (USUALLY ON FLOATS) THAT SPRAY THE WATER INTO THE AIR TO ENTRAIN OXYGEN AND PROVIDE MIXING.

10. Discuss the purpose of a Blower Air Relief Valve in a Pond Aeration System.

IN THE EVENT OF EXCESS PRESSURE (PLUGGED DIFFUSERS OR AIR LINES) THE PRESSURE RELIEF VALVE WILL OPEN TO RELEASE EXCESS PRESSURE AND PROTECT THE PIPING, DIFFUSERS, AND THE BLOWER.

11. Describe what is meant by the term "freeboard" in a Pond system.

FREEBOARD IS THE DISTANCE BETWEEN THE NORMAL MAXIMUM OPERATING WATER SURFACE OF THE POND, AND THE TOP OF THE DIKE. FREEBOARD IS NORMALLY 3 FEET (MEANING THE WATER LEVEL SHOULD BE KEPT WITHIN 3 FEET FROM THE DIKE TOP).

MODULE B: OPERATION AND MAINTENANCE

CONCEPT: OPERATION

12. Describe series and parallel modes of Pond operation, and state conditions when each should be used.

A STABILIZATION POND SYSTEM IS USUALLY COMPOSED OF A NUMBER OF INDIVIDUAL CELLS(PONDS) AND CAN BE OPERATED IN SEVERAL MODES.

SERIES: **IN THIS MODE THE FLOW GOES THROUGH EACH CELL(POND) IN SUCCESSION (E.G. PRIMARY CELL TO SECONDARY CELL TO TERTIARY CELL). THIS TYPE OF FLOW PATTERN NORMALLY PROVIDES THE BEST DEGREE OF TREATMENT AND MINIMIZES ALGAE IN THE EFFLUENT.**

PARALLEL: **IN THIS MODE OF OPERATION THE INFLUENT FLOW IS DIVIDED INTO TWO OR MORE PRIMARY CELLS. PARALLEL OPERATION IS NORMALLY USED WHEN LOADINGS EXCEED DESIGN LEVELS, WHEN ORGANIC OVERLOADS ARE EXPECTED,OR DURING WINTER CONDITIONS WHEN CLIMATIC CONDITIONS REDUCE THE AMOUNT OF DISSOLVED OXYGEN.**

13. Discuss why some Ponds have difficulty meeting suspended solids limits.

THE MOST COMMON PROBLEM IN MEETING SUSPENDED SOLIDS LIMITS IN POND SYSTEMS WOULD BE EXCESSIVE ALGAE GROWTH BEING DISCHARGED WITH THE FINAL EFFLUENT. OTHER MINOR PROBLEMS

THAT COULD CAUSE SUSPENDED SOLIDS EFFLUENT PROBLEMS ARE RISING SLUDGE, AND SOMETIMES ABUNDANT ZOOPLANKTON (SUCH AS DAPHNIA).

14. Explain why an operator should prefer to have a Pond dominated by green algae.

GREEN ALGAE IS THE PREFERRED SPECIES THAT INDICATES A PROPERLY FUNCTIONING POND SYSTEM. IF BLUE-GREEN ALGAE TAKE OVER (USUALLY INDICATING ORGANIC OVERLOADING), THEY CAN CAUSE BLACK-GREEN FLOATING MATS. THIS CAN CAUSE OPERATIONAL PROBLEMS SUCH AS SHORT-CIRCUITING, REDUCTION OF MIXING, POOR LIGHT PENETRATION, MAT REMOVAL PROBLEMS, ODOR, AND GENERAL UNSIGHTLINESS. SPRING TURN-OVER MAY CAUSE A BLUE-GREEN ALGAE BLOOM.

15. List ways most Ponds gain Dissolved Oxygen.

A. PHOTOSYNTHESIS BY ALGAE WITHIN THE POND(MAIN SOURCE OF OXYGEN IN MOST POND TYPE SYSTEMS, ESPECIALLY SHALLOW PONDS IN THE 3-5 FOOT DEPTH RANGE).
B. DIFFUSION OF ATMOSPHERIC OXYGEN AT THE POND SURFACE WITH THE ACTION OF THE WIND PROVIDING MIXING OF THE OXYGEN RICH SURFACE LAYER WITH THE WATER BELOW.
C. THE USE OF COMPRESSED AIR SYSTEMS OR SURFACE MECHANICAL AERATORS.

16. Explain why dissolved oxygen concentrations vary with Pond depth.

OXYGEN LEVELS VARY WITH DEPTH FOR A NUMBER OF REASONS. THE MAIN REASON IS THE RELATIONSHIP OF THE ORGANISMS WITHIN THE POND. OTHER REASONS ARE THE PHYSICAL ACTIONS WITHIN THE POND, AND THE LOADING TO THE POND.

THE RELATIONSHIP OF ORGANISMS INVOLVES THE GENERAL INTERACTION BETWEEN ALGAE AND BACTERIA. THE ALGAE ARE THE MAIN SOURCE OF OXYGEN IN A POND SYSTEM. ALGAE GROWTH IS GREATEST NEAR THE SURFACE WHERE LIGHT PENETRATION AND PHOTOSYNTHESIS IS THE GREAT- EST. OXYGEN LEVELS DECREASE WITH DEPTH, DUE TO LESS LIGHT PENETRATION NEEDED FOR PHOTOSYNTHESIS.

THE ALGAE USE CARBON DIOXIDE IN THE PROCESS OF PHOTOSYNTHESIS AND PRODUCE OXYGEN. THE BACTERIA STABILIZE ORGANIC MATTER USING THE OXYGEN AND PRODUCE CARBON DIOXIDE.

THE PHYSICAL DIFFUSION OF ATMOSPHERIC OXYGEN OCCURS AT THE SURFACE OF PONDS AND IS MIXED IN THE UPPER LAYERS BY WIND ACTION. THE AMOUNT OF MIXING IS LIMITED, SO THE OXYGEN LEVELS DECREASE WITH DEPTH.

THE FINAL FACTOR AFFECTING OXYGEN LEVELS IS THE ORGANIC LOADING TO THE SYSTEM. IF ORGANIC LOADINGS ARE SMALL, THE OXYGEN LEVELS WILL BE MAINTAINED AT GREATER DEPTHS. IF ORGANIC OVERLOADING OCCURS, THE WHOLE POND COULD GO ANAEROBIC.

17. List the steps to follow during start-up of a Pond system.

A. FILL WITH CLEAR WATER (RIVER OR WELL WATER) IF IT HAS A PLASTIC LINING, PARTLY FULL IF IT HAS A CLAY SEAL. THIS APPROACH PREVENTS WEEDS, DRYING OF THE POND, AND PREVENTS ODORS WHEN SEWAGE IS ADDED.

B. CONDUCT LEAKAGE TESTS.

C. BEGIN ADDING RAW WASTEWATER.

18. Describe strategies to use when operating a Fill and Draw Pond system.

FILL AND DRAW POND SYSTEMS ARE DESIGNED FOR INTERMITTENT DIS-CHARGE. DISCHARGES ARE USUALLY IN THE SPRING AND FALL WHEN STREAM FLOWS ARE HIGH AND TEMPERATURES LOW. LOW TEMPERATURE ALLOWS MORE OXYGEN TO BE DISSOLVED IN WATER. IT IS NECESSARY TO HAVE A HOLDING CAPACITY FOR A MINIMUM OF SIX MONTHS FLOW. SAMPLING OF THE POND TO BE DISCHARGED IS REQUIRED AND APPROVAL MUST BE OBTAINED FROM THE DNR.

19. Explain the conditions that indicate times to Drawdown and to Fill a Pond.

A. DRAWDOWN:

A POND SHOULD BE DRAWN DOWN IN FALL AFTER THE FIRST FROST AND WHEN THE ALGAE CONCENTRATION DROPS OFF, THE BOD IS STILL LOW, AND WHEN THE RECEIVING STREAM TEMPERATURE IS LOW WITH ACCOMPANYING HIGH DISSOLVED OXYGEN.
A POND SHOULD BE DRAWN DOWN IN SPRING BEFORE ALGAE CONCENTRATION INCREASES, WHEN THE BOD LEVEL IS ACCEPTABLE, AND WHEN THE RECEIV-ING STREAM FLOWS ARE HIGH (LOW TEMPERATURE WITH HIGH DISSOLVED OXYGEN HELPS). DURING THE ACTUAL DISCHARGE, THE EFFLUENT MUST BE SAMPLED FOR BOD, SUSPENDED SOLIDS AND pH AT A FREQUENCY SPECIFIED IN THE DISCHARGE PERMIT.

TO DRAW DOWN A POND, ISOLATE THE POND, IF POSSIBLE, ONE MONTH BEFORE THE DISCHARGE PERIOD. BEGIN TESTING TO MONITOR POND CONTENTS FOR BOD, SUSPENDED SOLIDS, AND pH. SEND RESULTS TO THE DNR AND OBTAIN APPROVAL TO DISCHARGE. CALCULATE WHAT VOLUME WILL BE NEEDED FOR STORAGE, AND DISCHARGE AT LEAST THAT AMOUNT. DETERMINE FROM THE DISCHARGE PERMIT DAILY DISCHARGE VOLUME, AND CALCULATE TOTAL DAYS

REQUIRED FOR DISCHARGE. CALCULATE, OR USE A CHART PROVIDED BY THE DESIGN ENGINEER, TO FIND WHAT LEVEL THE POND WILL BE LOWERED AND HOW MANY INCHES/DAY IT WILL DROP. PONDS ARE NEVER COMPLETELY DRAWN DOWN AS THIS COULD DRY OUT THE SEAL AND CAUSE LEAKAGE.

B. FILL A POND:

ALWAYS LEAVE AT LEAST ONE OR TWO FEET OF TREATED WASTEWATER IN A POND SO THE WASTEWATER WILL HAVE AN ACTIVE BACTERIAL CONCENTR-ATION. THIS GREATLY AIDS IN MAINTAINING OXYGEN AND PREVENTS ODORS OR ORGANIC UPSETS. IF POSSIBLE, FILL AS SLOWLY AS POSSIBLE, STARTING WITH THE PRIMARY POND. IF THERE ARE TWO OR MORE PRIMARIES, ALTERNATE FLOW TO EACH ON A DAILY BASIS. CONTINUE FILLING THE PRIMARY UNTIL IT IS FULL. THIS MAY TAKE SEVERAL MONTHS. ALLOW FLOW OF THE PRIMARY POND CONTENTS TO THE SECONDARY POND.

20. List the reasons why an operator would vary Pond levels.

A. TO DRAW DOWN THE CELL.
B. TO HOLD CONTENTS LONGER AND ALLOW MORE TREATMENT AND DETENTION TIME (ESPECIALLY IN WINTER).
C. TO REPAIR AERATION EQUIPMENT OR OTHER STRUCTURE.
D. TO REPAIR LEAKS.
E. TO CONTROL MUSKRATS.
F. TO CONTROL ROOTED WEEDS.
G. TO FLOOD CUT CATTAILS.

21. Describe the proper operation of Multiple Seepage Cells.

THE BEST OPERATION IS LOAD AND REST. DRYING OCCURS BETWEEN LOAD-INGS SO AN AEROBIC ZONE IS MAINTAINED IN THE SOIL. ALTERNATE EVERY THREE WEEKS, TO A MONTH. BEFORE DISCHARGING TO A SEEPAGE CELL, THE POND CONTENTS MUST BE MONITORED FOR BOD AND SUSPENDED SOLIDS. WHEN A DISCHARGE IS OCCURRING, A DAILY CHECK FOR THE VOLUME TO THE SEEPAGE CELL AND THE DEPTH OF WATER IN THE CELL IS APPROPRIATE. THE FLOW SHOULD BE UNIFORMLY DISTRIBUTED ACROSS THE ENTIRE SEEPAGE CELL.

22. Discuss how to transfer liquid from cell to cell.

IN A 2-CELL SYSTEM, ISOLATE CELL#2, DRAW DOWN CELL#2 FIRST, THEN REFILL CELL#2 FROM CELL#1. CONTROL VALVES BETWEEN CELLS ARE REGULATED SO THE TRANSFERS ARE GRADUAL.

23. Describe how to check for efficient aeration of a Pond.

MONITOR POND DISSOLVED OXYGEN, WATCH SURFACE AERATION PATTERNS FOR CHANGES, READ AIRLINE PRESSURE GAUGE, CHECK FOR CHANGES IN EFFLUENT BOD, AND MONITOR ALL AERATION EQUIPMENT. FOR PROPER TREATMENT, AN AERATED POND SHOULD HAVE AN

ADEQUATE SUPPLY OF DISSOLVED OXYGEN. FOR PRACTICAL PURPOSES, THE DISSOLVED OXYGEN IN THE SURFACE MIXED ZONE SHOULD AVERAGE APPROXIMATELY 2 mg/L.

24. Explain why pH values vary in a Pond.

THE VARIATION IN pH IN A FACULTATIVE POND NORMALLY OCCURS IN THE UPPER AEROBIC ZONE, WHILE THE ANAEROBIC AND FACULTATIVE ZONES WILL BE RELATIVELY CONSTANT. THIS VARIATION HAPPENS DUE TO THE CHANGES THAT OCCUR IN THE CONCENTRATION OF DISSOLVED CARBON DIOXIDE. WHEN CARBON DIOXIDE IS DISSOLVED IN WATER IT FORMS A WEAK CARBONIC ACID WHICH WOULD TEND TO LOWER pH. THE RELATIONSHIP BETWEEN ALGAE AND BACTERIA AFFECT THE CARBON DIOXIDE LEVELS. DURING INTENSE PHOTOSYNTHESIS, ALGAE USE CARBON DIOXIDE AND PRODUCE OXYGEN TO BE USED BY BACTERIA TO ASSIMILATE ORGANIC WASTES. THE ALGAE USE MUCH OF THE CARBON DIOXIDE AND THE pH CAN RISE SIGNIFICANTLY(pH IN THE 11 TO 12 RANGE IS NOT UNCOMMON).

DURING THE NIGHT OR DURING CLOUDY WEATHER, THE ALGAE RESPIRE AND ACTIVE PHOTOSYNTHESIS DOES NOT OCCUR. THE BACTERIA CONTINUE TO USE UP OXYGEN AND PRODUCE CARBON DIOXIDE. THIS CAN CAUSE A SIGNIFICANT DROP IN THE POND pH, ESPECIALLY IF THE INFLUENT WASTEWATER HAS LOW ALKALINITY. THIS SAME pH SWING CAN OCCUR IN NATURAL PONDS, LAKES, AND STREAM IMPOUNDMENTS. DURING PEAK SUMMER ALGAE ACTIVITY, THE DISSOLVED OXYGEN OF STREAM IMPOUNDMENTS HAVE VARIED FROM DAWN LEVELS OF LESS THAN 1 mg/L, TO LATE AFTERNOON VALUES OF 13-15 mg/L (SUPERSATURATION).

25. Describe the affects of seasonal changes on Pond treatment efficiency.

WINTER: TREATMENT EFFICIENCY DECREASES IN THE WINTER WITH COLDER TEMPERATURES AND LESS SUNLIGHT THROUGH THE ICE COVER. SHORTER PERIODS OF SUNLIGHT AND ICE COVER LIMITS THE AMOUNT OF PHOTOSYNTHESIS. THIS REDUCES DISSOLVED OXYGEN IN THE POND. THE COLD WATER ALSO SLOWS DOWN BACTERIAL ACTION, REDUCING TREATMENT EFFICIENCY. IF SUFFICIENT ICE COVER IS PRESENT,THE POND MAY GO ANAEROBIC. EMERGENT WEEDS AND DUCKWEED DIE-OFF. DURING THIS PERIOD, FILL AND DRAW PONDS ARE OPERATED BY STORING WASTEWATER FOR A SPRING DISCHARGE.

SPRING: AFTER ICE-OUT, ODORS MAY OCCUR FOR SEVERAL DAYS UNTIL DISSOLVED OXYGEN IS RESTORED. AS TEMPERATURES INCREASE, BIOLOGICAL ACTIVITY INCREASES FOR BOTH BACTERIA AND ALGAE. TREATMENT EFFICIENCY BEGINS TO IMPROVE WITH INCREASING BIOLOGICAL ACTIVITY. AFTER THE THE POND HAS STABILIZED, A SPRING DISCHARGE FOR

FILL AND DRAW TYPE SYSTEMS IS USUALLY DONE PRIOR TO ACTIVE ALGAE GROWTH.

SUMMER: THE LONG SUNNY DAYS PROVIDE MAXIMUM OXYGEN LEVELS FROM ALGAE PHOTOSYNTHESIS. WARM WATER TEMPERATURES INCREASE BACTERIA ACTION TO PROVIDE THE BEST ENVIRONMENT FOR EFFICIENT TREATMENT. OPERATIONAL PROBLEMS INCLUDE: CONTROLLING ROOTED EMERGENT WEEDS, REMOVING DUCKWEED AND CONTROLLING ALGAE BLOOMS. DURING THIS PERIOD, FILL AND DRAW POND SYSTEMS ARE OPERATED BY STORING WASTEWATER FOR A FALL DISCHARGE.

FALL: A TRANSITIONAL TIME, BUT IN REVERSE OF SPRING. WATER TEMPERATURES BEGIN DROPPING, REDUCING BACTERIAL ACTIVITY AND PHOTOSYNTHESIS AS THE DAYS GET SHORTER. TREATMENT EFFICIENCIES BEGIN TO DROP AS WINTER APPROACHES. WHEN THE ALGAE LEVELS DROP AND THE BOD STABILIZES, FILL AND DRAW TYPE SYSTEMS NORMALLY DISCHARGE.

26. Discuss the operating procedures for dealing with a spring thaw.

PONDS WILL USUALLY FILL UP FAST DURING SPRING THAW AND LEVELS MUST BE WATCHED SO DIKES DO NOT OVERFLOW. DISCHARGE SHOULD BE CONTINUOUS UNTIL LEVELS STABILIZE. START SPRING DRAW DOWN OF THE PONDS IF OPERATING ON FILL AND DRAW. THE COLLECTION SYSTEM USUALLY HAS INFILTRATION, AND FLOW IS QUITE LARGE DURING THE SPRING THAW. DRAW PONDS DOWN WHEN STREAMS ARE COLD AND FLOWS HIGH.

CONCEPT: MAINTENANCE

27. List some components of a maintenance management and recordkeeping system.

A. MAINTENANCE INVENTORY OF PARTS AND OIL.
B. WEATHERIZATION OF PLANT AND EQUIPMENT.
C. INSURE O&M MANUAL IS BEING FOLLOWED.
D. MAINTAIN A MANAGEMENT CHECKLIST WHICH MIGHT INCLUDE:
    1. EACH MAINTENANCE DUTY.
    2. FREQUENCY OF MAINTENANCE.
    3. INVENTORY OF PARTS NEEDED.
    4. DETAILED DESCRIPTION OF PROPER METHODS OF MAINTENANCE.

MAINTENANCE RECORD KEEPING IS THE USE OF VARIOUS FORMATS TO RECORD THE PERFORMANCE OF ACTUAL MAINTENANCE. TYPICAL EXAMPLES WOULD BE A FOLDER FILING SYSTEM (FILE CABINET). A CARD SYSTEM FOR RECORDING INFORMATION, AND THE USE OF MICROCOMPUTERS WITH APPROPRIATE SOFTWARE. ANY OF THESE

SYSTEMS CAN BE USED FOR RECORD KEEPING AND PLANNING MAINTENANCE.

28. Describe the meaning of air gauge readings on a blower.

HIGH READINGS OF AN AIR GAUGE ARE CAUSED BY PLUGGED AIRLINE, ORIFICES, DIFFUSERS, OR ICE CAP. LOW READINGS OF AN AIR GAUGE COULD BE CAUSED BY A FAULTY BLOWER, AN AIR LEAK, OR CLOGGED BLOWER INLET FILTER.

IN EITHER CASE, THERE IS A POSSIBILITY THAT THE BLOWER COULD OVERHEAT, CAUSING DAMAGE TO THE UNIT. A HOT BLOWER SHOULD BE SHUT-DOWN AND CORRECTIVE ACTION TAKEN.

29. List the most common maintenance problems associated with Pond systems.

A. WEED CONTROL - CATTAILS AND OTHER ROOTED AQUATIC PLANTS.
B. ALGAE CONTROL - BLUE-GREEN AND ASSOCIATED FLOATING ALGAE MATS.
C. BURROWING ANIMALS - MUSKRATS AND TURTLES.
D. DUCKWEED CONTROL AND REMOVAL.
E. FLOATING SLUDGE MATS.
F. DIKE VEGETATION - MOWING AND REMOVING WOODY PLANTS.
G. DIKE EROSION - RIP RAP AND PROPER VEGETATION.
H. FENCE MAINTENANCE TO RESTRICT ACCESS.
I. MECHANICAL EQUIPMENT - PUMPS, BLOWERS ETC.

30. Discuss the maintenance of seepage cells.

RAKE THE DRY SURFACE WITH EQUIPMENT THAT WILL NOT COMPACT SOIL. CONTROL WEEDS BY TILLING THE SOIL. KEEP LEVEL. SEEPAGE CELL MAINTENANCE INVOLVES AERATING THE SOIL CRUST WHICH BUILDS-UP AT THE SOIL-AIR INTERFACE. THIS CRUST IMPEDES WATER AND OXYGEN PERCOLATION INTO THE SOIL. ANY SUITABLE TILLING EQUIPMENT CAN BE USED. TILLING 6" TO 12" HELPS CONTROL WEED GROWTH WHICH PROLIF-ERATES ON THE SURFACE. AVOID UNNECESSARY SOIL COMPACTION.

31. Describe the ways to control aquatic vegetation.

ROOTED WEEDS CAN BE CONTROLLED BY PHYSICAL REMOVAL OF NEW GROWTH BY HAND, OR MOWING WITH A SICKLE BAR AFTER ICE HAS FORMED, RAISE WATER LEVEL ALLOWING THE ICE TO PULL THE WEEDS OUT. BY INCREASING THE WATER LEVEL TO REDUCE LIGHT PENETRATION TO STOP PHOTOSYNTHESIS. OTHER POSSIBLE WAYS WOULD BE TO LOWER THE WATER LEVEL AND BURN THE WEEDS OR USE AN APPROVED HERBICIDE.

32. Explain how to remove duckweed from the Pond surface.

DUCKWEED MUST BE PHYSICALLY REMOVED WITH A RAKE, PUSHBOARD OR BROOM. WITH SUFFICIENT WIND, THE DUCKWEED WILL BE PUSHED TO ONE SIDE OR CORNER OF A POND. THIS IS AN IDEAL TIME TO RAKE THEM OUT. IT IS IMPORTANT THAT DUCKWEED NOT BE ALLOWED TO BECOME TOO ABUNDANT, AS IT REDUCES OXYGEN TRANSFER AT THE WATER SURFACE, REDUCES LIGHT PENETRATION AND PHOTOSYNTHESIS, AND UPON DECOMPOSING, CAN CAUSE BOTH ODOR AND BOD PROBLEMS.

33. Discuss how to deal with floating mats.

FLOATING MATS ON POND SYSTEMS ARE CAUSED BY FLOATING SLUDGE, BLUE-GREEN ALGAE, OR OIL AND GREASE. THE MOST COMMON ARE THE SLUDGE AND ALGAE MATS. THE FIRST ATTEMPT TO CORRECT THIS WOULD BE TO TRY TO BREAK-UP THE SLUDGE OR ALGAE MATS, ALLOWING THEM TO SETTLE TO THE BOTTOM. IF THIS DOES NOT WORK, IT WILL BE NECESS-ARY TO RAKE THEM OUT AND DISPOSE OF THEM. IF OIL AND GREASE ARE A PROBLEM, THE SOURCE OF THIS MATERIAL SHOULD BE ELIMINATED TO PREVENT A RECURRENCE OF THIS PROBLEM.

34. Describe how cattails are controlled without chemicals.

CATTAILS CAN BECOME ESTABLISHED IN THE SHALLOW WATER ALONG THE DIKES. CONTROLLING CATTAILS IS A PROBLEM BECAUSE OF THEIR EX-TENSIVE ROOT SYSTEMS. PHYSICAL REMOVAL HAS THE POSSIBILITY OF DAMAGE TO THE POND LINER. WHEN CATTAILS ARE YOUNG, PULLING THEM OUT IS VERY AFFECTIVE. ANOTHER AFFECTIVE METHOD IS TO LOWER THE POND LEVEL, CUT THE CATTAILS, AND THEN RAISE THE WATER THREE FEET OVER THE CATTAILS WHICH EFFECTIVELY KILLS(DROWNS) THEM. ONE METHOD WOULD BE A BOAT MOUNTED WEED CUTTER TO CUT THEM OFF BELOW THE WATERLINE.

35. Identify types of dike vegetation, and how to control grass and other plant growths.
IT IS VERY IMPORTANT THAT DIKES HAVE A PROTECTIVE GRASS COVER TO PREVENT EROSION FROM RUNOFF AND WAVE ACTION. THE GRASSES USED SHOULD BE FAST GROWING, SPREADING, WITH SHALLOW, BUT DENSE ROOT SYSTEMS(E.G. RYE, BROME AND QUACK). MOWING SHOULD BE DONE PER-IODICALLY SO THAT THE DIKES CAN BE OBSERVED AND TO REDUCE BREEDING AREAS FOR INSECTS.

NO LONG ROOTED PLANTS SHOULD BE ALLOWED ON DIKES(ALFALFA, WILLOWS OR ANY WOODY SCRUBS) AS THEIR ROOT STRUCTURE COULD CAUSE DIKE LEAKAGE, DAMAGE TO THE POND SEAL, OR STRUCTURAL FAILURE TO THE DIKE. ALL WOODY PLANTS SHOULD BE REMOVED BY PULLING OR MOWING, AND IN THE EVENT THEY BECOME ESTABLISHED, IT WILL BE NECESSARY TO USE BRUSHING METHODS (EG. PRUNING, CHAIN SAW, BRUSH SAW, WEED WACKER, ETC.). GRAZING ANIMALS SHOULD NOT BE USED TO CONTROL DIKE VEGETATION AS THEY DAMAGE DIKES AND INCREASE EROSION PROBLEMS.

MODULE C: MONITORING AND TROUBLESHOOTING

CONCEPT: MONITORING

36. State the normal pressure reading range for the Discharge Gauge in a blower unit.

    5 - 14 PSI.

37. Describe what types of things that need monitoring usually found in the discharge permit for influent and effluent from a Pond system.

    THE DISCHARGE PERMIT WILL SPECIFY THE TYPE OF SAMPLES(GRAB OR COMPOSITE) REQUIRED, AND THE FREQUENCY OF SAMPLING. NORMALLY, BOTH RAW WASTEWATER AND FINAL EFFLUENT WILL REQUIRE SAMPLING FOR BOD, SUSPENDED SOLIDS, FECAL COLIFORM, AND pH. OTHER PARAMETERS MAY BE SPECIFIED IN INDIVIDUAL DISCHARGE PERMITS, SUCH AS, AMMONIA, TOTAL NITROGEN, NITRATES, CHLORIDES, TOTAL DISSOLVED SOLIDS OR TOXICS.

38. List ways to measure the Dissolved Oxygen level of a Pond.

    A. DISSOLVED OXYGEN METER.
    B. WINKLER DISSOLVED OXYGEN DETERMINATION.

39. Define where samples should be taken on a Pond to monitor the influent and effluent.

    SAMPLES OF RAW WASTEWATER SHOULD BE TAKEN AT THE POINT WHERE THE RAW WASTEWATER ENTERS THE WET WELL. SAMPLES OF FINAL EFFLUENT SHOULD BE TAKEN WHERE THE FINAL EFFLUENT LEAVES THE TREATMENT SYSTEM. USE THE LAST MANHOLE AFTER THE POND, JUST BEFORE DIS-CHARGE INTO A STREAM OR RIVER.

    THE MOST IMPORTANT FACTOR IN LOCATING AN INFLUENT SAMPLING POINT WOULD BE TO ENSURE THAT IT IS WELL MIXED AND IS REPRESENTATIVE OF THE RAW WASTEWATER. IF GRAB SAMPLES ARE SPECIFIED, THEY SHOULD NOT BE COLLECTED DURING UNUSUAL FLOW CONDITIONS, SUCH AS, VE~~RY~~ LOW FLOW PERIODS (EARLY MORNING, LATE EVENING OR WEEKENDS), DURING A MAJOR STORM, POWER OUTAGE, OR AN OBVIOUS SLUG LOADING. THE SAMPLE SHOULD BE REPRESENTATIVE OF THE NORMAL LOADING. THE FINAL EFFLUENT SAMPLE SHOULD BE AT A WELL-MIXED REPRESENTATIVE LOCATION.

40. Describe how to take a representative sample of the contents of a Pond.

OVERALL POND SAMPLING IS NORMALLY DONE FOR FILL AND DRAW TYPE SYSTEMS TO BE SURE POND CONTENTS ARE SUITABLE FOR DISCHARGE TO THE RECEIVING WATER COURSE. IT IS IMPORTANT THAT THIS SAMPLING BE REPRESENTATIVE OF THE POND CONTENTS. THIS WOULD MEAN THAT MULTIPLE SAMPLES SHOULD BE COLLECTED AND THEN COMPOSITED PRIOR TO LABORATORY ANALYSIS. ONE SUGGESTED METHOD WOULD BE TO TAKE SAMPLES AT FOUR LOCATIONS AROUND THE POND AT LEAST 8 FEET FROM THE DIKE AND FROM BELOW THE WATER SURFACE OF THE POND. MIX TOGETHER THOURGHLY PRIOR TO ANALYSIS.

41. Explain how the following samples should be collected and preserved for analysis:

A. BOD.
B. Fecal Coliform.
C. Suspended Solids.
D. Dissolved Oxygen.
E. pH.

A. BOD: SAMPLES SHOULD BE TAKEN AT PEAK FLOW TIMES(I.E.,11:00AM, 12:00 NOON AND 1:00PM), COMPOSITED TOGETHER, MIXED WELL, PRESERVED WITH ICE, AND SENT FOR ANALYSIS AS SOON AS POSSIBLE TO PREVENT DEGRADATION WITHIN 48 HRS @ 3^{0}C.

B. FECAL COLIFORM: THIS SAMPLE OF THE UNCHLORINATED EFFLUENT MUST BE COLLECTED IN A SEPARATE STERILIZED CONTAINER, PRESERVED WITH ICE AND SENT FOR ANALYSIS AS SOON AS POSSIBLE. IF SAMPLING A CHLORINATED DISCHARGE, SODIUM THIOSULFATE MUST BE ADDED AND NOTED ON THE LAB SLIP.

C. SUSPENDED SOLIDS: SAME AS FOR BOD.

D. DISSOLVED OXYGEN: MUST BE ANALYZED IMMEDIATELY AFTER SAMPLING.

E. pH: MUST BE ANALYZED IMMEDIATELY AFTER SAMPLING.

42. Discuss how to collect a representative sample from a groundwater monitoring well.

THE REQUIREMENTS FOR GROUND WATER SAMPLING POINTS AND PARAMETERS TO BE TESTED WILL BE IN THE DISCHARGE PERMIT. WHEN SAMPLING GROUND WATER, START WITH UP-GRADIENT WELLS FIRST, AND THEN MOVE TO DOWN-GRADIENT WELLS. THE SEQUENCE

OF PROCEDURES FOR OBTAIN-ING A REPRESENTATIVE SAMPLE WOULD BE:

1. DETERMINE GROUND WATER ELEVATION USING PROPERLY CLEANED AND RINSED EQUIPMENT (COPPER COATED TAPE, OR AN ELECTRIC TAPE).
2. SUBTRACT DEPTH TO WATER FROM REFERENCE POINT. USUALLY, WELL TOP TO GET GROUND WATER ELEVATION.
3. DETERMINE DEPTH OF THE WELL FROM REFERENCE POINT TO GET ELEV-ATION OF WELL BOTTOM.
4. SUBTRACT BOTTOM OF WELL ELEVATION FROM GROUND WATER ELEVATION TO OBTAIN DEPTH OF WATER IN THE WELL.
5. USE THE DEPTH OF WATER IN THE WELL AND THE INSIDE CASING DIAMETER TO GET VOLUME OF WATER IN THE WELL.
6. BAIL FOUR VOLUMES OF WATER AS DETERMINED FROM ABOVE VOLUME.
7. AFTER BAILING FOUR VOLUMES, COLLECT SAMPLE AND SEND TO THE LABORATORY, FOLLOWING ANY INSTRUCTION OF THE LABORATORY (E.G. PRE-FILTERING OR ANY PRESERVATION THAT MAY BE REQUIRED).

CONCEPT: TROUBLESHOOTING

43. List the possible causes of low water levels in a Pond.

A. LEAKING LINER.
B. LEAKING CONTROL STRUCTURES.
C. UNDERLOADED FACILITY (OVER DESIGNED).
D. DIKE LEAKS CAUSED BY BURROWING ANIMALS.
E. IMPROPER SETTINGS OF CONTROL STRUCTURES.

44. List the causes and corrective actions for Seepage Cells that do not seep.

	CAUSE	CORRECTIVE ACTION
A.	COMPACTED CELL BOTTOM.	REWORK CELL BOTTOM WITH MECHANICAL EQUIPMENT TO LOOSEN AND AERATE SOIL.
B.	HYDRAULIC OVERLOAD.	REDUCE OVERLOAD BY ALTERNATING SEEPAGE CELL LOADING.
C.	SLUDGE BUILD-UP.	REMOVE SLUDGE FROM CELL. CORRECT OPERATION OF TREATMENT PONDS PRECEDING SEEPAGE CELLS.

45. List some causes of Pond short-circuiting, and give corrective action for each.

CAUSE	CORRECTIVE ACTION
A. EXCESSIVE ROOTED WEED GROWTH.	CONTROL WEED GROWTH TO RESTORE NORMAL FLOW PATTERNS AND DETENTION TIME.
B. HYDRAULIC OVERLOADING.	REDUCE HIGH LOADINGS BY ELIMINATING EXCESS I/I IN THE COLLECTION SYSTEM.
C. DESIGN RELATED PROBLEMS.	CHANGE INLET OR OUTLET STRUC TURE LOCATION TO STOP SHORT-CIRCUITING OR ADD BAFFLES AS NEEDED.

46. Discuss the causes and corrective action for a Pond having a suspended solids violation while meeting BOD limits.

THE MOST PROBABLE CAUSE OF THIS PROBLEM WOULD BE AN ALGAE BLOOM. DEPENDING ON THE TYPE OF ALGAE PRESENT, THE CORRECTION OF THE CAUSE OF THE BLOOM CAN BE:

IF, THE ALGAE IS OF THE NORMAL GREEN VARIETY, POSSIBLE SOLUTIONS COULD INCLUDE:

A. DRAW OFF EFFLUENT FROM BELOW THE SURFACE TO TRY TO REDUCE ALGAE CONCENTRATION.
B. CONSTRUCT BAFFLES TO GET A BETTER QUALITY EFFLUENT.
C. IF POSSIBLE, USE ANOTHER CELL AND LET THE OTHER "REST" UNTIL THE BLOOM SUBSIDES.
D. CONSIDER USE OF A SAND FILTER FOR ALGAE REMOVAL.
E. SWITCH TO SERIES OPERATION OF PRIMARY CELLS IF YOU ARE PRESENTLY OPERATING IN PARALLEL.
F. CONSIDERATION CAN BE GIVEN TO USE OF AN APPROVED ALGICIDE.
G. IF OPERATIONAL CHANGES CANNOT CORRECT THE PROBLEM, AN ALGAE PERMIT VARIANCE CAN BE PURSUED.
H. CHECK COLLECTION SYSTEM FOR EXCESS NUTRIENT LOADING (ESPECIALLY PHOSPHORUS).

IF THE ALGAE PROBLEM IS CAUSED BY THE BLUE-GREEN VARIETY, POSS-IBLE SOLUTIONS WOULD BE:

A. CORRECT THE OBVIOUS ORGANIC OVERLOADING THAT IS OCCURRING.
B. IF ORGANIC LOADING CANNOT BE REDUCED, CONSIDERATION FOR MECHANICAL SURFACE AERATION MUST BE CONSIDERED.
C. IF OPERATING PRIMARY CELLS IN SERIES, CONSIDER PARALLEL OPERATION.

D. IF SIGNIFICANT INDUSTRIAL LOADING IS PART OF THE RAW WASTE-WATER, CHECK NUTRIENT BALANCE OF BOD TO NITROGEN TO PHOS-PHORUS (THE RATE OF 100/5/1 SHOULD BE ADEQUATE FOR AEROBIC TREATMENT). CHECK FOR LOW pH.
E. IF LOADING AND NUTRIENTS ARE IN THE ACCEPTABLE RANGE, CON- SIDERATION CAN BE GIVEN TO AN APPROVED ALGICIDE.
F. IF OPERATIONAL CHANGES CANNOT CORRECT THE PROBLEM, FACILITY RE-DESIGN IS PROBABLY REQUIRED.

47. Describe what might be done if a system has unacceptable high effluent pH values.

IF HIGH EFFLUENT pH IS OCCURRING, IT WILL BE NECESSARY TO DET-ERMINE THE CAUSE OF THIS PROBLEM. IF IT IS CAUSED BY INFLUENT FLOWS, THE SOURCE OR SOURCES MUST BE FOUND AND CORRECTIVE ACTION TAKEN. MOST LIKELY, IT WOULD BE AN INDUSTRIAL SOURCE AND PRE-TREATMENT WOULD HAVE TO BE INSTITUTED.

HIGH EFFLUENT pH ATTRIBUTED TO NORMAL ALGAE PHOTOSYNTHESIS WOULD ALMOST BE IMPOSSIBLE TO CONTROL (RECIRCULATION COULD BE TRIED, BUT IT WOULD NOT BE VERY AFFECTIVE). THE ONLY OTHER ALTERNATIVE OF HIGH pH CAUSED BY ALGAE PHOTOSYNTHESIS WOULD BE TO APPLY FOR AN ADJUSTMENT IN pH LIMITATIONS IN THE DISCHARGE PERMIT.

48. Discuss the causes and corrective actions for a Pond with odor problems.

WHEN PROPERLY OPERATED AND LOADED, POND SYSTEMS WILL NORMALLY EXPERIENCE ODOR PROBLEMS ONLY IN THE SPRING, RIGHT AFTER ICE-OUT. THIS ODOR IS CAUSED BECAUSE OF ANAEROBIC CONDITIONS THAT OCCURRED UNDER THE ICE. IN MOST CASES, THIS CONDITION MAY ONLY LAST FROM A FEW DAYS TO A WEEK, UNTIL NORMAL AEROBIC CONDITIONS ARE RESTORED. WHEN A POND SYSTEM IS NOT OPERATED PROPERLY; WHEN RECEIVING AN INDUSTRIAL SLUG LOAD, OR, WHEN BEING OVERLOADED ORGANICALLY, ANAEROBIC CONDITIONS CAN PERSIST FOR SOME TIME WITH SIGNIFICANT ODORS FROM BOTH ANAEROBIC CONDITIONS AND THE DIE-OFF OF BLUE-GREEN ALGAE DOMINATING THE SYSTEM. THE POND SYSTEM MAY HAVE BLUE-GRAY APPEARANCE WITH THE ODOR.

TO CORRECT THIS SITUATION, THE OPERATOR SHOULD MAKE OPERATIONAL CHANGES (EG. FROM SERIES TO PARALLEL OPERATION, REDUCE ORGANIC OVERLOAD IF POSSIBLE, ISOLATE THE REST OF THE PROBLEM CELL, OR CONTROL OF ALGAE AND DUCKWEED). POSSIBLE CHEMICAL CONTROL FOR POND ODORS WOULD INCLUDE THE USE OF HYDROGEN PEROXIDE, SODIUM NITRATE OR MASKING AGENTS. IT IS ALWAYS BEST TO CORRECT ODOR PROBLEMS WITH OPERATIONAL CHANGES BEFORE RESORTING TO CHEMICAL MEANS. IF THE OPERATIONAL CHANGES CANNOT CORRECT THE PROBLEM, CHEMICALS CAN BE USED UNTIL THE FACILITY CAN BE RECONSTRUCTED, OR ADDITIONAL AERATION CAN BE PROVIDED.

49. Describe the consequences of not controlling floating and rooted weeds in a Pond system.

FLOATING WEED MATS COULD PREVENT SUNLIGHT FROM ENTERING THE POND, CAUSING ANAEROBIC CONDITIONS. FLOATING DUCKWEED, IF NOT REMOVED, WILL CONTINUE TO REPRODUCE AND MAKE THE PROBLEM WORSE. THESE MATS WILL BLOCK SUNLIGHT FROM ENTERING THE POND SLOWING ALGAE PHOTO-SYNTHESIS AND REDUCING OXYGEN PRODUCTION. THE POND COULD GO ANAEROBIC. MATS ALSO BLOW INTO DEAD ZONES OF THE POND AND REDUCE THE AFFECTIVE AREA OF THE TREATMENT POND, AND WOULD HINDER SURFACE AERATION BY REDUCING WIND TURBULENCE. ROOTED WEEDS COULD PIERCE THE POND SEAL AND LEAD TO LEAKS. THIS IS ESPECIALLY TRUE FOR WOODY VEGETATION. THE ROOTED WEEDS ARE FOOD AND COVER HABITAT FOR MUSKRATS. MUSKRATS BUILD DENS INTO THE BANKS WHICH ALSO LEAD TO SIGNIFICANT LEAKAGE. LARGE AMOUNTS OF ROOTED WEEDS IN THE POND COULD ALSO CAUSE SHORT-CIRCUITING.

50. List some burrowing animals that cause damage to dikes, and discuss control methods for each.

ANIMAL	CONTROL
A. GOPHERS AND BADGERS.	TRAPPING/SHOOTING (PERMIT REQUIRED).
B. MUSKRATS.	VARYING WATER LEVELS, REMOVE FOOD SOURCES (ROOT WEEDS) OR TRAPPING/ SHOOTING (PERMIT REQUIRED).
C. TURTLES(MINOR PROBLEM).	TRAPPING/SHOOTING (POSSIBLE PERMIT).

51. Discuss how to legally remove burrowing animals from a Pond system.
MUSKRATS, GOPHERS AND BADGERS, ARE FURBEARERS. THEIR REMOVAL AND POSSESSION IS SUBJECT TO DNR FURBEARER REGULATIONS. CONTACT SHOULD BE MADE WITH THE COUNTY DNR WARDEN FOR SPECIFICS ON HOW TO REMOVE ANIMALS AND WHAT PROCEDURES TO FOLLOW. THERE MAY BE A REQUIREMENT FOR SPECIAL DISPOSITION OF THE HIDES AND CARCASSES.

DNR WARDENS AND WILDLIFE MANAGERS HAVE THE AUTHORITY TO GIVE SPECIAL REMOVAL PERMITS. ASK FOR PERMISSION TO USE DEN SETS AND GROUP SETS. SOME OF THESE TRAPS ARE ILLEGAL, BUT THE POND OPERATOR CAN USE THEM TO SPEED REMOVAL. THE DNR CAN ALSO ISSUE PERMITS TO SHOOT ILLEGAL RATS.
DO NOT USE POISON BAIT AROUND BERMS. ANIMALS OR EVEN HUMANS MIGHT INGEST THE POISON. PREDATOR ANIMALS (HAWKS AND OWLS) MIGHT FEED ON A POISONED ANIMAL AND COULD DIE.

MODULE D: SAFETY AND CALCULATIONS

CONCEPT: SAFETY

52. Describe how a Pond could be judged an "Attractive Nuisance."

    THE TERM "ATTRACTIVE NUISANCE" IS A LEGAL EXPRESSION THAT IMPLIES THE POND COULD BE ATTRACTIVE TO POTENTIAL USERS, SUCH AS, DUCK HUNTERS, FISHERMAN OR PLAYING CHILDREN. SINCE PONDS HAVE FAIRLY STEEP SLOPES, THE POTENTIAL FOR SOMEONE FALLING-IN AND DROWNING IS A SIGNIFICANT LEGAL PROBLEM THAT MUST BE A CONCERN. IT IS IMPORTANT THAT ADEQUATE FENCING AND SIGNING BE PROVIDED.

53. Discuss reasonable Pond security precautions against trespassing and vandalism.

    SECURITY IS NECESSARY TO PROTECT THE AREA FROM UNAUTHORIZED ACCESS AND TO PROTECT THOSE WHO ENTER THE FACILITY. THE COMM-UNITY COULD BECOME SUBJECT TO LIABILITY AND LEGAL ACTION IF IT FAILS TO MAKE A REASONABLE EFFORT TO RESTRICT TRESPASSING.

    REASONABLE FENCING INCLUDES:

    A. GATES AND LOCKS WHICH ARE KEPT SECURE AT ALL TIMES. GATES TO RESTRICT VEHICLES AND ATV'S. AT A MINIMUM, STEEL OR ALUMINUM GATES WITH SOLID ANCHOR POSTS AND A SIGN ARE REQUIRED.
    B. FENCES INCLUDE A STURDY WIRE FENCE WITH SIGNS. FENCE LINES SHOULD BE BRUSHED AND SIGNED AT SUITABLE INTERVALS.
    C. REGULAR DRIVE-BY PATROL BY THE LOCAL POLICE IS RECOMMENDED. WORK WITH ADJACENT PROPERTY OWNERS TO REPORT SUSPICIOUS VEHICLES OR PEOPLE IN THE AREA.

54. List the personal safety precautions that should be practiced by persons operating a Pond system.

    A. DO NOT ENTER A MANHOLE ALONE, OR ANY CONFINED SPACE, WITHOUT PROPER EQUIPMENT AND SOMEONE TO ASSIST YOU.
    B. WEAR LIFE JACKETS WHEN WORKING AROUND PONDS.
    C. LEARN TO SWIM.
    D. WASH-UP AFTER CONTACT WITH SEWAGE.
    E. LOCK-OUT ELECTRICAL CIRCUITS TO SHUT-DOWN AERATORS. (THIS

REFERS TO THE TENDENCY OF AERATORS TO LOWER THE SPECIFIC GRAVITY OF WATER. A PERSON WHO FALLS OVERBOARD COULD SINK FASTER IF THE AERATORS WERE WORKING. OPERATORS HAVE BEEN KNOWN TO DROWN EVEN WITH LIFE JACKETS ON).

F. NEVER PERFORM ANY HAZARDOUS TASK AROUND A POND WITHOUT BEING ACCOMPANIED BY SOMEONE.

G. USE CARE WHEN MOWING OR TRIMMING GRASS AROUND BURIED ELECTRIC CONDUITS.

55. Discuss the risks involved while walking on the ice of a Pond to collect samples.

THE BIGGEST PROBLEM IN WALKING ON THE ICE OF A TREATMENT POND WOULD BE THE POSSIBILITY OF THE ICE BREAKING AND CAUSING A POTENTIAL DROWNING. INFLUENT WASTEWATER IS WARM ENOUGH TO CAUSE POSSIBLE THIN ICE NEAR THE INFLUENT PIPING. SAFETY PRECAUTIONS SHOULD BE USED WHEN GOING OUT ON THE ICE, SUCH AS: FLOTATION EQUIPMENT, A ROPE CONNECTED TO SHORE, LIFE JACKETS, AND ALWAYS BE ACCOMPANIED BY SOMEONE ELSE (IT WOULD ALSO BE ADVISABLE TO HAVE COMMUNICATION EQUIPMENT AVAILABLE SUCH AS A RADIO OR MOBILE TELEPHONE IN CASE OF EMERGENCY). ANOTHER RISK IS THE POSSIBILITY OF FALLING, WHICH CAN BE REDUCED BY GOOD FOOTWEAR.

56. Discuss the safety precautions that should be practiced while using grass cutting equipment around a Pond.

USE CAUTION WHEN CUTTING NEXT TO ELECTRICAL CABLES. USE CARE WHEN SPRAYING WEEDS AROUND ELECTRICAL CABLES AND EQUIPMENT. THE SPRAY COULD CONDUCT A CURRENT AND CAUSE ELECTRICAL SHOCK.

BE CAREFUL OPERATING MOWING EQUIPMENT ON BANKS. STEEP BANKS CAN BE VERY HAZARDOUS. ALL MOWERS SHOULD HAVE THROTTLE KILL-SWITCHES. MAKE SURE THE MANUFACTURERS EQUIPMENT OPERATION DIRECTIONS ARE UNDERSTOOD AND FOLLOWED.

CONCEPT: CALCULATIONS

57. Given data, calculate Pond surface area in acres.

GIVEN: POND LENGTH = 400 FEET
POND WIDTH = 300 FEET

FORMULA:

(ONE ACRE = 43,500 SQUARE FEET)

AREA OF POND = LENGTH (FT) X WIDTH (FT)
(IN SQ.FT.)

$$\text{AREA OF POND (IN ACRES)} = \frac{\text{SURFACE AREA (SQ.FT.)}}{\text{1 ACRE (SQ.FT.)}}$$

AREA OF POND = LENGTH (FT) X WIDTH (FT)
= 400 X 300
= 120,000 SQUARE FEET

$$\text{AREA OF POND (IN ACRES)} = \frac{\text{SURFACE AREA (SQ.FT.)}}{\text{1 ACRE (SQ.FT.)}}$$

$$= \frac{120{,}000}{43{,}560}$$

= 2.75 ACRES

58. Given data, calculate Pond volume in gallons.

GIVEN: POND WIDTH AT MID-DEPTH = 200 FEET
POND LENGTH AT MID-DEPTH = 500 FEET
POND DEPTH = 6 FEET

FORMULA:

AREA (AT MID-DEPTH) = LENGTH (FT) (AT MID-DEPTH) X WIDTH (FT) (AT MID-DEPTH)

VOLUME = AREA (AT MID-DEPTH) X DEPTH
1 CUBIC FOOT = 7.5 GALLONS

AREA = 200 X 500

= 100,000 SQ. FEET AT MID-DEPTH

VOLUME = 100,000 X 6

= 600,000 CU.FT.

$$\text{GALLONS} = \frac{\text{600,000 CU.FT.}}{\text{7.5GAL/CU.FT.}}$$

= 4,500,000 (4.5) MILLION GALLONS

59. Given data, calculate the volume of water discharged in gallons.

GIVEN: DRAWDOWN DEPTH = 3.0 FEET
POND LENGTH = 675 FEET
POND WIDTH = 420 FEET

FORMULA:

VOLUME = LENGTH (FT) X WIDTH (FT) X DRAWDOWN DEPTH (FT) X 7.5

(1 CUBIC FOOT = 7.5 GALLONS)

VOLUME = 675 X 420 X 3.0 X 7.5

= 6,380,000 GALLONS

= 6.4 MILLION GALLONS

60. Given data, calculate a lagoons detention time in days.

GIVEN: SURFACE AREA = 4 ACRES
AVERAGE DEPTH = 4 FEET
AVERAGE DAILY FLOW = 60,000 GALLONS/DAY

FORMULA:

(1 CU.FT. = 7.5 GALLONS)

$$\underset{\text{(GAL)}}{\text{VOLUME}} = \underset{\text{(SQ.FT./ACRES)}}{43{,}560} \text{ X ACRES X DEPTH X } 7.5$$

= 43,560 X 4 X 4 7.5

= 5,227,200 GALLONS

$$\underset{\text{(DAYS)}}{\text{DETENTION TIME}} = \frac{\text{VOLUME (GAL)}}{\text{AVG. DAILY FLOW (GAL/DAY)}}$$

$$= \frac{5{,}227{,}200}{60{,}000}$$

= 87.12 DAYS

RESOURCES

1. **CONTROLLING WASTEWATER TREATMENT PROCESSES.** (1984). Cortinovis, Dan. Ridgeline Press, 1136 Orchard Road, Lafayette, CA 94549.

2. **GROUNDWATER SAMPLING PROCEDURES.** Lindorff, David; Feld, Jodi; and, Connelly, Jack. PUBL. WR-168-87 (1987). Department of Natural Resources, Bureau of Water Resources,P.O. Box 7921, Madison, WI 53707.

3. **OPERATION OF MUNICIPAL WASTEWATER TREATMENT PLANTS.** Manual of Practice No.11 (MOP 11), 2nd Addition (1990), Volumes I,II,andIII. Water Environment Federation (Old WPCF), 601 Wythe Street, Alexandria, VA 22314-1994. Phone (800) 666-0206. http://www.wef.org/

4. **OPERATION OF WASTEWATER TREATMENT PLANTS.** 3rd Edition (1990), Volumes 1 and 2, Kenneth D. Kerri, California State University, 6000 J Street, Sacramento, CA 95819-6025. Phone (916) 278-6142. http://www.owp.csus.edu/training/

5. **OPERATION OF WASTEWATER TREATMENT PLANTS.** Manual of Practice No.11 (MOP 11)(1976). Water Pollution Control Federation, 601 Wythe Street, Alexandria, VA 22314-1994. Phone (800) 666-0206. (Probably Out-Of-Print, See Reference Number 3).

6. **OPERATIONS MANUAL: STABILIZATION PONDS.** Zickenfoose, Charles and Hayes, R.B.Joe EPA-430/9-77-012 (1977). U.S. Environmantal Protection Agency, Office of Water Program Operation, Washington, DC 20460.

7. **STABILIZATION POND OPERATION AND MAINTENANCE MANUAL.** Sexauer, Willard and Karn, Roger (1979). Operator Training Unit, Minnesota Pollution Control Agency, 1935 West County Road B-2, Roseville, MN 55113. Phone (612) 296-7373.

STUDY GUIDE

ADVANCED STABILIZATION PONDS AND AERATED LAGOONS

SUBCLASS D

WISCONSIN DEPARTMENT OF NATURAL RESOURCES
BUREAU OF SCIENCE SERVICES
OPERATOR CERTIFICATION PROGRAM
http://dnr.wi.gov/org/es/science/opcert/

NOVEMBER 1998 EDITION *

* NOTE – As of Jan 2010, this study guide contains objectives plus key knowledges.

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Jim Hansen, Northwest District, Park Falls
Janet Hopke, Northwest District, Spooner
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Tom Kroehn, Madison
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PREFACE

This operator's study guide represents the results of an ambitious program. Operators of wastewater facilities, regulators, educators and local officials, jointly prepared the objectives and exam questions for this subclass.

The objectives in this study guide have been organized into modules, and within each module they are grouped by major concepts.

NOTE: As of January 2010, this study guide also includes key knowledges.

HOW TO USE THESE OBJECTIVES WITH REFERENCES

In preparation for the exams, you should:

1. Read all of the key knowledges for each objective.
2. Use the resources listed at the end of the study guide for additional information.
3. Review all key knowledges until you fully understand them and know them by memory.

IT IS ADVISABLE THAT THE OPERATOR TAKE CLASSROOM OR ONLINE TRAINING IN THIS PROCESS BEFORE ATTEMPTING THE CERTIFICATION EXAM.

Choosing A Test Date

Before you choose a test date, consider the training opportunities available in your area. A listing of training opportunities and exam dates is available on the DNR Operator Certification home page http://dnr.wi.gov/org/es/science/opcert/ It can also be found in the annual DNR "Certified Operator" or by contacting your DNR regional operator certification coordinator.

TABLE OF CONTENTS

MODULE A: PRINCIPLE, STRUCTURE AND FUNCTION

CONCEPT: PRINCIPLE OF PONDS

1. Describe how the stabilization of organic waste material occurs in nature and in a wastewater treatment plant.

   STABILIZATION OF ORGANIC WASTE IS ACCOMPLISHED BY BACTERIAL DEGRADATION OF ORGANIC WASTE MATERIAL AEROBICALLY, ANAEROBICALLY OR A COMBINATION OF THE TWO. THE AEROBIC ORGANISMS ARE PROVIDED OXYGEN FROM PHOTOSYNTHESIS BY ALGAE.

2. Explain photosynthesis.

   PHOTOSYNTHESIS IS THE CREATION OF PLANT CELL MASS USING CARBON DIOXIDE, WATER, AND NUTRIENTS, WITH SUNLIGHT AS THE ENERGY SOURCE AND CHLOROPHYLL AS A CATALYST. DURING THIS PROCESS, FREE OXYGEN IS GIVEN-OFF.

3. Explain respiration.

   RESPIRATION IS THE PROCESS BY WHICH AN ORGANISM (PLANT OR ANIMAL) ASSIMILATES OXYGEN AND RELEASES CARBON DIOXIDE.

4. Relate photosynthesis and respiration to BOD removal.

   THE OXYGEN PRODUCED BY PHOTOSYNTHESIS CAN BE USED BY BACTERIA IN THEIR LIFE PROCESSES (RESPIRATION), THIS INCLUDES DEGRADING ORGANIC MATERIAL, WHICH REDUCES BOD.

5. Relate pH, carbon dioxide, and dissolved oxygen concentrations to photosynthesis and respiration.

   DURING <u>PHOTOSYNTHESIS</u>, GREEN PLANTS USE CARBON DIOXIDE AND PRODUCE OXYGEN. THIS CAUSES AN INCREASE IN DISSOLVED OXYGEN AND pH (THE pH INCREASE IS DUE TO THE LOSS OF DISSOLVED CARBON DIOXIDE WHICH WOULD NORMALLY FORM A WEAK CARBONIC ACID).
   DURING <u>RESPIRATION</u>, PLANTS OR ANIMALS USE DISSOLVED OXYGEN TO ASSIMILATE ORGANIC MATERIAL AND GIVE OFF CARBON DIOXIDE. THIS CAUSES A REDUCTION IN THE DISSOLVED OXYGEN AND pH (THE pH DROP IS CAUSED BY THE INCREASE IN CARBON DIOXIDE, CAUSING A WEAK CARBONIC ACID).

6. Explain why a Pond may violate pH permit limits during periods of intense photosynthesis.

INTENSE SUNLIGHT SPEEDS-UP ALGAE PHOTOSYNTHESIS. ALGAE USE UP CARBON DIOXIDE WHICH RAISES pH TO VERY HIGH LEVELS (11 + SU).

7. Discuss some innovative uses of Aerated Lagoon Systems.

REARING MINNOWS OR OTHER FOOD FISH. MINNOWS THAT FORAGE ON PLANKTONIC MATERIAL ARE A FOOD SOURCE FOR LARGER FISH.

THIS PROCESS WILL WORK PROVIDING THERE IS ENOUGH DISSOLVED OXYGEN. A CONCERN WOULD BE POSSIBLE AMMONIA TOXICITY.

CONCEPT: STRUCTURE AND FUNCTION

8. Identify the valve action necessary to bypass a Pond cell.

CLOSE THE INLET AND OUTLET VALVES ON THE UNIT TO BE BYPASSED. OPEN THE VALVE ON THE BYPASS LINE.

9. Discuss different flow patterns that are used in Multiple Pond treatment systems.

THERE ARE VARIOUS WAYS TO ROUTE THE HYDRAULIC FLOW THROUGH MULTI-PLE POND SYSTEMS. WITH PROPER VALVING, PONDS CAN BE OPERATED IN EITHER SERIES OR PARALLEL MODES. IN MOST CASES, MULTIPLE CELL TREATMENT IS DESIGNED AND OPERATED IN SERIES. IN A THREE POND SYSTEM, SERIES OPERATIONS WILL MINIMIZE ALGAE IN THE FINAL CELL. IF HIGH ORGANIC LOADING TO THE PRIMARY CELL IS OCCURRING, (ESPECIALLY DURING WINTER MONTHS AND ICE COVER), IT MAY BE DESIRABLE TO OPERATE THE CELLS IN PARALLEL TO REDUCE THE AFFECT OF THIS OVER-LOADING. THIS SHOULD BE CONSIDERED A SHORT TERM SOLUTION, AND IF CONTINUOUS OVERLOADING OCCURS, ACTION NEEDS TO BE TAKEN TO REDUCE THE OVERLOAD OR SYSTEM RE-DESIGN SHOULD BE EVALUATED.

10. Discuss the advantage of Helical Diffusers over Floating Mechanical Aerators.

HELICAL DIFFUSERS ARE MUCH LESS AFFECTED BY ICE BUILD-UP DURING WINTER WEATHER.

MODULE B: OPERATION AND MAINTENANCE

CONCEPT: OPERATION

11. Explain the theory of isolation of a Pond cell which is experiencing an algae bloom in a Series Pond System.

    ISOLATION OF A POND CELL WHICH IS EXPERIENCING AN ALGAE BLOOM GIVES THE CELL A CHANCE TO "REST" AND RECOVER.

12. Discuss the use of chemicals to control weeds.

    IF CHEMICALS ARE USED FOR POND WEED CONTROL, THEY MUST BE APPROVED FOR THAT SPECIFIC USE AND LABEL DIRECTIONS MUST BE FOLLOWED PRECISELY. IT MAY BE NECESSARY TO PROVIDE ADDITIONAL MONITORING FOR TOXICS. MANY TIMES, THE USE OF A SURFACTANT IS RECOMMENDED TO IMPROVE THE "WETTING" ABILITY OF THE MIXTURE SO IT ADHERES BETTER TO THE TREATED PLANTS.

13. Describe how Pond depth and bubble size affect aeration efficiency.

    THE DEEPER THE POND, THE LONGER THE CONTACT TIME BEFORE THE BUBBLES REACH THE SURFACE. THE SMALLER THE BUBBLES, THE MORE CONTACT SURFACE BETWEEN THE AIR AND WATER, WHICH INCREASES THE TRANSFER RATE.

14. Explain how to balance aerators within and between Ponds.

    BALANCING OF AERATION WITHIN AND BETWEEN PONDS IS ACCOMPLISHED BY USING THE VALVES ON THE MANIFOLD TO GET AN EVEN AGITATION PATTERN.

15. Discuss when Floating Aerators are used for temporary additional aeration capacity.

    FLOATING AERATORS ARE USED FOR ADDITIONAL AERATION CAPACITY TO HANDLE LARGER THAN EXPECTED ORGANIC LOADS DURING THE SUMMER MONTHS.

16. List the important issues to consider in developing a public relations program for a Pond system.

A. POST A NOTICE EXPLAINING THE PRINCIPLES OF OPERATION OF A POND SYSTEM.
B. DEVELOP AN EDUCATIONAL PROGRAM FOR LOCAL ELECTED OFFICIALS.
C. EXPLAIN TO THE PUBLIC HOW UNAUTHORIZED ENTRANCE AND USE CAN JEOPARDIZE A PONDS AFFECTIVENESS.
D. EXPLAIN WHY CHILDREN SHOULD NOT BE ALLOWED IN THE AREA OF THE PONDS "RAPID DEPTH".
E. EXPLAIN WHY YOU SHOULD STAY OFF THE ICE OF A POND AT ALL TIMES.
F. ALL PUBLIC INFORMATION AND EDUCATION PROGRAMS COULD BE DONE WITH PUBLIC NOTICE.

17. Explain why alternate discharges to seepage cells should be practiced in a Multiple Seepage Cell system.

THE ALTERNATE LOADING/RESTING OF SEEPAGE CELLS IS DONE FOR A NUMBER OF REASONS. ONE REASON IS TO ALLOW THE OPERATOR TO PHYSICALLY WORK-UP (DISK,ROTOTILL,OR DRAG) AND CLEAN THE CELL BOTTOM. IT ALSO ALLOWS THE OPERATOR TO CONTROL VEGETATION WITHIN THE CELL. FINALLY, WITH THE NEW GROUND WATER RULES, IT ALLOWS SPREADING THE LOAD OVER A LARGER AREA TO PREVENT GROUND WATER EXCEEDANCES. THE NEW GROUND WATER STANDARDS WILL CHANGE SEEPAGE CELL OPERATIONS TO MEET THESE STANDARDS. THIS MAY MEAN MORE CELLS (LARGER AREA) OR EVEN POSSIBLE DISCONTINUANCE OF SEEPAGE CELLS.

18. List the considerations a Pond operator would have to make when considering accepting septic tank waste.

A. BOD CONCENTRATION OF HAULERS LOAD.
B. SOLIDS LOADING OF THE LOAD.
C. D.O. CAPACITY.
D. GRIT.
E. HYDRAULIC LOADING.

NORMALLY, PONDS AND AERATED LAGOONS ARE NOT DESIGNED WITH HOLDING TANKS TO ACCEPT SEPTAGE

CONCEPT: MAINTENANCE

19. Identify the items to be included in a Preventive Maintenance plan for a Pond system.
MONITOR ALL EQUIPMENT: BLOWERS, CHECK VALVES, AIR DIFFUSER ORIFICES, DIKES, ALL PUMPS, CONTROL MANHOLES, AND SHEAR GATES. MAINTAIN SEEPAGE CELLS. A PLANNED MAINTENANCE PROGRAM WILL PRE-VENT PROBLEMS AND WILL IDENTIFY POTENTIAL CONCERNS BEFORE THEY ACTUALLY BECOME PROBLEMS. MAINTENANCE AT A POND SYSTEM INVOLVES SIMPLE HOUSEKEEPING ITEMS WHICH ARE CRITICAL TO GOOD TREATMENT.

<u>GOOD HOUSEKEEPING ITEMS ARE</u>:

A. REMOVE ANY SCUM WHICH IMPEDES OXYGEN TRANSFER AND CAUSES ODORS.
B. MOW DIKES TO THE WATER LINE TO KEEP WEEDS DOWN, DISCOURAGE BURROWING MUSKRATS, AND PROMOTE WIND MIXING.
C. MAINTAIN DIKES BY RESTORING ANY EROSION AND/OR FILL MUSKRAT DENS.
D. SKIM FLOATING DUCKWEED REGULARLY.
E. CONTROL CATTAILS REGULARLY.
F. PERFORM PREVENTIVE MAINTENANCE ON ALL MECHANICAL EQUIPMENT AS INSTRUCTED IN THE O&M MANUAL AND THE EQUIPMENT MANUFACTURERS MANUALS.
G. EXERCISE VALVES IN THE SYSTEM ON A REGULAR BASIS.

20. List the maintenance items on Aeration Equipment.

A. <u>PIPING</u>: CHECK ALL AIR PIPING, INCLUDING VALVES AND DIFFUSERS TO ENSURE THAT THERE ARE NO BLOCKAGES.

B. <u>CENTRIFUGAL BLOWERS</u>: CHECK OIL LEVELS, AIR FILTERS, RELIEF VALVES, AND DRIVE MOTORS.

C. <u>POSITIVE DISPLACEMENT BLOWERS</u>: MAINTAIN OIL LEVELS, AIR RELIEF VALVES, V-BELTS, AIR FILTERS, AND DRIVE MOTORS.

D. <u>FLOATING AERATORS</u>: MAINTAIN FLOATS, ELECTRIC LINES, CHECK OIL LEVELS, ANCHORS, DRIVE MOTORS. MAKE SURE IMPELLERS ARE NOT CLOGGED.

21. Explain how to clean clogged Air Diffusers.

CLEANING OF AIR DIFFUSERS IN POND SYSTEMS CAN BE DONE IN SEVERAL WAYS. IF THE PLUGGING IS MINOR, THE AIR FLOW CAN BE INCREASED BY SHUTTING DOWN SOME SECTIONS TO INCREASE THE AIR TO THE REMAINING SECTIONS OR BY INCREASING BLOWER OUT-PUT (IF POSSIBLE). ANOTHER CLEANING METHOD WOULD BE TO INTRODUCE HYDROGEN CHLORIDE OR OXYGEN /OZONE GAS THROUGH THE AIR LINES. IN SOME INSTANCES, DIVERS HAVE BEEN USED TO MECHANICALLY CLEAN THE DIFFUSERS (ROLLING TUBING THROUGH A FLEX TOOL OR OTHER METHODS). IF NONE OF THESE PROCEDURES WORK, THE LAST OPTION WOULD BE TO DRAW THE POND DOWN TO REPAIR/REPLACE DIFFUSERS.

22. Describe the function and maintenance of the Blower Inlet Filter.

THE INLET AIR FILTER REMOVES PARTICULATES FROM THE AIR BEFORE THE COMPRESSION STAGE SO DEBRIS DOES NOT GET INTO THE AIR LINE AND PLUG DIFFUSER ORIFICES. IT IS ALSO ESSENTIAL TO

PROTECT THE COMPRESSOR FROM ANY DAMAGE, ESPECIALLY FROM GRITTY MATERIALS. THE MAIN MAINTENANCE REQUIREMENT IS TO KEEP THE FILTER CLEAN. USUALLY, THIS IS DONE BY REMOVING THE FILTER AND BLOWING IT OUT WITH COMPRESSED AIR. THE FREQUENCY OF CLEANING IS DEPENDENT ON FILTER SIZE AND AMBIENT AIR QUALITY. OTHER MAINTENANCE ACTIV-ITIES SHOULD BE SPECIFIED BY THE MANUFACTURER OR AS LISTED IN THE O&M MANUAL. FAILURE TO ADEQUATELY CLEAN FILTERS CAN CAUSE REDUCED BLOWER AIR OUTPUT, AN OVERHEATED BLOWER, POSSIBLE DIFFUSER CLOGGING, AND POSSIBLE DAMAGE TO BLOWER AND DRIVE MOTOR.

23. Explain methods of controlling dike erosion.

THE MAIN METHODS FOR PREVENTING DIKE EROSION ARE PROPER DIKE VEGETATION AND THE USE OF RIP RAP AROUND THE NORMAL OPERATING POND LEVELS TO PREVENT EROSION FROM WAVE ACTION.

24. Discuss how to prevent ice damage to floating aeration equipment.

ICE DAMAGE OCCURS MOST OFTEN TO FLOATING AERATORS WHEN THEY TIP OVER. THE MOTORS AND POWER CABLES CAN BE DAMAGED OR BROKEN DURING TIPPING. THE BEST METHOD IS TO STABILIZE THEM WITH ADEQUATE GUY CABLES. SINCE OXYGEN REQUIREMENTS ARE LOWER IN THE WINTER, IT IS POSSIBLE TO PROTECT EQUIPMENT BY REMOVING SOME OF THE AERATORS.

MODULE C: MONITORING AND TROUBLESHOOTING

CONCEPT: MONITORING

25. Set-up a sampling schedule for a Fill and Draw Pond system.

SAMPLING LOCATIONS SHOULD BE ABOUT EIGHT FEET FROM EACH CORNER AND BELOW THE SURFACE OF THE POND. SAMPLES SHOULD BE COLLECTED ABOUT A WEEK PRIOR TO PROPOSED DISCHARGE. DURING THE ENTIRE DURATION OF DISCHARGE, CHECK THE POND LEVEL DAILY. SAMPLE AT THE CONTROL MANHOLE ON THE FREQUENCY SPECIFIED IN THE DISCHARGE PERMIT.
PRIOR TO DRAWING DOWN A POND, THE OPERATOR SHOULD SAMPLE THE POND CONTENTS FOR pH, BOD, AND SUSPENDED SOLIDS. IT IS ALSO NECESSARY TO DETERMINE THE VOLUME NEEDED TO HOLD FLOWS UNTIL THE NEXT DRAW-DOWN (USUALLY 180 DAYS).

26. Describe two ways to determine Dissolved Oxygen levels in a Pond.

A. USE A DISSOLVED OXYGEN METER.
B. PERFORM A WINKLER DISSOLVED OXYGEN TEST.

27. Discuss the requirements for groundwater monitoring.

CHANGES IN STATE LAW HAVE ESTABLISHED REQUIREMENTS FOR GROUND WATER (NR 140). THIS IS IMPORTANT FOR WASTEWATER SYSTEMS, ESPEC- IALLY LAND DISPOSAL SEEPAGE FACILITIES AND LAGOONS THAT MAY BE LEAKING. NORMALLY, UP-GRADIENT AND DOWN-GRADIENT WELLS WILL BE LOCATED TO DETERMINE IF A SYSTEM IS AFFECTING GROUND WATER. CONCERN AT MUNICIPAL TYPE TREATMENT PLANTS WOULD BE TOTAL DISS-OLVED SOLIDS, CHLORIDES, AND NITROGEN SERIES (AND MORE SPECIF-ICALLY, NITRATES). THESE PARAMETERS WILL BE THE POTENTIAL AREAS THAT POND SYSTEMS MIGHT EXPECT EXCEEDANCES OF THE GROUND WATER STANDARDS WHICH WILL REQUIRE OPERATIONAL CHANGES, DISCHARGE LOCATION CHANGES, OR FUTURE RECONSTRUCTION.

CONCEPT: TROUBLESHOOTING

28. Describe how to determine if a drop in Pond water levels is caused by seepage or evaporation.

A. CALIBRATE THE FLOW METER TO DETERMINE IF THE TOTALIZER IS WORKING PROPERLY.
B. CHECK THE RESULTS OF GROUNDWATER SAMPLES TO SEE IF DOWN-GRADE WELLS SHOW SIGNIFICANT CHANGES IN WATER QUALITY.

CALIBRATE THE FLOW METER TO DETERMINE IF YOU HAVE ACCURATE INFLUENT FLOW DATA. SET-UP A STAFF GAUGE TO ACCURATELY MEASURE POND ELEVATION. BY FILLING A 55-GALLON DRUM, OR SIMILAR HOLDING DEVICE WITH WATER, THIS CAN BE USED TO DETERMINE THE AFFECTS OF PRECIPITATION AND EVAPORATION. COLLECTING THIS DATA OVER A PERIOD OF TIME CAN BE USED TO DETERMINE THE RATE OF POND SEEPAGE.

29. List the chemical and non-chemical controls for the following Pond conditions:

A. Algae.
B. Rooted Weeds.
C. Duckweed.
D. Organic Overload.

CONDITIONS	CHEMICAL	NON-CHEMICAL
A. ALGAE	COPPER SULFATE	FILTRATION
B. ROOTED WEEDS	HERBICIDE	CUTTING, PULLING OR VARY POND LEVELS
C. DUCKWEED	HERBICIDE	WIND AND RAKE
D. ORGANIC OVERLOAD	SODIUM NITRATE	REDUCE LOAD AND USE AERATION

NOTE: **FOR CATTAIL CONTROL, HERBICIDES ARE USUALLY MOST AFFECTIVE DURING DEVELOPMENT. CUTTING CATTAILS BELOW THE WATER LINE IN FALL IS ALSO AN AFFECTIVE CONTROL METHOD.**

30. State the action to take if a polishing Pond produces worse suspended solids effluent than its influent.

THIS SITUATION IS NORMALLY A PROBLEM ASSOCIATED WITH AN ALGAE BLOOM. THE ALTERNATIVES TO CORRECT THIS PROBLEM WOULD BE TO BY-PASS THE POLISHING POND, OR TO ATTEMPT TO WITHDRAW EFFLUENT FROM A DIFFERENT ELEVATION.

31. List the conditions that might lead to solids build-up on the bottom of a Pond.

A. HIGH INFLUENT TSS.
B. EXCESSIVE WEED GROWTH.
C. OVERLOADING.
D. POOR TREATMENT.
E. HIGH INFLUENT BOD.
F. INORGANIC SOLIDS.

32. List some possible consequences of exceeding the design organic loading rate of a Pond system.

A. POOR TREATMENT.
B. HIGH EFFLUENT BOD.
C. INCREASE OF SLUDGE SOLIDS.
D. POTENTIAL FOR OBJECTIONABLE ODORS.
E. EXCESSIVE ALGAE (BLUE-GREEN FILAMENTOUS MATS).

33. Discuss the significance of long-term domination of a Pond by blue-green algae.

BLUE-GREEN ALGAE DOMINANCE OF A POND SYSTEM IS AN INDICATION OF INCOMPLETE OR POOR TREATMENT. THE PROBLEM WITH BLUE-GREEN ALGAE HAPPENS WHEN THE ALGAE DIES-OFF AND FOUL ODORS OCCUR. IF OPERATIONAL CHANGES CANNOT BE MADE TO ELIMINATE THE BLUE-

GREEN ALGAE, THEN CONSIDERATIONS NEED TO BE GIVEN TO PLANT RE-DESIGN.

34. Explain why a Pond receiving a white dairy waste might turn red.

HIGH PROTEIN WASTE IS CAUSING RED ALGAE TO BLOOM.

35. Describe when and how to use copper sulfate to achieve maximum control of algae.

USE COPPER SULFATE WHEN ALGAE BECOMES EXCESSIVE. USE LABEL DIRECTIONS FOR MIXING AND APPLYING THIS CHEMICAL. ADDITIONAL MONITORING FOR TOXICS MAY BE REQUIRED. A CONCERN THAT NEEDS WATCHING WHEN TREATING THE ENTIRE POND IS THAT DISSOLVED OXYGEN LEVELS CAN DECREASE DUE TO THE DIE-OFF AND DECOMPOSITION OF ALGAE.

36. List some alternatives to using Copper Sulfate for algae control.

A. INTRODUCTION OF FISH.
B. SPRAY THE PONDS WITH ANOTHER APPROVED ALGICIDE.
C. CHANGE OPERATIONAL MODE (IF POSSIBLE).
D. DISCHARGE EFFLUENT FROM A DIFFERENT POND LEVEL.
E. APPLY FOR AN ALGAE VARIANCE.

37. Describe short circuiting and possible causes and problems it creates.

SHORT CIRCUITING IS A HYDRAULIC CONDITION WHICH MAY OCCUR IN PARTS OF A POND WHEN THE FLOW PASSES THROUGH MORE QUICKLY THAN THE THEORETICAL DETENTION. THIS TYPE OF FLOW PATTERN REDUCES DETENTION TIME AS COMPARED WITH EVEN UNIFORM FLOW THROUGH THE POND.

SHORT CIRCUITING CAN BE CAUSED BY POOR DESIGN AND/OR CONSTRUCTION OF INLET AND OUTLET STRUCTURES, UNEVEN POND BOTTOMS, SHAPE OF THE CELLS, PREVAILING WINDS, AND EXCESSIVE GROWTH OF ROOTED WEEDS.

PROBLEMS ASSOCIATED WITH SHORT CIRCUITING INCLUDE: DEAD SPOTS, UNEVEN OXYGEN LEVELS, SLUDGE BUILD-UP, ODOR PROBLEMS, AND, A REDUCTION IN TREATMENT EFFICIENCY.

MODULE D: SAFETY AND CALCULATIONS

CONCEPT: SAFETY

38. List the characteristics of an affective safety program.

   A. RED CROSS FIRST AID TRAINING.
   B. C.P.R. TRAINING.
   C. PROPER EQUIPMENT OPERATION NEAR PONDS (MOWING, SNOW REMOVAL FROM DIKES, ETC.).
   D. WEARING PROPER APPAREL WHEN ENTERING CONTROL STRUCTURES.
   E. CONFINED ENTRY TRAINING.
   F. WATER SAFETY COURSE TRAINING.
   G. UNDERSTANDING USAGE OF CHEMICALS.

39. List some Pond security measures.

   A. FENCING TO PREVENT ANY UNAUTHORIZED ENTRY.
   B. ERECTING SIGNS WITH PROPER MESSAGE.
   C. PASSING AN ORDINANCE TO REGULATE USE OF THE AREA AND PENALIZE VIOLATORS.

CONCEPT: CALCULATIONS

40. Given data, calculate pounds BOD per acre per day.

GIVEN: POND SURFACE = 6.2 ACRES
AVERAGE DAILY FLOW = 50,000 GPD
INFLUENT BOD_5 = 220 mg/L

FORMULA:

$$\text{SURFACE LOADING RATE} = \frac{\text{POUNDS OF BOD PER DAY}}{\text{POND SURFACE AREA}}$$

POUNDS OF BOD/DAY = CONCENTRATION(mg/L) X FLOW(MG) X 8.34

= 220 X .05 X 8.34

= 91.7 Pounds BOD/DAY

SURFACE LOADING RATE = 91.7

14

6.2

= 14.8 POUNDS BOD/ACRE/DAY

41. Given data, calculate the cost of a chemical ($/Pound) needed to control Duckweed.

GIVEN: POND SURFACE AREA = 12.5 ACRES
APPLICATION RATE = 2.1 PER ACRE
CHEMICAL COST = $8.75 PER ACRE

FORMULA:

COST = AREA X APPLICATION RATE X COST
($) (ACRES)

= 12.5 X 2.1 X 8.75

= $230

42. Given data, calculate the theoretical detention time of a Pond.

GIVEN: VOLUME = 5.2 MGD
FLOW = .05 MGD

FORMULA:

$$\text{DETENTION TIME} = \frac{\text{VOLUME (MGD)}}{\text{FLOW RATE (MGD)}}$$

(FOR POND SYSTEMS, DETENTION TIME IS USUALLY EXPRESSED IN DAYS)

43. Given data, calculate a discharge flow rate to achieve a given Pond draw-down.

GIVEN: POND DIMENSIONS = 200 FEET X 400 FEET
(AT MID-POINT OF DRAWDOWN)
DRAWDOWN DESIRED = 4 FEET
DURATION OF DRAWDOWN = 100 HOURS
(ONE CUBIC FOOT = 7.5 GALLONS)

FORMULA:

$$\text{FLOW RATE} = \frac{\text{VOLUME TO BE DISCHARGED}}{\text{DURATION OF DRAW-DOWN}}$$

$$= \frac{\text{200 FT. X 400 FT. X 4 FT. X 7.5}}{\text{100 HOURS X 60 MIN./HR.}}$$

= 400 GALLONS PER MINUTE

44. Given data, for a Fill and Draw Pond system, calculate the amount of draw-down required and the time required to achieve draw-down.

GIVEN:

$$\text{AMOUNT OF DRAW-DOWN REQ.} = \frac{\text{VOLUME REQ.FOR DESIRED STORAGE TIME}}{\text{VOLUME PER FOOT OF DEPTH}}$$

$$\text{TIME REQ. FOR DRAW-DOWN} = \frac{\text{VOLUME OF DRAW-DOWN NEEDED}}{\text{MAXIMUM DRAW-DOWN RATE}}$$

(1 CUBIC FOOT = 7.5 GALLONS)

A POND IS BEING OPERATED FILL AND DRAW WITH THE OPERATOR DRAWING-DOWN A POND TO MEET A DESIRED DETENTION TIME OF 180 DAYS. THE POND DIMENSIONS ARE 400 FEET BY 600 FEET AT AVERAGE DEPTH. THE MEASURED WATER DEPTH IS 6 FEET, WITH THE MAXIMUM OPERATING DEPTH OF 6 FEET, AND THE MAXIMUM DRAW-DOWN RATE OF 0.5 FEET PER DAY. THE INFLUENT FLOW TO THE SYSTEM IS 30,000 GPD. WHAT IS THE MINIMUM NUMBER OF DAYS THAT IT WILL TAKE TO DRAW THE POND DOWN?

45. Given data, calculate the volume of water in a groundwater monitoring well casing.

GIVEN: INSIDE WELL CASING DIAMETER = 2 INCHES
DEPTH OF WATER = 15 FEET

FORMULA:

$$\text{VOLUME (GALLONS)} = 3.14 \text{ X } R^2 \text{ X DEPTH X } 7.5$$

(1 CUBIC FOOT = 7.5 GALLONS)
(1 CUBIC FOOT = 1728 CUBIC INCHES)

$$\text{VOLUME} = \frac{3.14 \text{ X } 1 \text{ X } 1 \text{ X } (15\text{(FT) X } 12\text{(IN) X } 7.5}{1728}$$

VOLUME = 2.45 GALLONS

RESOURCES

1. **CONTROLLING WASTEWATER TREATMENT PROCESSES**. (1984). Cortinovis, Dan. Ridgeline Press, 1136 Orchard Road, Lafayette, CA 94549.

2. **GROUNDWATER SAMPLING PROCEDURES**. Lindorff, David; Feld, Jodi; and, Connelly, Jack. PUBL. WR-168-87 (1987). Department of Natural Resources, Bureau of Water Resources,P.O. Box 7921, Madison, WI 53707.

3. **OPERATION OF MUNICIPAL WASTEWATER TREATMENT PLANTS**. Manual of Practice No.11 (MOP 11), 2nd Addition (1990), Volumes I,II,andIII. Water Environment Federation (Old WPCF), 601 Wythe Street, Alexandria, VA 22314-1994. Phone (800) 666-0206. http://www.wef.org/

4. **OPERATION OF WASTEWATER TREATMENT PLANTS**. 3rd Edition (1990), Volumes 1 and 2, Kenneth D. Kerri, California State University, 6000 J Street, Sacramento, CA 95819-6025. Phone (916) 278-6142. http://www.owp.csus.edu/training/

5. **OPERATION OF WASTEWATER TREATMENT PLANTS**. Manual of Practice No.11 (MOP 11)(1976). Water Pollution Control Federation, 601 Wythe Street, Alexandria, VA 22314-1994. Phone (800) 666-0206. (Probably Out-Of-Print, See Reference Number 3).

5. **OPERATIONS MANUAL: STABILIZATION PONDS**. Zickenfoose, Charles and Hayes, R.B.Joe EPA-430/9-77-012 (1977). U.S. Environmantal Protection Agency, Office of Water Program Operation, Washington, DC 20460.

6. **STABILIZATION POND OPERATION AND MAINTENANCE MANUAL**. Sexauer, Willard and Karn, Roger (1979). Operator Training Unit, Minnesota Pollution Control Agency, 1935 West County Road B-2, Roseville, MN 55113. Phone (612) 296-7373.

APPENDIX G

Discharge Guidance

Discharge Guidance and Procedure

Minnesota Pollution Control Agency

1

Introduction

- Minimum detention time required
  - 180 or 210 days
- Designed as a controlled discharge system
  - Spring/Fall discharge
- Must avoid discharge during "Problem Discharge Period"
  - Summer or Winter
  - Need adequate flow and reaeration capacity of receiving waters

2

Changes to Acceptable Discharge Periods

- Modified with permit reissuance
- Must continue with dates currently in permit
- Spring – March 1 instead of April 1
- Fall – December 31 instead of December 15

3

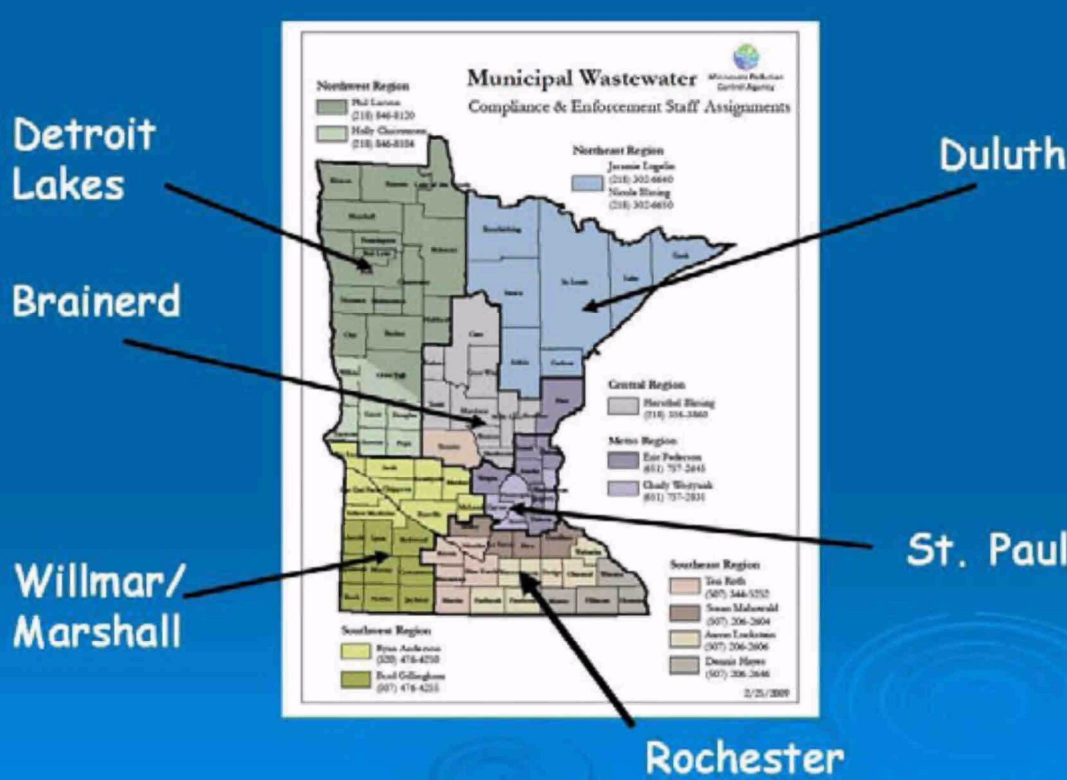
Revised Acceptable Discharge Periods

- MPCA Regional Offices
- Detroit Lakes, Brainerd, Duluth
- Spring
  - March 1 to June 30
- Fall
  - Sept 1 to Dec 31

- MPCA Regional Offices
- Willmar/Marshall St. Paul, and Rochester
- Spring
  - March 1 to June 15
- Fall
  - Sept 15 to Dec 31

4

MPCA Regional Offices @ 800-657-3864



5

POND DISCHARGE PROCEDURE

1. Is discharge needed; if so how much ?
2. Sampling prior to discharge
3. Is discharge notification required ?
4. Actual discharge
5. Sampling during discharge
6. Complete monthly report

6

Is Discharge Needed ? If so, how much ?

- What level are ponds at ?
- How much wastewater will I receive ?
- How many days do I provide storage for ?
- Calculate discharge volume (in feet)

7

Discharge Calculation Worksheet

- Will determine amount of discharge in feet
- Pond Discharge Calculation Worksheet Master.xls

8

Sampling Prior to Discharge

- Pre-discharge sampling required (Max. 2 weeks prior to discharge)
  - CBOD, TSS, Fecal Coliform, DO, pH, Total Phosphorus
- Four-sided composite sample
  - CBOD, TSS, Total Phosphorus
- Total phosphorus sample container
  - Special container
- Separate sterilized grab sample for fecal coliform
- Grab samples for dissolved oxygen and pH
  - within 24 hours prior to discharge

9

Is Discharge Notification Required ?

- Notification <u>not required</u> if
  - <u>All</u> samples results meet permit limits
  - Within "Acceptable Discharge Period"
- Notification <u>required</u> if
  - One or more sample results <u>do not</u> meet permit limits, or
  - Time between pre-discharge sample and discharge exceeds two weeks, or
  - If any of the discharge is in "Problem Discharge Period", or
  - If any of the discharge is to ice covered receiving waters
- Notification (if required) is to MPCA Regional Office

10

Actual Discharge

- Allowable discharge rate
  - six inches per day
- Can exceed after discussion with MPCA Regional office
- Avoid discharge to ice covered receiving waters
- If unavoidable, call MPCA Regional Office

11

Sampling Required During Discharge

- Grab sample from discharge control structure
- CBOD, TSS, Fecal Coliform, DO, pH, Total Phosphorus
  - Fecal and phosphorus in special container
- Twice per week (once every 3-4 days)

12

Complete Monthly Reports

- Supplemental Monthly Report
  - -Record discharge results
  - -Do NOT include pre-discharge results
- Discharge Monitoring Report (DMR)
  - -Summarize calculations
- Postmarked by 21st day of following month

13

Monthly Supplemental Report

Supplemental Report Form

Facility Name ______________________
Month ____________________, 20____

Permit Number ____________
Outfall Number ____________

DATE	DAY OF WEEK	PRECIPITATION (INCHES)	INFLUENT FLOW (MGD)	EFFLUENT FLOW (MGD)	INFLUENT CBOD5 (mg/L)	EFFLUENT CBOD5 (mg/L)	PERCENT REMOVAL CBOD5	EFFLUENT CBOD5 (kg/day)	INFLUENT TSS (mg/L)	EFFLUENT TSS (mg/L)	PERCENT REMOVAL TSS	EFFLUENT TSS (kg/day)	INFLUENT pH	EFFLUENT pH	EFFLUENT PHOSPHORUS (mg/L)	EFFLUENT PHOSPHORUS (kg/day)	EFFLUENT DO (mg/L)	FECAL COLIFORM (number/100 ml)	EFFLUENT CHLORINE RESIDUAL (mg/L)	CHLORINE USED (lbs)	DECHLORINATION AGENT USED (lbs)
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21
1																					
2																					
3																					
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31																					
Total																					

This form replaces the 703 and 704 forms, and should be submitted with your preprinted Discharge Monitoring Report (DMR) forms. 1/00

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Monthly Supplemental Report

Minnesota Pollution Control Agency

SUPPLEMENTAL DATA and COMMENTS

Comments—Include information on violations, bypasses/overflows, maintenance conducted, etc. (Attach additional pages if necessary.)

Weekly Observations for Stabilization, Aerated, Polishing or Absorption Ponds

TYPE OF POND (AERATED, PRIMARY, ETC.)	_____ POND _____ ACRES					_____ POND _____ ACRES					_____ POND _____ ACRES				
WEEK OF THE MONTH	1ST	2ND	3RD	4TH	5TH	1ST	2ND	3RD	4TH	5TH	1ST	2ND	3RD	4TH	5TH
1. DATE OF OBSERVATION															
2. ODOR (YES OR NO)															
3. AQUATIC PLANTS (% OF COVERAGE, TYPE)															
4. FLOATING MATS (% OF COVERAGE, TYPE)															
5. WATER DEPTH (UNITS_____)															
6. MUSKRATS, RODENTS, ETC. (YES OR NO)															
7. DIKE CONDITION (EROSION, ETC.)															
8. ICE COVER (% OF COVERAGE)															

Send with preprinted DMRs to:

Minnesota Pollution Control Agency
520 Lafayette Road North
Saint Paul, Minnesota 55155-4194
Attention: Discharge Monitoring Reports

____
Signature of Authorized Agent

____
Signature and Phone Number of Certified Operator

1/00

15

Discharge Monitoring Report
Influent

FACILITY NAME/ADDRESS:

WASTEWATER TREATMENT
DISCHARGE MONITORING REPORT

PERMITTEE NAME/ADDRESS:

PERMIT #	LIMIT STATUS	FORMER #
	FINAL	

STATION INFORMATION:
WS-001 (Influent waste stream)
Waste Stream, Influent Waste

MONITORING PERIOD
FROM YEAR MO. DAY 2003/09/01 TO YEAR MO. DAY 2003/09/30

No Flow

MPCA LB

PARAMETER		QUANTITY		UNITS	CONCENTRATION			UNITS	FREQUENCY OF ANALYSIS	SAMPLE TYPE
Precipitation	SAMPLE VALUE	------		in	------	------	------	****		
00193	PERMIT	------	REPORT CalMoTot		------	------	------		1 x Day	Measur
Flow	SAMPLE VALUE	------		MG	------			mgd		
50050	PERMIT	------	REPORT CalMoTot		------	REPORT CalMoAvg	REPORT CalMoMax		1 x Day	MeaCon
CBOD 05 Day (20 Deg C)	SAMPLE VALUE	------	------	****	------		------	mg/L		
80082	PERMIT	------	------		------	REPORT CalMoAvg	------		1 x Month	Comp-4
TSS	SAMPLE VALUE	------	------	****	------		------	mg/L		
00530	PERMIT	------	------		------	REPORT CalMoAvg	------		1 x Month	Comp-4
pH	SAMPLE VALUE	------	------	****	------	------		SU		
00400	PERMIT	------	------		------	------	REPORT SingleVal		1 x Month	Grab
Phosphorus Total (as P)	SAMPLE VALUE	------	------	****	------	------		mg/L		
00665	PERMIT	------	------		------	------	REPORT SingleVal		1 x Month	Comp-4

Send original with supplemental DMR (if applicable) by the 21st day of month following reporting period to:
MINNESOTA POLLUTION CONTROL AGENCY
520 LAFAYETTE RD
ST. PAUL, MN 55155-4194
ATTN: Discharge Monitoring Report

I certify that I am familiar with the information contained in this report and that to the best of my knowledge and belief the information is true, complete, and accurate.

SIGNATURE OF PRINCIPAL EXECUTIVE OFFICER OR AUTHORIZED AGENT | DATE

SIGNATURE OF CHIEF OPERATOR | PHONE | DATE | CERTIFICATION#

COMMENTS:

Page 02 of 02

16

Discharge Monitoring Report
Effluent

FACILITY NAME/ADDRESS:

WASTEWATER TREATMENT
DISCHARGE MONITORING REPORT

PERMITTEE NAME/ADDRESS:

PERMIT #	LIMIT STATUS	FORMER #
	FINAL	

STATION INFORMATION:
SD 001 (Total Facility Discharge)
Surface Discharge, Effluent To Surface Water

MONITORING PERIOD
FROM YEAR MO. DAY 2003/09/01 TO YEAR MO. DAY 2003/09/30

No Discharge

MPCA LB

PARAMETER		QUANTITY		UNITS	CONCENTRATION			UNITS	FREQUENCY OF ANALYSIS	SAMPLE TYPE
Flow	SAMPLE VALUE	------		MG	------		------	mgd		
50050	PERMIT	------	REPORT CalMoTot		------	REPORT CalMoAvg	------		1 x Day	MeaCon
CBOD 05 Day (20 Deg C)	SAMPLE VALUE			kg/day	------			mg/L		
80082	PERMIT	23.1 CalMoAvg	37 MxCalWkAvg		------	25.0 CalMoAvg	40.0 MxCalWkAvg		2 x Week	Grab
TSS	SAMPLE VALUE			kg/day	------			mg/L		
00530	PERMIT	41.6 CalMoAvg	60.0 MxCalWkAvg		------	45.0 CalMoAvg	65.0 MxCalWkAvg		2 x Week	Grab
pH	SAMPLE VALUE	------	------	----		------		SU		
00400	PERMIT	------	------		6.0 CalMoMin	------	9.0 CalMoMax		2 x Week	Grab
Phosphorus Total (as P)	SAMPLE VALUE	------	------	----	------		------	mg/L		
00665	PERMIT	------	------		------	REPORT CalMoAvg	------		2 x Week	Grab
Fecal Coliform, MPN/ Membrane Fltr 44.5C	SAMPLE VALUE	------	------	----	------		------	#/100ml		
46201	PERMIT	------	------		------	200 CalMoGeoMn	------		2 x Week	Grab
Oxygen, Dissolved	SAMPLE VALUE	------	------	----		------	------	mg/L		
00300	PERMIT	------	------		REPORT CalMoMin	------	------		2 x Week	Grab

Send original with supplemental DMR (if applicable) by the 21st day of month following reporting period to:
MINNESOTA POLLUTION CONTROL AGENCY
520 LAFAYETTE RD
ST. PAUL, MN 55155-4194
ATTN: Discharge Monitoring Report

I certify that I am familiar with the information contained in this report and that to the best of my knowledge and belief the information is true, complete, and accurate.

SIGNATURE OF PRINCIPAL EXECUTIVE OFFICER OR AUTHORIZED AGENT | DATE

SIGNATURE OF CHIEF OPERATOR | PHONE | DATE | CERTIFICATION#

COMMENTS:

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Discharge During Problem Discharge Periods-Winter

- Winter (1/1 to 2/28)
- Notification required
  - MPCA Regional Office @ 800-657-3864
- Ice covered receiving waters
  - -100% ice coverage from bank to bank
  - -Concern is adequate receiving waters D.O.
  - -Violation of permit; regardless of date and sample results
  - -If unavoidable, must have adequate dilution ratio
  - -If dilution is not available, receiving waters sampling is required
  - -<u>Can not</u> begin until two weeks after ice cover has left
  - -Must complete Discharge Evaluation Report

18

Discharge During Problem Discharge Periods-Summer

- Summer (7/1 to 8/31 or 6/16 to 9/14)
- Notification required (MPCA Regional Office)
- Pre-discharge samples <u>MEET</u> permit limits
  - -Must calculate dilution ratio
  - -Receiving water flow Vs. discharge flow
  - -Discharge OK if adequate dilution is available
  - -Lack of dilution ratio
    - -receiving waters monitoring required (D.O.)
    - -could reduce discharge rate
    - -could discharge continuously (overflow at maximum depth)
  - -Subject to enforcement action
  - -Complete Discharge Evaluation Report

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Discharge During Problem Discharge Periods-Summer

- Pre-discharge samples DO NOT meet permit limits
  - -Resample and/or take corrective action
  - -Must calculate dilution ratio
  - -Lack of dilution ratio
    - -receiving waters monitoring required (D.O.)
    - -could reduce discharge rate
    - -could discharge continuously
  - -Adequate dilution ratio
    - -only effluent monitoring required
  - -Subject to enforcement action
  - -Complete Discharge Evaluation Report

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CBOD Vs. Dilution Ratio

CBOD Concentration (mg/L)	Dilution Ratio Receiving waters Vs. Discharge rate
Less than 5	No min. dilution ratio needed
5 - 10	3 : 1
11 - 15	5 : 1
16 - 20	7 : 1
21 - 25	10 : 1
Greater than 25	Call MPCA Regional Office

21

Dilution Ratio Worksheet

1. Calculate discharge rate in cubic feet per second (cfs)
   - At 6 inches per day, take size of pond (in acres) divided by 4
2. Calculate flow rate in receiving waters in cfs
   - Measure width, depth, and stream flow
   - surface flow is typically 70% faster than actual flow

Example: Secondary pond is 16 acres.

Discharge rate of 6 inch per day is 16 acres/4 = 4 cfs

Receiving waters is 5 feet across, average depth is 2 feet and the time for an object to travel 20 feet was 5 seconds. Receiving water flow rate is:

5 ft x 2 ft x 20 feet/5 second x 0.7 = 28 cfs

22

Dilution Ratio Worksheet

$$\text{Dilution Ratio} = \frac{\text{Stream flow}}{\text{discharge flow}} = \frac{28\text{ cfs}}{4\text{ cfs}} = 7 : 1$$

- From dilution ratio chart, adequate dilution will be available as long as CBOD is 20 mg/L or less

CBOD Concentration (mg/L)	Dilution Ratio Receiving waters Vs. Discharge rate
Less than 5	No min. dilution ratio needed
5 – 10	3 : 1
11 – 15	5 : 1
16 – 20	7 : 1
21 - 25	10 : 1
Greater than 25	Call MPCA Regional Office

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Discharge Evaluation Report

- Facility Information
- Hydraulic Capacity Evaluation
- Organic Capacity Evaluation
- Discharge Evaluation

24

APPENDIX H

Guidance for Deploying Barley Straw

APPENDIX H

GUIDANCE FOR DEPLOYING BARLEY STRAW

Using Barley Straw to Reduce Algal Growth in Wastewater Treatment Pond Systems
Charles E. Corley, Illinois Environmental Protection Agency

Q: If you are not Anheuser Busch, what does one do with barley?

A: The days of filling a feedbag of a horse-drawn milk wagon being long gone, the sensible use is float it, the straw portion at least, on your pond, ditch, impoundment or reservoir. Why? Well, as some people have found from German Valley to Manchester, Illinois and England respectively, your water will be clearer, cleaner and lower in suspended solids due to algae.

Studied in the UK, the decomposition of barley straw and the observed effects on algae has been recorded since the late 1980's by multiple researchers including Dr. Jonathon Newman, University of Bristol, Department of Agricultural Sciences, Reading, U.K. Over ten year's worth of observations has distinguished "rotting barley straw" as an effective inhibitor of the color and suspended solids attributed to various types of algae. The research was done in "impoundments", slow moving "canals", and many other bodies of water; and has been confirmed by laboratory studies. This has led researchers to propound: "Decomposing barley straw inhibits the growth of both filamentous and blue-green algae species in all types of water bodies so far assessed".[1]

What causes barley to be so effective is not truly identified. Again, researchers have analyzed many chemical constituents produced by rotting barley straw.[2] No one chemical is predominant and the combined effect appears to be the controlling factor. Not the presence of the straw, but the decomposition products appears to provide the effect. Other straws and plant material have been tested and dismissed in preference to barley.[3] For example, green plant material like alfalfa and hay impart an organic load on the system while wheat straw, corn and lavender stalks, two quite common Illinois plant materials the latter less so, seem to have poorer effects and longevity. Despite uncertainty of the exact mechanism or product that produces the benefit, a benefit it is. One easily measured and observed, at that.

Transferring the technology, if one can refer to rotting barley straw as technology, to wastewater systems at best would seem a stretch. Newman's own studies indicate that algae growth continues with sufficient nutrient concentrations.[4] Further, algae and fungi appear to be affected while all other aquatic animal and plant life are not. Nor is dissolved oxygen. Therefore, no detrimental conditions would be expected and, if any benefit accrues in wastewater systems, all would be positive steps.

The application, of this truly natural and beneficial product, to water bodies of all types is fundamentally simple. Bundle it, float it and watch it rot! No need to search for the "right" type of barley straw or the vintage year, if there is one. Contact the nearest, cheapest and most readily available source and have at it. A slight oversimplification perhaps, but the years of observation have demonstrated these few basics. All confirmed by trials in Illinois communities and industries.

What few basics tenets have been displayed in use in Illinois include these: First, the straw must be floating throughout the application period. When allowed to sink, the thought is that it becomes a detrimental organic load. Secondly, since the original uses were in surface water ponds and impoundments and not wastewater systems, repeated applications are necessary form spring thru warm weather. Warm weather and wind action on the surface are two necessary ingredients. Also, keeping the straw loosely packed inside a long open-web material such as common snow fence is ideal and preferred to the more open-weave Christmas wrap where straws can escape.

Success can be found in all corners of our state. From Gardner to Ohio and Sorrento to Hudsonville barley straw decomposition abounds. Measurable and observable benefit without any detrimental environmental effects abound. First used in Gardner at the wastewater pond system, it reduced the use of copper sulfate while improving the effluent suspended solids for weeks in the hottest part of the summer of 2000. The operator at Sorrento experienced similar benefits during the summer of 2002 at the water plant where lower turbidity was demonstrated and fewer applications of copper sulfate were needed. These two have a sided benefit of reduced applications and reduced cost of an admittedly useful but hazardous material, copper sulfate. Other wastewater applications include the ash pond treatment at the Amerens Hudsonville Generating station. Barley straw here reduced the algae count in the effluent along with the suspended solids while positively affecting the pH of the discharge to the Wabash River. Using the straw at Ohio was done late in the summer in 2001. Not expecting a huge margin of success as a result of sludge pockets in all pond cells, the floating barley straw booms were effective in keeping the effluent suspended solids from exceeding the permitted limits for weeks.

These and others stories could be repeated throughout the Illinois with willing participants and experimentation-minded communities.[5.] Who knows, the result might be cleaner, clearer ponds with fewer green discharges to Illinois' surface waters. Better water quality. What a concept![6.]

Materials List and Cost:
Note: An application rate of 20 grams straw/m2 is the same as 1oz / sq yd.
Four to five forty-pound bales of barley straw for each acre @ $5 – $35
Two 100 ft. rolls of snow fence @ $20
Two rolls 350 lb test polyethylene rope @ <$10
Two fence post @ $1.89
Sixty one-gallon and half-gallon bottles
One tube silicon sealant @ $2.95
Nylon zip ties @ lowest cost
Two and a half hours on a sunny day.

Supplies and Method:
Flotation: Both gallon and half gallon bottles spaced 5 feet apart
Sealant: Silicon seal the inside of bottle caps
Configuration: Sausage Boom
Size: Two @ 95 ft. in series or parallel (or divide length as needed)
Location: Diagonally, upwind, in mixing pattern
Anchor: Double strands of poly rope tied to posts.

Documentation:
Determine BOD and TSS loading on all cells and compare to the design
Sample early spring, before algae bloom starts or before first application
Sample treated cell influent and effluent TSS weekly
Compare results to same times in previous years

(Send your results to the Illinois EPA Rockford Regional Office or Charles Corley, 815 987-7760.)

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1. Newman, Jonathon R and Barrett, P.R.F.; “Control of Microcystis aeruginosa by Decomposing Barley Straw”. 1993, Aquatic Plant Management. 31: 203 - 206

2. Newman, Jonathon (1999) “Information Sheet 3 – The Control of Algae Using Straw”. Copyright ICAR-Centre for Aquatic Plant Management.

3. ibid

4. ibid

5. No conflict with the EPA Federal Insecticide Fungicide and Rodenticide Act (FIFRA) would be expected when used in the privacy of one’s own non-public water-body or pond; which the above were. In fact, barley straw has been promoted without apparent conflict in the landscape pond industry for decades.

6. Further information or a presentation of success stories in Illinois can be obtained from Charles Corley, Rockford Region 815 987-7760 charles.corley@epa.state.il.us or from any Regional office.