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Subsurface Flow Constructed Wetlands For WasteWater Treatment

A Technology Assessment

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EXECUTIVE SUMMARY

Interest in, and the utilization of, constructed wetlands for treatment of a variety of wastewaters has grown rapidly since the mid 1980s. However, a lack of consensus has resulted in the use of different, and often conflicting, criteria and guidance for the design of these systems. Therefore a better understanding of the internal renovative processes in these systems was essential for the future application ‘of this promising technology, and to ensure reliable and cost-effective design procedures. A consensus on the report contents was reached via several review cycles by an internationally recognized panel of experts and via discussions at a two-day workshop in September 1992.

Two types of constructed wetlands are in common use: the first type, the free water surface (FWS) wetland, exposes the water surface in the system to the atmosphere. The second type, the subsurface flow (SF) wetland, maintains the water level below the surface of gravel or other media placed in the wetland bed. This report is concerned with the SF wetland type.

This report verifies that SF constructed wetlands can be a reliable and cost-effective treatment method for a variety of wastewaters. These have included. domestic, municipal, and industrial wastewaters as well as landfill leachates. Applications range from single family dwellings, parks, schools, and other public facilities to municipalities and industries. It can be a low-cost, low-energy process requiring minimal operational attention. As such the concept is particularly well suited for small to moderate sized facilities where suitable land may be available at a reasonable cost. Significant advantages include lack of odors, lack of mosquitoes and other insect vectors, and minimal risk of public exposure and contact with the water in the system.

The process can remove BOD_5 and suspended solids to very low concentrations and produce the equivalent of tertiary effluent. Interim design guidelines for BOD_5 removal are provided in the report. Nitrogen removal to very low levels is possible if sufficient detention time and oxygen to support the necessary nitrification reactions are present. Many of the early systems were deficient in both respects. Corrective action is possible, and the report presents tentative methods for appropriate design. A limited data base supports the capability of the SF wetland process for effective removal of metals and other priority pollutants. However, the process has limited capacity for removal of phosphorus as presently conceived, and supplemental treatment may be necessary. A one- or two-log reduction in fecal coliforms can be reliably achieved with this process, lower levels may require post disinfection.

In addition to design guidelines, the report provides an assessment of concept applicability and the research needs for a better understanding of the process.

PREFACE

This report was sponsored by the U.S. EPA Office of Wastewater Enforcement and Compliance under the direction of Mr. Robert K. Bastian, and Mr. Robert E. Lee, of the Municipal Technology Branch.

It is the intent of this report to present current (1993) information and guidance on design, construction, performance, operation and maintenance of subsurface flow constructed wetlands used for treatment ‘of domestic and municipal wastewaters. Subsurface flow constructed wetlands are in relatively common use in Europe, Australia, and in a number of states in the United States. However, there has been no apparent consensus on design procedures or performance expectations. A two-step procedure was used for the preparation of this report to develop a consensus among knowledgeable experts.

The first step was to submit a draft report prepared by Mr. Sherwood Reed, E.E.C., to selected experts for their detailed review. These individuals included: Mr. Donald Brown, U.S. EPA RREL; Dr. Dennis George, Tennessee Tech University; Mr. Michael Ogden, Southwest Wetlands Group: Dr. Richard Gersberg, San Diego State University; Mr. Ronald Crites, Nolte & Associates; Dr. Robert Knight, CH2MHill; Dr. George Tchobanoglous, University of California-Davis; Mr. Michael Hines, Tennessee Valley Authority; Dr. Frank Saunders, McCulley, Frick & Gilman, Inc.; and Dr. Peter Jenssen, Jordforsk, Norway.

The second step was accomplished at a two-day workshop held in New Orleans,.-LA on September 24 and 25, 1992. Participants in that meeting included invited experts, representatives from US EPA Region VI, state and local officials, and local design firms with experience with this technology. These participants are listed in Appendix A of this report. The workshop commenced with a detailed presentation of the contents of the draft report. This was followed by intensive discussion by participants on all of the major topics of concern. Additional time was taken at the meeting to identify research needs for further optimization of this technology. A revised version of the text was then circulated to all participants and previous reviewers for their comments.

The preparation of this final report considers all of the comments and suggestions received from all sources. As a result, this report represents a consensus among the experts participating in this effort on design, construction, and operation and maintenance of subsurface flow constructed wetlands. Current and future research will undoubtedly improve understanding of the basic concepts involved and lead to more sophisticated design models. The design procedures and related guidance in this report are therefore considered to be valid, but subject to further improvement.

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LIST OF ABBREVIATIONS

A_c	Cross-sectional area
ac	Acres
ac/mgd	Acres per million gallons per day
BOD_5	5-day biochemical oxygen demand
OC	Degrees Celsius
Ce	Concentration effluent
-cm	Centimeters
cm/d	Centimeters per day
co	Concentration influent
COD	Chemical oxygen demand
CSO	Combined sewer overflow
d	Days
dh/dL	Hydraulic gradient
EPA	Environmental Protection Agency
ET	Evapotranspiration
ft	Feet
FWS	Free water surface
gal	Gallons
gal/d	Gallons-per day

gm	Grams
gpd	Gallons per day
ha	Hectare
HRT	Hydraulic residence time
in/d	Inches per day
kg	Kilograms
kg/d	Kilograms per day
kg/ha/d	Kilograms per hectare per day
ks	Hydraulic conductivity
lb/ac/d	Pounds per acre per day
L	Total bed length
L:W	Length to width
m	Meters
m^2	Square meters
m^2	Square meters per population equivalent
m^3/d	Cubic meters per day
$M^3/d/PE$	Cubic meters per day per population equivalent
mgd	Million gallons per day
mg/L	Milligrams per liter
ml	Milliliters
mm	Millimeters

N	Nitrogen
n	Porosity
NH_3	Ammonia
NPDES	National Pollutant Discharge Elimination System
$0_2/m^2/d$	Oxygen- per square meter per day
O L	Organic loading
P.E.	Professional engineer
PE	Population equivalent
PVC	Polyvinyl chloride
Q	Flow
RREL	Risk Reduction Engineering Laboratory
SF	Subsurface flow
TKN	Total Kjeldahl nitrogen
TSS	Total suspended solids
TVA	Tennessee Valley Authority,
U.S.	United States of America
U.S. EPA	United States Environmental Protection Agency
W	Bed width
WPCF	Water Pollution Control Federation

CHAPTER 1

INTRODUCTION

This report describes the design, construction and performance of subsurface flow constructed wetlands as used in the United States for wastewater treatment. Utilization of this technology has grown very rapidly in the past five years (1988-93), and it is clear from the discussion in the next section of the report that there has been no general consensus on design of these systems or their performance expectations.

In recognition of that situation, various offices within U.S. EPA, as well as other agencies and groups, have sponsored several efforts to better understand wetland systems and their capabilities and limitations for wastewater treatment. These efforts have included: a detailed, multi-year monitoring program at several constructed wetlands in -Kentucky (sponsored by EPA headquarters, the National Small Flows Clearinghouse, EPA Region IV, and TVA); an inventory of all constructed wetlands used for wastewater treatment in the U.S. (sponsored by EPA RREL); site visits with evaluations and reports at selected operating subsurface flow wetlands in the U.S. (sponsored by EPA RREL); a brief performance evaluation at three subsurface flow systems in Louisiana (sponsored by EPA Headquarters); detailed performance monitoring and evaluation at two subsurface flow systems, also in Louisiana (sponsored by EPA RREL); creation of a detailed data base covering design, operating and performance data for natural and constructed wetland systems treating municipal and industrial wastewater and storm water runoff in the U.S. (sponsored by EPA Corvallis); and several workshops and seminars sponsored by EPA Region VI.

All of these sources were utilized in the preparation of this report and the discussion and evaluations which are’ included. The focus of this report is the subsurface flow type wetland. Information on other types of wetland systems has has been included for comparative purposes.

CHAPTER 2

BACKGROUND

Wetlands are defined as land where the water surface is near the ground surface lung enough each year to maintain saturated soil conditions, along with the related vegetation. Marshes, bogs, and swamps are all examples of naturally occurring wetlands. A "constructed wetland" is defined as a wetland specifically constructed for the purpose of pollution control and waste management, at a location other than existing natural wetlands. There are two basic types of constructed wetlands, the free water surface wetland and the subsurface flow wetland. Both types utilize emergent aquatic vegetation and are similar in appearance to a marsh.

The free water surface (FWS) wetland typically consists of a basin or channels with some type of barrier to prevent seepage, soil to support the roots of the emergent vegetation, and water at a relatively shallow depth flowing through the system. The water surface is exposed to the atmosphere, and the intended flow path through the system is horizontal.

The subsurface flow (SF) wetland also consists of a basin or channel with a barrier to prevent seepage, but the bed contains a suitable depth of porous media. Rock or gravel are the most commonly used media types in the U.S. The media also support the root structure of the emergent vegetation. The design of these systems assumes that the water level in the bed will remain below the top of the rock or gravel media. The flow path through the operational systems in the U.S. is horizontal.

The SF type of wetland is thought to have several advantages over the FWS type. If the water surface is maintained below the media surface there is little risk of odors, odors, exposure, or insect vectors. In addition, it is believed that the media provides greater available surface area for treatment than the FWS concept so the treatment responses may be faster for the SF type, which therefore can be smaller in area than a FWS system designed for the same wastewater conditions. The subsurface. position of the water and the accumulated plant debris on the surface of the SF bed offer greater thermal protection in cold climates than the FWS type.

Subsurface flow constructed wetlands first emerged as a wastewater treatment technology in Western Europe based on research by Seidel (1) commencing in the 1960s, and by Kickuth (2) in the late 1970s and early 1980s. Early developmental work in the United States commenced in the early 1980s with the research of Wolverton, et al. (3) and Gersberg et al. (4).

The SF concept developed by Seidel included a series of beds composed of sand or gravel supporting emergent aquatic vegetation such as cattails *(Typha),* bulrush *(Scirpus),* and reeds *(Phragmites),* with *Phragmites* being the most commonly used. In the majority of cases, the flow path was vertical through each cell to an underdrain and then onto the next cell. Excellent performance for removal of BOD_5 TSS, nitrogen, phosphorus, and more complex organics was claimed. Pilot studies of the concept in the United States were marginally successful, and it has not been utilized in recent years in this country.

Kickuth proposed the use of cohesive soils instead of sand or gravel; the vegetation of preference was *Phragmites* and the design flow path was horizontal through the soil media. Kickuth's theory suggested that the growth, development and death of the plant roots and rhizomes would open up flow channels, to a depth of about 0.6 m (2 ft) in the cohesive soil, so that the hydraulic conductivity of a clay-like soil would gradually be converted to the equivalent of a sandy soil. This would permit flow through the media at reasonable rates and would also take advantage of the adsorptive capacity of the soil for phosphorus and other materials. Very effective removal of BOD_5 TSS, nitrogen, phosphorus, and more complex organics was claimed. As a result, by 1990 about 500 of these "reed bed" or "root zone" systems had been constructed in Germany, Denmark, Austria, and Switzerland. The types of systems in operation include on-site single family units as well as larger systems treating municipal and industrial wastewaters. Many of the -early systems were designed (5) with a criterion of 2.2 m^2 of bed surface area per population equivalent (PE). A PE in European terms is equivalent to the organic loading from one person, or approximately 0.04 kg/d BOD_5 in typical primary effluent. That is equal to a surface organic loading of about 180 kg/ha/d (162 Ib/ac/d). The more recently constructed systems in Europe (15) have been designed for 5 to 10 m^2/PE (40 - 80 kg/ha/d). The hydraulic loading at 5 m^2/PE (at an assumed 0.2 m^3/d/PE) would be about 4 cm/d (1.6 in/d), which, in a commonly used term in the U.S., is equivalent to 23 acres/mgd of design flow, and would provide a hydraulic residence time (HRT) of about six days. For comparison, FWS wetlands in Europe are typically designed at 10, m^2/PE, which results in a surface area about twice that required for the SF type (39).

Commencing in 1985, a number of "reed bed" systems were constructed in Great Britain based on Kickuth's concepts, but in many cases gravel was used as the bed media rather than cohesive soil (5) due to concerns regarding soil hydraulic conductivity. Many of these beds were built with a sloping bottom (0.5 to 1%) and a flat surface. The purpose of the sloping bottom was to provide sufficient hydraulic gradient to ensure subsurface flow in the bed. The flat upper surface would allow temporary flooding as a weed control measure to kill undesirable plants. Some of these systems also had an adjustable outlet which permitted easy maintenance of the desired water level in the bed (19).

Wolverton's work in Louisiana began with experimental bench scale trays in a greenhouse containing rock or gravel media and supporting a stand of emergent aquatic vegetation (3). The trays were filled with wastewater, and then drained after a certain number of hours (range 12 to 48 hours). In essence the procedure was a fill and draw batch type process. Excellent performance was demonstrated for BOD_5 TSS, and NH,, and moderate performance for phosphorus with a one-day HRT (3) The typical organic loading during these experiments (at one-day HRT) was about 58 kg/ha/d (52 Ib/ac/d), and the hydraulic loading was about 8 cm/d (3.5 in/d). Design criteria based on this work (16) included one day HRT, about five acres of bed surface area per mgd, and up to 15:1 aspect ratio (L:W). These 'criteria, or variations, have been widely applied and, as of 1991, there were about 60 systems in operation or in various stages of design in the south central U.S., based on these values. These systems range from on-site single family units to large-scale municipal systems (up to 4 mgd) (16).

Gersberg's work (4) was conducted over a period of several years in Santee, CA, in large-scale, continuous flow, field experiments using 0.76 m (2.5 ft) deep gravel beds. The removal of BOD_5 TSS, and NH, was correlated with the depth of root penetration for the plant varieties *(Typha, Scirpus, Phragmites)* with the best removals occurring with the deepest root penetration *(Scirpus* then *Phragmites).* The organic loading was approximately 55 kg/ha/d (49 Ib/ac/d and the hydraulic loading about 5 cm/d (2 in/d). This hydraulic loading is equal to 18 ac/mgd using the common U.S. term. The HRT in this system was about six days, compared to only one day from Wolverton's (3) work and possibly up to six days in the new European systems (15).

Beginning in the mid 1980s, the Tennessee Valley Authority (TVA) began a program of research and technical assistance on constructed wetlands for treatment of a variety of waste streams (municipal wastewater, acid mine drainage, agricultural wastes and runoff, etc.) (9). Their criteria for subsurface flow wetlands designed for wastewater treatment, originally derived from the work of Kickuth (18), have been modified significantly in subsequent years. By 1991 there are probably at least 80 subsurface flow systems, in operation in a number of states, based on criteria and assistance provided by TVA (20). These systems range in size from on-site single family units to larger municipal systems (3785 m^3/d, 1 mgd).

The organic loading on one of the early TVA systems (Benton, KY) was 81 kg/ha/d (72 Ib/ac/d) and the design hydraulic loading 14 cm/d (5.6 in/d) (3.6 ac/mgd).

The theoretical HRT at this hydraulic loading would be about 2 d. At one of the more recently constructed systems (Bear Creek, AL) the organic loading is about 4 kg/ha/d (3.4 Ib/ac/d) with a hydraulic loading of about 3 cm/d (1.2 in/d)(31 ac/mgd). The theoretical HRT, at design flow, would be about three days. The current TVA recommendations (17) for small-scale systems using a septic tank and a bed depth of 0.5 m (1.5 ft) are 62 kg/ha/d (56 Ib/ac/d) organic loading and 5 cm/d (1.8 in/d) hydraulic loading (20 ac/mgd). The theoretical HRT at this hydraulic loading is about 4 d. Table 1 summarizes the various design approaches discussed above.

Table 1. Historical Design Approaches, European and U.S. Sources

Source	Organic Loading (kg/ha/d)[a]	Hydraulic Loading (cm/d)[b]	Area ($m^2/m^3/d$)[c]	Hrt (d)
Europe				
Boon (1985)	180	9	11	2.6
Cooper (1990)	80	4	24	6
U.S.				
Wolverton (1983)	58	8	13	1
Gersberg (1.985)	55	5	20	6
TVA				
Benton, KY (1989)	81	14	7	2
Bear Cr, AL (1991)	4	3	33	3

a. kg/ha/d x 0.892 = Ib/ac/d
b. cm/d x 0.394 = in/d
c. $m^2/m^3/d$ x 2.091 = ac/mgd

A comparison of the data presented in Table 1 indicates there has been no consensus regarding design criteria for SF wetlands. This obviously has a significant impact ***on*** system costs and may affect concept feasibility. The U.S. EPA, and others, undertook the investigative tasks described in the Introduction to this report, and the results of those studies are discussed in the next section.

CHAPTER 3

PERFORMANCE EVALUATIONS

This section compares performance expectations for the SF. concept to actual field results obtained by those studies sponsored by the U.S. EPA and others, which were described previously. The data from those ***sources*** are not specifically identified in the graphical presentations which follow but are listed in Table 2 below, and described in more detail in Appendix B. The fourteen systems listed in Table 2 are believed to be representative examples of the systems in operation in the United States. They were selected for this analysis since each had a significant body of relatively reliable input/output water quality data which characterizes the performance of the wetland component in the system. Many other operating systems only have limited effluent data collected to verify their compliance with NPDES requirements. All of the data shown in the subsequent graphical presentations are averages over the available period of record for each of these systems. None of these systems have been operating longer than five years.

Table 2. Data Sources for Performance Evaluation

Location	Wastewater Type	Design Flow (m^3/d)[e]	Treatment Area (ha)[f]
Greenleaves, LA	Municipal	564	0.44
Degussa Co., MS	Industrial	6737	0.89
Bear Creek, AL	Domestic	59	0.20
Monterey, VA	Municipal	83	0.02
Denham Springs, LA	Municipal	6548	6.15
Benton, LA	Municipal	378	0.61
Haughton, LA	Hospital	380	0.61
Carville, LA	Municipal	465	0.26
Mandeville, LA	Municipal	4633	1.85
Benton, KY	Municipal	685	1.46
Hardin, KY[a]	Municipal	236	0.32
Hardin, KY[b]	Municipal	186	0.32
Utica, MS”	Municipal	189	0.61
Utica, MS[d]	Municipal	416	0.81

a. *phragmites bed,* b. *Scirpus bed,* c. North system, d. South system
e. $m^3/d \times (2.64 \times 10^{-6}$ = mgd. f. Ha x 2.47 = acres.

BOD_5 REMOVAL

The physical removal of BOD_5 is believed to occur rapidly through settling and entrapment of particulate matter in the void spaces in the gravel or rock media. Soluble BOD_5 is removed by the microbial growth on the media surfaces and attached to the plant roots and rhizomes penetrating the bed. Some oxygen is believed to be available at microsites on the surfaces of the plant roots, but the remainder of the bed can be expected to be anaerobic.

Compared to other forms of wastewater treatment, both SF and FWS wetland systems, are unique in that BOD_5 is actually produced within the system due to the decomposition of plant litter and other naturally occurring organic materials. As a result, the systems can never achieve complete BOD_5 removal and a residual BOD_5 from 2 to 7 mg/L is typically present in the effluent.

Figure 1 presents BOD_5 input versus BOD_5 output data for the systems listed in Table 2. All effluent values are well below the 20 mg/L reference level which is a common permit requirement, and this can be achieved regardless of the input concen-

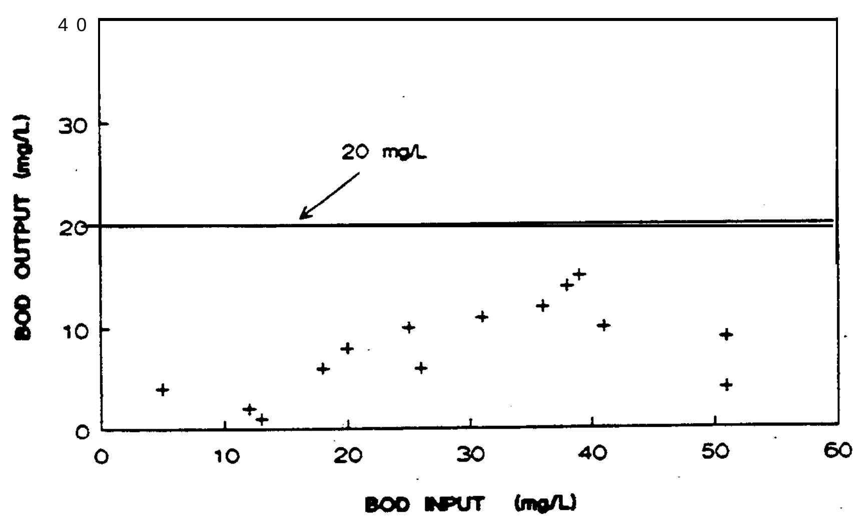

Figure 1. BOD_5 Input Versus Output.

tration (within the range shown). Data from similar systems in Europe show essentially the same relationship with input BOD_5 concentrations up to 150 mg/L (39). The low values, in the lower left portion of the graph, are influenced by the presence of the residual BOD_5 discussed above, which limits final effluent levels to a range of 2 to 7 mg/L.

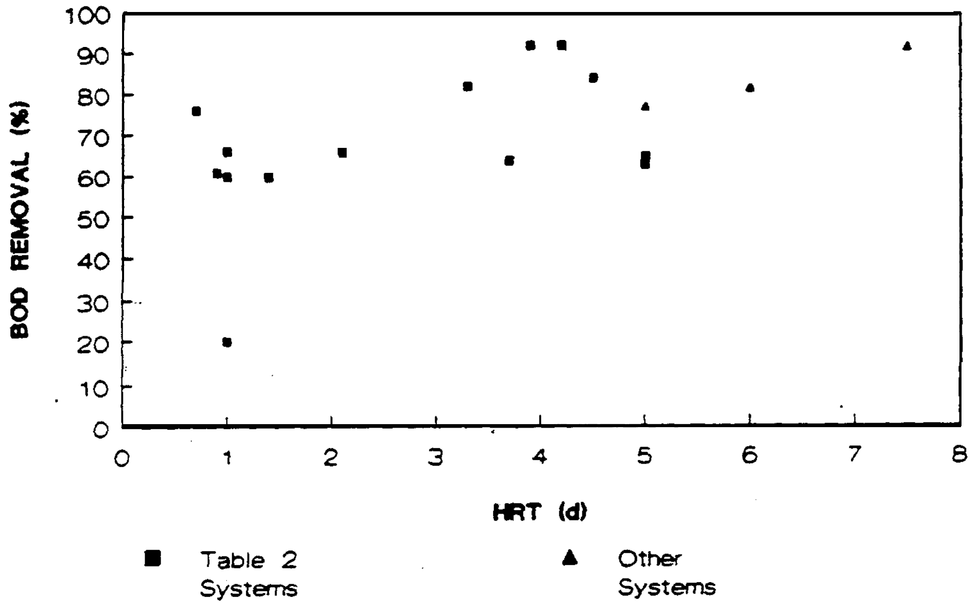

Figure 2. BOD_5 Removal Versus Hydraulic Residence Time

A comparison of dissolved and total BOD_5 and COD for influent and effluent at two sites in Louisiana suggest that most of the BOD_5 leaving those systems was residual organics from the system and not of wastewater origin (27).

Figure 2 presents BOD_5 removals versus the calculated HRT for these same systems. It seems clear that after one- to one- and one half-day HRT, the removal of BOD_5 is not strongly dependent on HRT since removal improves only slightly thereafter, up to an HRT of 7.5 d. The 60 to 65 percent removal values shown at the one-day HRT are not due to poor BOD_5 removal, but rather to relatively low input levels (see Appendix B for data). The lowest removal, at 20 percent, is due to an input BOD_5 (5 mg/L) which was already at the threshold for residual. The fourteen systems listed in Table 2 are designated by square symbols in the figure; three additional systems, from other sources, are also included to extend the range of HRT shown.

The BOD_5 removal values for these data can be reasonably approximated by a first order plug flow relationship up to about ± 2 d. The BOD_5 removal thereafter is limited and is believed to be influenced by the production of residual BOD_5 in the system, as mentioned previously. This is compatible with the hypothesis that BOD_5 is removed rapidly in the front part of these systems, and suggests that the removals, indicated on the figure, at the longer detention times may have actually been obtained at an earlier point in the bed prior to sampling and testing of the final effluent.

It has been suggested in the past that these SF wetland systems must have a high aspect ratio (L:W) to ensure maintenance of plug flow conditions and high levels of performance (16). A common recommendation indicated that the L:W should be at least 10:l and that lesser levels might impair removal efficiency. Figure 3 tests that hypothesis by comparing the average BOD_5 removal achieved to the aspect ratio of the various systems listed in Table 2.

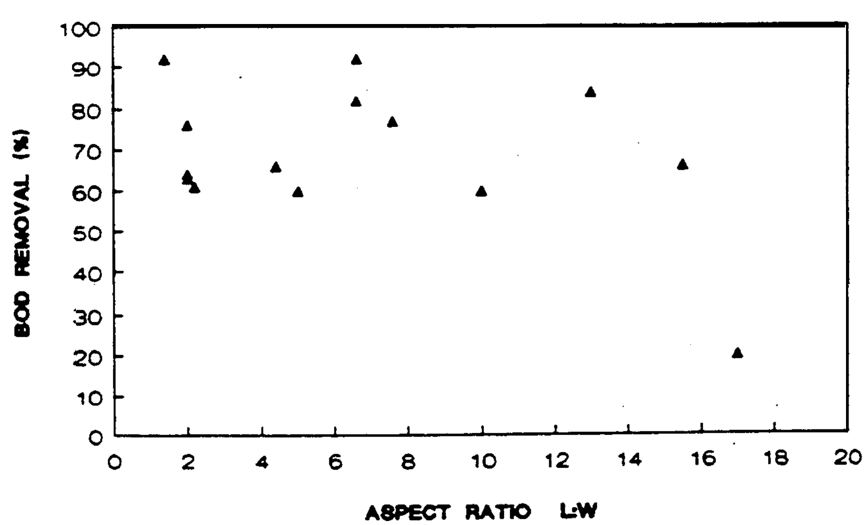

Figure 3. System Aspect Ratio Versus BOD_5 Removal.

In these cases the aspect ratio varied from less than 2:1 to over 17:1. As **illustrated by the figure, there does not appear to be any relationship between aspect ratio and BOD_5 removal capabilities.** The very low removal associated with the highest L:W (17:1) in the plot is due to the very low input BOD_5 (< 5 mg/L) and is not related to the aspect ratio. In such cases, further significant BOD_5 removal cannot be expected regardless of aspect ratio or HRT. In this case, any reduction of the wastewater from an input value of 5 mg/L was probably replaced, in part, by the residual BOD_5 from plant detritus.

Other natural treatment systems, such as facultative lagoons and land treatment systems, have displayed a near linear relationship between mass organic loading and mass removal rates, up to relatively high loading rates.

Figure 4 illustrates this relationship for SF wetlands and confirms that a linear relationship does exist between these two parameters. The r^2 value for the curve fit is 0.97, indicating an excellent correlation.

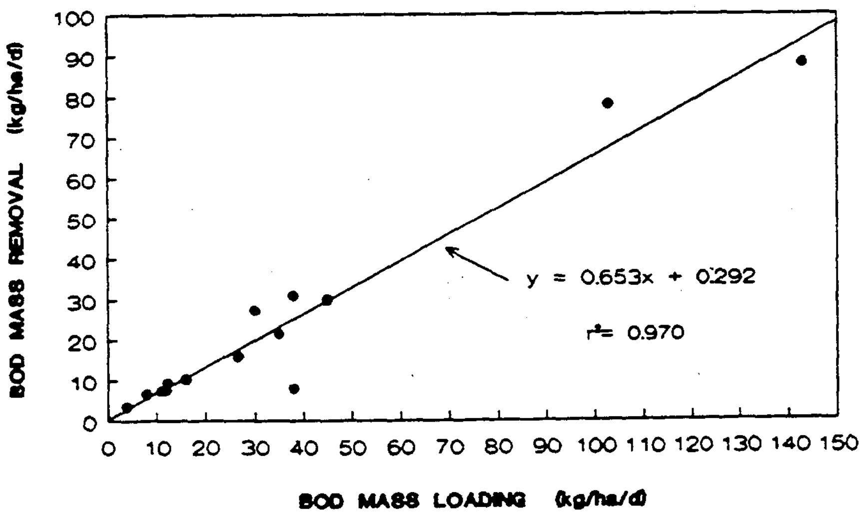

Figure 4. BOD_5 Loading VS BOD_5 Removal, Mass Basis.

Caution is necessary when discussing mass organic loadings on these wetland systems. The values shown on Figure 4, and similar values which can be found in other references, are not the actual areal organic loading on the system, but rather are the "apparent" organic loadings obtained by dividing the daily organic load by the total surface, area of the system. That approach implies that the organic load is applied uniformly over the entire surface area of the system. As discussed previously, much of the input solids and BOD_5 are probably removed rapidly near the front end of the system, so the actual organic loading 'on this zone is much higher than on the rest of the system. This situation also prevails in FWS wetlands, most facultative lagoons, and most overland flow land treatment systems where the influent is typically applied. at the head of the treatment unit. In slow rate land treatment systems where sprinklers are used to distribute the wastewater over the entire treatment surface the "apparent" organic loading may be equal to the actual organic loading. A few FWS wetland systems designed for further polishing of highly treated tertiary effluent are dealing with low levels of essentially soluble BOD_5 and in these cases the "apparent" organic loading may approximate the actual areal organic loading.. In overland flow systems dealing with high concentrations of organic solids, sprinklers are also used to ensure a more uniform distribution over the available treatment area. In wetland systems, this purpose could be achieved with step feed of the influent at more than one point along the flow path to ensure a more uniform loading.

This non-uniform application of organic wastes complicates development of an accurate and precise design model for BOD_5 removal, since it is likely that the actual removal rates may vary along the flow path, and, concurrently, residual BOD_5 is being produced from decomposing plant detritus. The situation is further complicated in that the only data currently available are input/output data from a limited number of systems, some of which have a longer HRT than is probably necessary to achieve the measured effluent.

The development of the ultimate design model must wait for collection of a sufficient body of reliable data describing the internal performance within these systems. In the interim, some rational design approach is necessary to ensure that these systems are cost-effective and can reliably produce the expected effluent quality. The same situation prevails in the design of overland flow land treatment, facultative lagoons, and similar aquatic concepts. In these cases, it is the consensus opinion that a first order plug flow model provides an acceptable and rational basis for design. Advances have been made with lagoon systems, and more complex models accounting for dispersion and mixing have been developed. However, the first order plug flow model still prevails as the most commonly used.

A first order plug flow model for BOD_5 removal also has been used by a number of engineers for design of these SF wetland systems - in the United States, Europe, and Australia (6,7,8,9,10,12,13,14,15,19,20). The general form of the model is presented below:

$$\frac{C_e}{C_o} = e^{(-K_T t)} \qquad \textbf{(1)}$$

Where:

C_e = Effluent BOD_5 (mg/L)
C_o = Influent BOD_5 (mg/L)
K_T = temperature dependent rate constant (d^{-1})
t = hydraulic residence time (d)
K_T = $K_{20}(1.06)^{(T-20)}$
K_{20} = rate constant at 20o C, (d^{-1})
= 1.104 d^{-1}
T = temperature of liquid in the system (°C)

A rate constant K_{20} equal to 1.104 d^{-1} for use in equation (1) for the design of SF constructed wetlands has been proposed and published in several sources (6,7,8). Independent support for that value can also be found in other published literature (19). This value is believed to be conservative and is associated with an "apparent" organic loading on the system of about 110 kg/ha/d (= 98 Ib/ac/d). The highest "apparent" organic loading shown on Figure 4 is about 143 kg/ha/d (= 128 Ib/ac/d) and the associated "apparent" rate constant would be about 1.385 d^{-1} In theory, it should be possible to use this higher value for design, but the more conservative 1.104 d^{-1} is recommended since there is some independent support for that value. Collection of additional performance data in the future may permit further optimization and possibly a significant increase in the 1.104 d^{-1} rate constant.

It is believed that the plug flow rate constant for these SF wetlands is higher than those for facultative lagoons or for FWS wetlands because the surface area available on the media in the SF wetlands is much higher than in the other two cases. This surface area supports the development, and retention, of attached -growth microorganisms which are believed to provide most of the treatment responses in the system. Table 3 compares the rate constants for these three treatment concepts, at an "apparent" organic loading of 110 kg/ha/d (98 Ib/ac/d). The rate constant for the SF wetland is about an order of magnitude higher than facultative lagoons, and about double the value for FWS wetlands.

Table 3. Comparison of First Order Plug Flow Rate Constants (7,11,40)

Treatment Process	Rate Constant[a] $(d)^{-1}$
Subsurface Flow Wetland	1.104
Facultative Lagoon	0.117
Free Water Surface Wetland	0.501

a. At an apparent organic loading rate of about 110 kg/ha/d

TSS REMOVAL

Suspended solids removal is very effective in SF constructed wetlands. Most of the removal probably occurs within the first few meters of travel distance from the inlet zone.

Figure 5 presents the input TSS versus output TSS for the sites listed in Table 2, with 20 mg/L shown as a reference index. Except for one excursion (believed due to extensive surface flow and short circuiting), all of the systems produce a final effluent with less than 20 mg/L TSS regardless of the input level (up to 118 mg/L). The systems with high TSS input values (< 50 mg/l) typically have facultative lagoons and the high solids are due to algae carry-over from the lagoon.

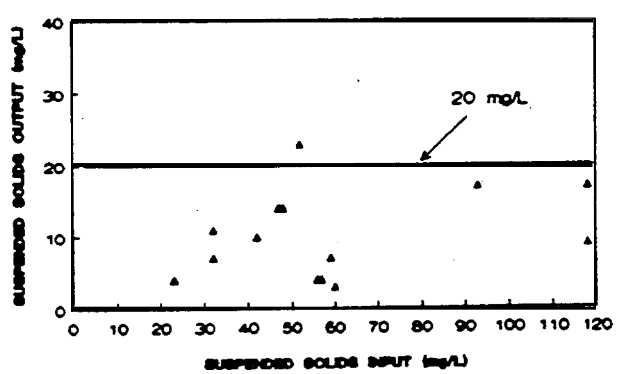

Figure 5. Suspended Solids, Input Versus Output.

Figure 6 compares the TSS removal rate to the HRT for the systems examined. The relationship is similar to that shown on Figure 2 for BOD_5 in that after an HRT of one day there is little improvement in removal of suspended solids.

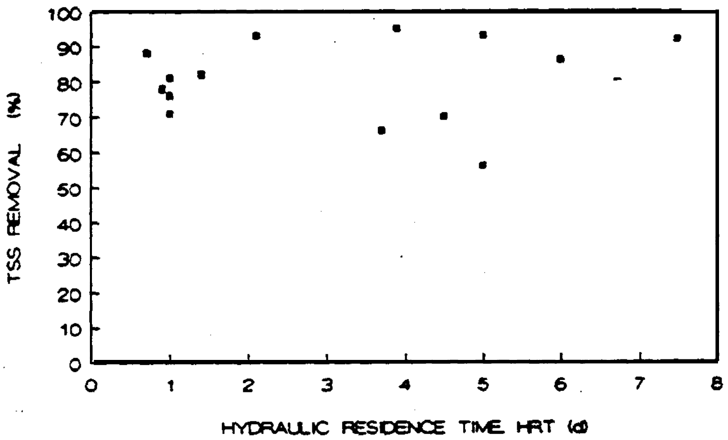

Figure 6. Suspended Solids Removal Versus HRT.

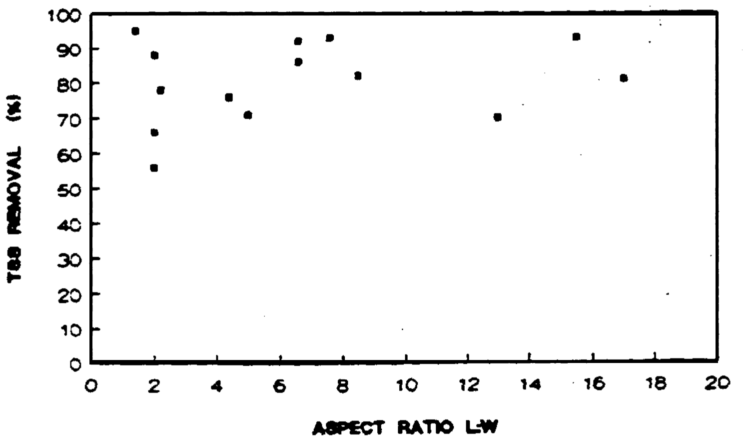

Figure 7. Suspended Solids Removal Versus System Aspect Ratio.

Figure 7 compares the TSS removal rate to the aspect ratio of the various systems. **These results are similar to those shown in Figure 3 for BOD_5 and indicate no relationship between the system aspect ratio and TSS removal.**

A kinetic design model is not available for TSS removal. However, based on the relationships illustrated in Figures 1 to 7, it is apparent that TSS removal follows the same pattern as BOD_5. This suggests that when a system is designed for a particular level of BOD_5 removal, the TSS removal will be comparable as long as subsurface flow is maintained in the bed.

NITROGEN REMOVAL

The removal of non-ionized ammonia is typically the major nitrogen parameter of concern due to its toxicity for fish and other aquatic animals, and to its added oxygen demand on receiving streams. Many of the earliest SF wetland systems were only required to remove BOD_5 and TSS and have done so successfully. In some cases, their permits have since been revised to require ammonia removal. Many of the new systems also have ammonia limits (depending on receiving water. requirements).

The nitrogen entering wetland systems can be measured as organic nitrogen and ammonia (expressed as TKN), or as nitrate, or a combination of both nitrogen measurements. Septic tanks, primary treatment systems, and facultative lagoon effluents do not usually contain nitrate, but can have significant levels of organic N and ammonia. During the warm summer months, facultative lagoons can have low levels of ammonia in the effluent, but often contain high concentrations of organic N associated with the algae leaving with the effluent. Aerated secondary treatment system effluents typically have low levels of organic N but contain significant concentrations of ammonia and nitrate. Systems with high intensity or long-term. aeration can have most of the nitrogen in the nitrate form.

The organic N entering a SF wetland is typically associated with particulate matter such as organic wastewater solids and/or algae. The plant detritus and other naturally occurring organic materials in the wetland can also be a source for organic N. **Decomposition and mineralization processes in the wetland will convert a significant part of this organic N to ammonia.**

Biological nitrification followed by denitrification is believed to be the major pathway for ammonia removal in both types of constructed wetlands, as they are presently operated (4,7,10). Plant tissue analysis at several locations indicate that a single annual harvest of the plant material might account for ten percent or less of the nitrogen- removed by the system (7,22,23). A more frequent harvesting program might increase this potential, but it would also increase the costs for operation of the system..

Figure 8 presents ammonia input versus output data for the systems included in this study which have a continuous discharge and without recycle. The line on the graph indicates the condition when input equals output. One half of the systems are at or above that line, indicating that there is a net production of ammonia as the wastewater passes through the bed. The source of this “extra” ammonia is believed to be from the anaerobic decomposition of the organic nitrogen trapped in the bed as particulate matter. Since the bed is anaerobic, there is then insufficient oxygen available in the bed to oxidize this ammonia to nitrate. Figure 9 presents ammonia removal for some of the systems versus HRT. The results for several systems (Denham Springs, LA for example, NH, in = 0.7 mg/L, NH, out = 10 mg/L, removal = - 1370%) are below the lower limit on the graph and are therefore not shown. One point (the open square in the upper right corner) represents data from Santee, CA; the remainder are data from the Table 2 systems.

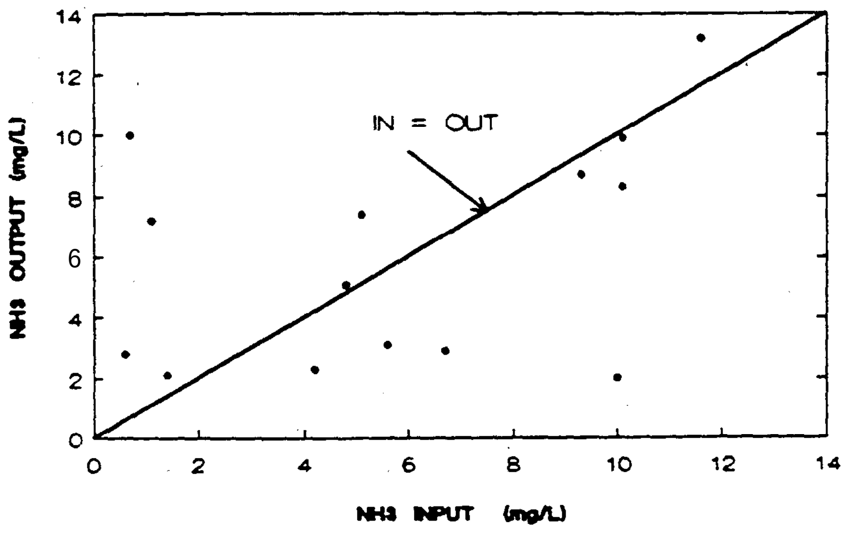

Figure 8. Ammonia Input Versus Output.

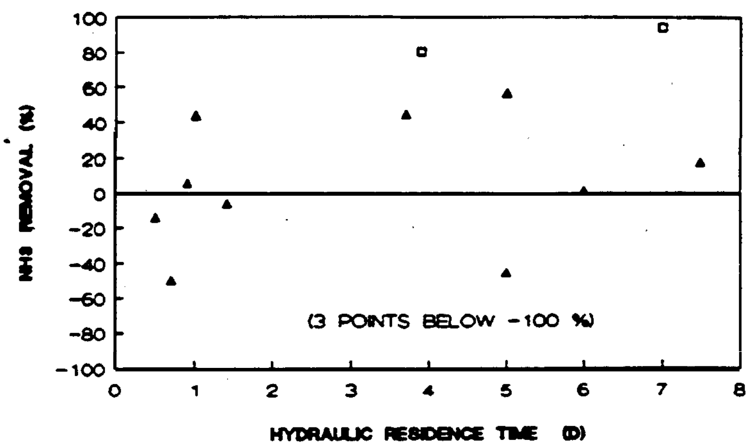

Figure 9. Ammonia Removal Versus HRT

Most of the systems shown on Figure 9 display a marginal or a negative ammonia removal rate regardless of the detention time in the system. Two systems, shown as squares in the figure, display very high removal rates at HRT levels comparable to the other systems. **The difference is believed to be the lack of algae, the availability of oxygen, and sufficient HRT in the latter two systems, so that high levels of nitrification can occur in these beds.**

The primary oxygen source in these two cases is believed to be from the 'roots of the emergent aquatic vegetation. In the first case, (Bear Creek, AL) the bed is only 0.3 m (1 ft) deep and supports a stand of *Typha* with the roots observed penetrating to the bottom of the bed; ammonia removal reaches 80 percent with an HRT of 3.9 d. The second case is the *Scirpus* bed at the Santee, CA pilot system, where the roots also penetrated to the bottom of the 0.76 m (2.5 ft) deep bed; ammonia removals of 94 percent were achieved with an HRT of seven. days (4).

It is also informative to compare the Bear Creek results using *Typha* to the 0.76 m-deep bed at Santee, CA which also supported a stand of *Typha.* **In the latter case,** the plant roots only penetrated to 0.3 m (same depth as at Bear Creek) and the ammonia removal was only 29 percent with the 7 d HRT. In the *Typha* bed at Santee, about 60 percent of the flow passed through the bed below the root zone where oxygen was not likely to be present and limited ammonia removal was achieved. At Bear Creek, the entire flow passed through the *Typha* root zone and 80 percent ammonia, removal was produced in 3.9 d. This strongly supports the hypothesis that the plant roots in these SF wetland beds are the primary source of oxygen needed for nitrification of ammonia and other biochemical responses, and that it is therefore essential to bring the wastewater into direct contact with the root zone.

Table 4 summarizes the site conditions and the ammonia removal performance of the 14 systems used in this analysis. It is clear from an examination of these data that systems with no algae, with longer detention times, and with deep root penetration produce the best ammonia removal results. The two systems in the list (Santee and Bear Creek) which incorporate all three factors produced the best results of all.

Table 4. Ammonia Removal in SF Wetlands.

Location	Ammonia Removal %	Algae Present	HRT d	Bed Depth m	Root[a] Depth %
Denham Springs, LA	-1328	Yes	1	0.61	50
Haughton, LA	-554	Yes	4.5	0.76	50
Carville, LA	-22	Yes	1.4	0.76	50
Benton, KY	-45	Yes	5	0.61	40
Mandeville, LA	-50	No	0.7	0.61	50
Greenleaves, LA	-14	No	1	0.61	50
Hardin, KY (bulrush)	18	No	3.3	0.61	50
Hardin, KY (reeds)	2	No	3.3	0.61	40
Utica, MS (north)	57	Yes	5	0.64	60
Utica, MS (south)	45	Yes	3.7	0.64	60
Degussa Corp., AL	45	No	1	0.61	50
Monterey, VA	6	No	0.9	0.91	30
Bear Creek, AL	80	No	3.9	0.30	100
Santee, CA *(Scitpus)*	94	No	7	0.61	100

a. Root depth expressed as a percentage of total bed depth.

The early investigators, and designers in the U.S. and Europe, believed that oxygen from the plant roots would be available throughout the SF wetland bed. Implicit in that belief is the assumption that the plant roots would penetrate the full depth of the bed. A procedure advocated by Kickuth (5) to achieve that result was to lower the water level in the bed during the fall of the year, for three successive years, to induce maximum root penetration. That procedure has not been attempted in the U.S. to date.

The media depth in most of the beds in the U.S. is about 0.6 m (2 f-t), but in most cases, the plant roots have been observed to penetrate only to 0.3 m (1 ft) or less. Deeper penetrations have been observed when nutrient levels in the water are low or when the plants are located at the sides of the cells and other possible “dead spots” with less flow than the main portion of the bed. As a result, about half of the flow in most U.S. systems occurs in a zone where oxygen is not likely to be present and nitrification cannot occur. The results shown for most of the data points in Figure 9 are typical expectations for the current mode of SF wetland system operation and bed configuration. To ensure significant nitrification in these systems, it will be necessary to develop an appropriate oxygen source. Further discussion on that topic is provided in a later section of this report.

Figure 10 compares the mass removal rate for total Kjeldahl nitrogen (TKN) to the mass loading (most of the systems in Table 2 have no TKN data). TKN is essentially equal to total nitrogen (TN) in these cases since there is typically little nitrate entering or leaving the SF systems. The line shown on the figure is the result of a regression analysis for FWS wetland systems for these same parameters (24).

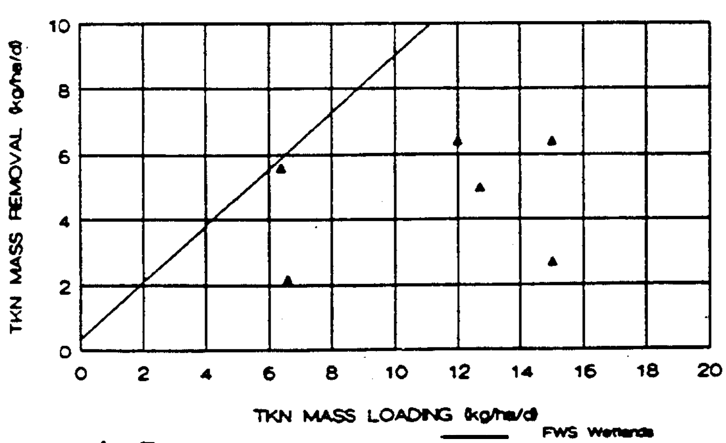

Figure 10. TKN Mass Loading Versus Mass Removal

There is no apparent relationship between mass loading of TKN and removal for the six SF systems where data were available. All but one of these data points are significantly below the FWS line, indicating that these SF systems are less efficient at TKN (or TN) removal than FWS systems. The difference is again believed to be the availability of oxygen. In FWS systems, the water surface is exposed to the atmosphere and some direct oxygen transfer is possible. The only data point on Figure 10 which is close to the line is the Bear Creek, AL system, where (as discussed previously) oxygen from the plant roots is apparently available in this shallow bed (0.3m) to support nitrification.

It has been suggested that the effluent ammonia from these wetland systems is related to the mass loading of TKN or TN on the system (41). Figure 11 presents this comparison for six of the Table 2 SF systems where sufficient data were available. The regression curve of best fit is also shown, but the low r^2 value indicates that the correlation is not very significant. A missing element in this approach is the residue from decomposition of plant matter in the system since the TKN loading is only a measure of nitrogen in the entering wastewater. Figure 11 suggests that a TKN mass loading of less than 2 kg/ha/d (1.8 lb/ac/d) would be necessary to consistently achieve an effluent ammonia of 2 to 3 mg/L in SF wetlands as they are currently operated with an oxygen deficiency.

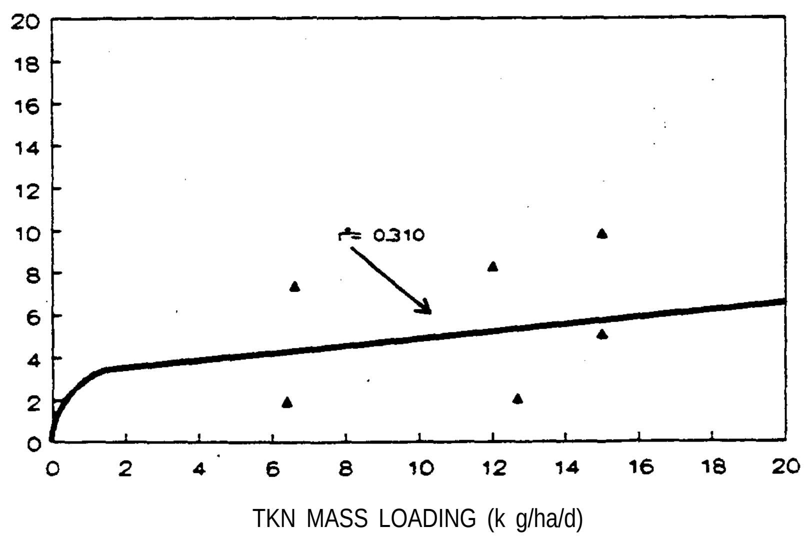

Figure 11. Effluent Ammonia Versus TKN Mass Loading

Many of the older systems in the Gulf States used soft tissue flowering plants for aesthetic reasons. Their decomposition is very rapid in the fall and after a frost, and measurable increases in effluent BOD_5 and ammonia can be observed (25). The same effect is likely when significant surface flow is allowed on these systems; surface flow will result in a much more rapid breakdown of plant detritus and release of ammonia to the water. If the surface of the bed is maintained in a dry condition (except during rainfall events), plant detritus will decompose much more slowly.

PHOSPHORUS REMOVAL

Figure 12 presents phosphorus input versus output data for the systems where data were available. The sloping line on the figure represents the condition where input equals output. Two of the systems fall on that line and several others show marginal removal capabilities. Only one data point shows excellent removal -- the Bear Creek, AL system. The media in this case are fine river gravel, and the presence of oxides of iron and aluminum associated with this media may be responsible for the 95 percent removal of phosphorus observed during the first year of system operation.

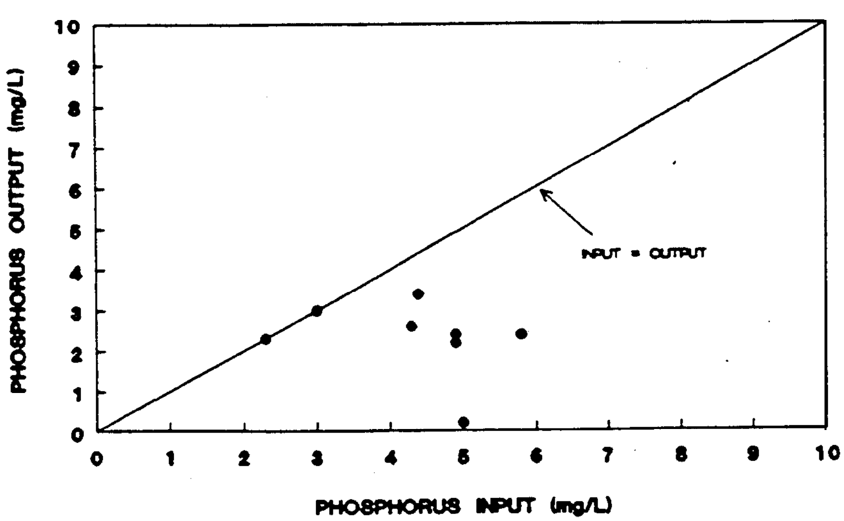

Figure 12. Phosphorus Input Versus Output

Phosphorus removal in most constructed wetland systems is not very effective because of the limited contact opportunities between the wastewater and the soil. Some experimental and developmental work has been undertaken using expanded clay aggregates and the addition of iron and aluminum oxides; some of these treatments may have promise but the long-term expectations have not been defined. Some systems in Europe use sand instead of gravel to increase the phosphorus retention capacity, but selecting this media results in a larger system because of the reduced hydraulic conductivity of sand compared to gravel. **If significant phosphorus removal is a project requirement, then very large land areas or alternative treatment methods will probably be required.**

FECAL COLIFORM REMOVAL

These SF wetland systems are, in the general case, capable of a one- to two-log reduction in fecal coliforms, which in many cases is not enough to routinely satisfy discharge requirements which often specify < 200/100 ml. Peak flows in response to intense rainfall events also disrupt removal efficiencies for fecal coliforms. As a result, most of the systems listed in Table 2 utilize some form of final disinfection. One exception is the lightly loaded system in Bear Creek, AL, which uses a small sized-river gravel as the bed media. At Bear Creek the fecal coliforms are reduced from 8×10^4/100 ml to 10/l00 ml or less on an average basis.

CHAPTER 4

DESIGN CONSIDERATIONS

The major concerns in the design of SF constructed wetlands include:

- Hydraulic and hydrological conditions,
- BOD_5 and TSS removal mechanisms,
- Nitrogen removal efficiency,
- Vegetation selection and management,
- Construction details, and
- ♍□⬧♦⬧☜

HYDRAULICS & HYDROLOGY

A basic intent of the SF wetland treatment concept is the maintenance of flow-beneath the surface of the media in the bed. However, a significant number of operating SF constructed wetlands are exhibiting varying degrees of surface flow on top of the media bed. Since these systems were designed for complete subsurface flow, this condition represents a potential design deficiency. It has been suggested that this surface flow is due to clogging of the void spaces in the bed either by the vegetative roots and associated plant materials or by the accumulation of suspended solids separated from the wastewater stream. However, most of these systems have been in operation for less than five years and surface flow was observed at many soon after flow commenced.

BED CLOGGING

A preliminary, unpublished 1990 investigation by Reed suggested a relationship between the organic loading on the cross section of the bed at the entry zone and observed surface flow on the SF wetland bed. The assumption was that clogging at the entry zone was causing the surface flow. A cross sectional loading of < 0.5 kg

$BOD_5/m^2/d$ (0.1 lb $BOD_5/ft^2/d$) was associated with no observed surface flow. Systems with observed surface flow had higher organic loadings, > 0.5 $kg/m^2/d$. Unfortunately, this apparent relationship has now been published in a number of sources and has become part of some design guidelines (17). Subsequent investigations at the original sites have indicated that the observed surface flow at most of the systems can be explained by inadequate hydraulic design and inattention to the requirements of Darcy's Law (described below). The organic loading approach may have some merit in that a reduced organic loading on the cross section should certainly reduce the potential for clogging. There are, however, no data available at present to support the selection of a specific cross sectional organic loading. The net affect of this approach is to increase the cross sectional area of the bed and thereby reduce the aspect (L:W) ratio. That same result can be achieved, in a more rational manner, by proper application of Darcy's Law.

Pits have been excavated in six of these SF systems (two in Kentucky, four in Louisiana) to observe any clogging substances, and to determine their characteristics (26,27,28). At only one Kentucky site, by June 1990, a persistent gelatinous substance had almost completely clogged the void spaces in the first 25 percent of the bed. Laboratory tests indicated that this material was about 80 percent inorganic, with that fraction composed of silica, clay minerals', and limestone dust. The clogging at this site occurred rapidly during the first year of operation and did not significantly expand in area in subsequent years. Pits excavated at the same locations in 1992 showed no evidence of gelatinous substances anywhere in the bed. It is likely that this isolated case of severe clogging may have been due to the overloaded condition of the bed during its first year of operation.

At the four sites in Louisiana, and the second site in Kentucky, the suspended matter in the rock voids was not gelatinous in character and washed easily from the rock surfaces (27,281. It was similar in appearance to a mixed liquor sample, with a slight odor in some cases. At three of the four sites in Louisiana, the solids present occupied less than two percent of the void space available for flow of water; in the worst case the solids present occupied about six percent of the available void spaces at a location close to the inlet pipe. In all cases, these solids were at least 80 percent inorganic material. Plant roots and related detritus were not encountered below depths of about 0.3 m (1 ft) in any of these systems (27,281. As a result, at these five sites it does not appear that accumulation of TSS or plant detritus were responsible for clogging or for any surface flow which may have occurred.

At all six of the sites investigated, the rock media were delivered by truck over unpaved roads, in all kinds of weather, and, as described by the operators, the trucks tended to follow the same pathway entering and leaving the wetland bed. It is quite possible that a large portion of the inorganic solids observed in the void spaces of the media was due to soil from the truck tires, and soil, rock dust and fines from the rock media transported in the truck. At the formerly partially clogged Kentucky site, these inorganic materials may have then trapped algal solids entering the bed, resulting in formation of the gelatinous material creating the clogging. The fact that the clogged zone never expanded beyond the first 25 percent of the bed, and that the gelatinous material has now disappeared, suggests that construction activity may have been the primary cause of the clogging instead of a continuing biochemical reaction.

HYDRAULIC DESIGN

When subsurface flow conditions are expected in the SF wetland bed it is common practice to use Darcy's Law, which describes the flow regime in a porous media. Darcy's Law is typically defined with equation (2).

$$Q = k_s A\, S \tag{2}$$

Where:

- Q = flow per unit time, m^3/d (ft^3/d), or (gal/d), etc.
- k_s = hydraulic conductivity of a unit area of the medium perpendicular to the flow direction, $m^3/m^2/d$ ($ft^3/ft^2/d$), or (gal/d), etc.
- A = total cross-sectional area, perpendicular to flow, m^2 (ft^2).
- S = hydraulic gradient of the water surface in the flow system dh/dL, m/m, (ft/ft).

(All units must be consistent)

Darcy's Law is not strictly applicable to subsurface flow wetlands because of physical limitations in the actual system. It assumes laminar flow conditions, which may not be the case when large rock or very coarse gravel are used as the media. Turbulent flow will occur in these coarse media when the hydraulic design is based on a high hydraulic gradient. Darcy's Law also assumes that the flow (Q) in the system is constant and uniform, but in the actual case in a SF wetland the input versus output Q may vary due to precipitation, evaporation, and seepage; and short circuiting of flow may occur due to unequal porosity or poor construction. **Ail of these factors limit the theoretical applicability of Darcy's Law, but it remains as the only reasonably accessible model for design of these SF systems.** If small to moderate sized gravel (< 4 cm) is used as the media, if the system is properly constructed to minimize short circuiting, if the system is designed to depend on a minimal hydraulic gradient, and **if the Q in equation (2) is considered to be the "average" flow ([Q_{in} + Q_{out}]/2) in the system to account for any gains or losses due to precipitation, evaporation or seepage,** then Darcy's Law can provide a reasonable approximation of the hydraulic conditions in these SF beds.

Some of the constraints on Darcy's-Law can be reduced by conducting predesign tests with the actual media to be used to determine the "effective" hydraulic conductivity under various flow and hydraulic gradient conditions, and to ensure that laminar flow conditions prevail. These tests are recommended for large-scale projects and/or for repetitive use of the same media on a number of small-scale projects. The test can use a flume or trough of reasonable length (< 6 m) and reasonable cross sectional area (depends on size of media to be tested, but generally < 0.2 m^2). The inlet end of the trough is capable of being raised above the datum to produce the desired test slope. The gravel is contained within perforated plates in the trough, and manometers are installed at appropriate locations to measure the head differential (dh) for calculation of the hydraulic gradient (dh/dL). Clean water is used in the test, and the inflow adjusted so that the gravel is saturated at the head of the flume but with no surface flow. During' the test the outflow (Q) is measured with a stop watch and a conveniently sized container, and the depth of the wetted zone (A) at the perforated outflow plate is measured. It is possible with these data to then calculate the "effective" hydraulic conductivity for design of the system, and to calculate the Reynolds number to ensure laminar flow conditions.

It is believed that the surface flow observed on many of the operational SF systems in the U.S. is the result of an inadequate hydraulic gradient provided by the system's design and selected configuration (13). Many of the problem systems in the U.S. have been constructed with a very high aspect ratio (L:W), and without any bottom slope or water level controls at the outlet works. In some cases, the outlet ports in the effluent manifold were at or near the top of the bed, thereby negating the development of any significant hydraulic gradient in the bed and ensuring the occurrence of surface flow from the beginning of operations. Systems in the U.S. and. Europe with successful hydraulic performance (i.e.,: maintenance of subsurface flow) do so with either **a** sloping bottom and/or adjustable outlet works which allow the water level to be lowered at the end of the bed. A sloped bottom or lowering the water level at the end of the bed then produces the pressure head required to overcome resistance to flow through the media and the maintenance of subsurface flow conditions. **An adjustable outlet provides greater flexibility and control and is the recommended approach.**

Aspect Ratio

The aspect ratio (L:W) of the wetland bed is a very important consideration in the hydraulic design of SF wetland systems, since the maximum potential hydraulic gradient is related to the available depth of the bed divided by the length of the flow path. Many of the early systems designed with an aspect ratio of 10:1 or more and a total depth of 0.6 m (2 ft) have an inadequate hydraulic gradient and surface flow is inevitable. The hydraulic gradient (S factor in equation 2) defines the total head available-in the system to overcome the resistance to horizontal flow in the porous. media.

For example, in a SF bed 200 m long and 0.6 m deep, if the water level is at the surface of the media at the influent end and near the bottom of the bed at the effluent end (water depth = 0.2 m), the available hydraulic gradient would be 0.4m/200m or 0.002. If this wetland bed were 100 m wide (L:W = 2:1) and used **a** gravel media with a hydraulic conductivity (k_s) of 10,000 $m^3/m^2/d$, the maximum subsurface flow, based on equation (2), would be 800 m^3/d (0.21 mgd). Using the same volume of gravel in a bed 450 m long results in a bed width of about 45 m (L:W = 10:1) and a hydraulic gradient of 0.0009; the maximum subsurface flow in this case would be 162 m^3/d (0.04 mgd), which is 20 percent of the flow allowed by the shorter bed. If the design flow were actually 800 m^3/d in the second case, then surface flow on top of the bed would be unavoidable, even though the bed contains exactly the same volume of media.

Bed Slope

SF systems in Europe (29) have been constructed with up to 8 percent slope on the bottom of the bed to maintain an acceptable hydraulic gradient. However, it is not practical and probably not possible with SF systems to precisely design and construct the bed for a specific hydraulic gradient due to variabilities in the media used and in construction techniques, and the potential for longer term partial clogging. In addition, the construction of a bed with a sloping bottom provides no flexibility for 'future adjustments. Greater flexibility and control is possible with an adjustable outlet' which permits control of the water level over the entire design depth of the bed. In this case, the bottom of the bed could be flat or with a very slight slope to ensure drainage, when required. However, because of the hydraulic gradient requirements, the aspect ratio (L:W) will have to be relatively low (in the range of 0.4:1 to 3:I) to provide the flexibility and the reserve capacity for future operational adjustments.

Media Types

Table 4 presents a summary of typical characteristics for the types of media which have been used in SF constructed wetlands. Essentially all of the operational SF constructed wetlands in the U.S. have used media ranging from medium gravel to coarse rock. The values in Table 4 are intended as preliminary information only. **Following selection of a media type and size; the hydraulic conductivity and porosity of the material should be determined in the field or laboratory, prior to system design,**

The recent. trend in the Gulf States toward the use of larger sizes of rock is believed due to the impression created by the surface flow conditions on many of the early systems. It was apparently thought that the surface flow was caused by clogging and that the use of a coarser rock with larger void spaces and a higher hydraulic conductivity would overcome the problem. In most cases the problem has not been overcome since the hydraulic gradient provided is too small. The use of smaller rock sizes has a number of advantages in that there is more surface area available on the media for treatment, and the smaller void spaces are more compatible with development of the roots and rhizomes of the vegetation, and the flow conditions should be closer to laminar. When turbulent flow occurs in the coarser media listed in Table 5, the "effective" hydraulic conductivity will be less than the values listed in the table.

Table 5. Typical Media Characteristics for SF Wetlands

Type	Effective Size D_{10} mm	n [a] Porosity %	k_s [b] Hydraulic Conductivity $m^3/m^2/d$
Coarse Sand	2	32	1,000
Gravelly Sand	8	35	5,000
Fine Gravel	16	38	7,500
Medium Gravel	32	40	10,000
Coarse Rock	128	45	100,000

a. The porosity is used to determine the actual flow velocity in the void spaces, and in equations (3) and (5) to determine the size of the SF bed. Porosity is equal to Void Volume/Total Volume, and is expressed as a percent.

b. Assuming non-turbulent, near laminar flow conditions, with clean. water. $m^3/m^2/d \times 24.6 = gpd/ft^2$.

The hydraulic conductivity (k_s) values in Table 5 assume that the media and the water flowing through it are clean so that clogging is not a factor. As discussed in a previous section, some clogging can occur in these systems, especially near the inlet zone where most of the suspended solids will be removed. As noted previously, the observed clogging represented less than 6 percent of the void spaces in the systems investigated. The majority of the material (>80%) was inorganic and believed to be the residue from construction activities, and should not, therefore, have a cumulative impact on hydraulic conductivity. It is, however, necessary to provide a large safety factor against these contingencies and adoption of an approach similar to that used in the design of land treatment systems (30) is proposed. **It is therefore recommended that a value < 113 of the "effective" hydraulic conductivity (k_s) be used for design. The initial design, for the same reasons, should not utilize more than 70 percent of the potential hydraulic gradient available in the proposed bed. These two limits, combined with an adjustable outlet for the bed discharge should ensure a more than adequate safety factor in the hydraulic design of the system.** These two limits will also have the practical effect of limiting the aspect ratio of the bed to < 3:1 for 0.6 m (2 ft) deep beds and to about 0.75:1 for 0.3 m (1 ft) deep beds. Using such a low value for hydraulic gradient will help maintain near laminar flow in the bed and further validate the use of Darcy's Law for design of these systems. Since this approach ensures a relatively wide entry zone, it will also result in a low organic loading on the cross sectional area and thereby reduce concerns over clogging.

In addition to the internal hydraulic concerns discussed above, it is necessary to have adequate inlet and outlet structures for the bed to assure proper distribution and collection of flow and maximum utilization of the media provided in the bed.

<u>Inlet Structures</u>

The inlet devices at operational systems include surface and subsurface manifolds, an open trench perpendicular to the flow direction, and simple, single point weir boxes. The manifold designs include a variety of features. In some cases perforated pipe is used for both surface and subsurface installations. In one case the subsurface- manifold utilized two to three valved outlets in the cell. A surface manifold developed by TVA uses multiple, adjustable outlet ports (31,32). This allows the operator to make adjustments for differential settlement of the pipe and to maintain uniform distribution of the wastewater. The proponents of subsurface inlet manifolds claim they are necessary to avoid the build-up of algal slimes on the rock surfaces and resulting clogging adjacent to a surface manifold. The disadvantages of a subsurface manifold are the inability for future adjustment and the limited access for maintenance. In one case, a buried manifold became clogged with turtles (entered the piping system from the preliminary treatment lagoon) and had to be removed.

A surface manifold, with adjustable outlets, seems to provide the maximum flexibility for future adjustments and maintenance and is recommended. Use of a coarse rock (8 to 15 cm [3 - 6”]) in this entry zone, coupled with an adequate hydraulic gradient for the bed, should ensure rapid infiltration and prevent ponding and algae development. In continuously warm and sunny climates, shading of this entry zone with either vegetation or a structure may also be necessary. In cold winter climates, some thermal protection for an above-surface manifold will probably be necessary.

Outlet Structures

Outlet structures in use at operational SF wetland systems include subsurface manifolds, and weir boxes or similar gated structures. The perforated subsurface manifold is the most commonly used device; however, the location of that manifold in the bed has varied considerably. In a few cases it has been located in a shallow trench, below the bottom of the bed, permitting complete drainage of the bed and development of the maximum hydraulic gradient for the system. In many cases, the manifold and/or the outlet ports have been located above the bottom of the bed, and in some cases the outlet ports have been iocated near the top of the bed. As indicated previously, this latter practice results in surface flow on the bed.

In most cases, the subsurface outlet manifold connects directly to the final discharge pipe, and/or to a concrete channel used for final disinfection. Some system designs in Europe and those in the U.S. derived from that practice (31,32), connect the subsurface manifold to an adjustable outlet for water level control. Flow then proceeds to either discharge or disinfection.

The use of an adjustable outlet was previously recommended to maintain an adequate hydraulic gradient in the bed. This device can also have significant, operational and maintenance benefits. The surface of the bed can be flooded to encourage, development of newly planted vegetation and to suppress undesirable weeds, and the water level can be lowered in anticipation of major storm events and to provide additional thermal protection against freezing during winter operations in cold climates.

The use **of a perforated subsurface manifold connected-to an adjustable outlet would seem to offer the maximum flexibility and reliability as the outlet device for SF** wetland systems. Since the manifold is buried and inaccessible following construction, careful grading and subbase compaction would be required during construction and clean-out risers should be provided in the line.

BOD_5 REMOVAL

When process kinetics were given any consideration, most of the existing systems in the U.S. and. Europe were designed as an attached growth biological reactor using a first order plug flow model, shown previously as equation (1):

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

The effluent BOD_5 (C,) in equation 1 is, as previously discussed, influenced by the production of residual BOD_5 within the wetland from decomposition of plant detritus and other naturally occurring organics. This residual BOD_5 is typically in the range of 2 to 7 mg/L. **As a result, equation (1) should not be used for designs for a final $BOD_5 < 5$ mg/L.**

It has been argued that plug flow kinetics do not apply to SF constructed wetland systems because the dye and tracer studies which have been performed do not exhibit the ideal plug flow response. Figure 13 presents the results of a tracer study, using lithium chloride (an inorganic, conservative tracer), conducted in 1980 at the operational SF wetland system at Carville, LA (27). Essentially 100 percent of the tracer was accounted for in the effluent, so this can be considered a valid study. It clearly did not exhibit ideal plug flow responses, but the curve is much closer to plug flow conditions than to the complete mix alternative. The centroid of the curve indicates an HRT of 48 hours, which is identical to the theoretical detention time calculated with the actual flow, measured porosity, and wetland cell dimensions. There was no surface flow during this test. Data from similar tests at other sites show an even closer resemblance to plug flow.

The shape of the curve on Figure 13 is similar to those observed with facultative ponds and similar wastewater treatment concepts where plug flow conditions are also assumed as the basis for design (33). Models are available for these systems which attempt to define conditions between plug flow and complete mix, but the difficulty in defining the necessary parameters has resulted in minimal use of these alternatives.

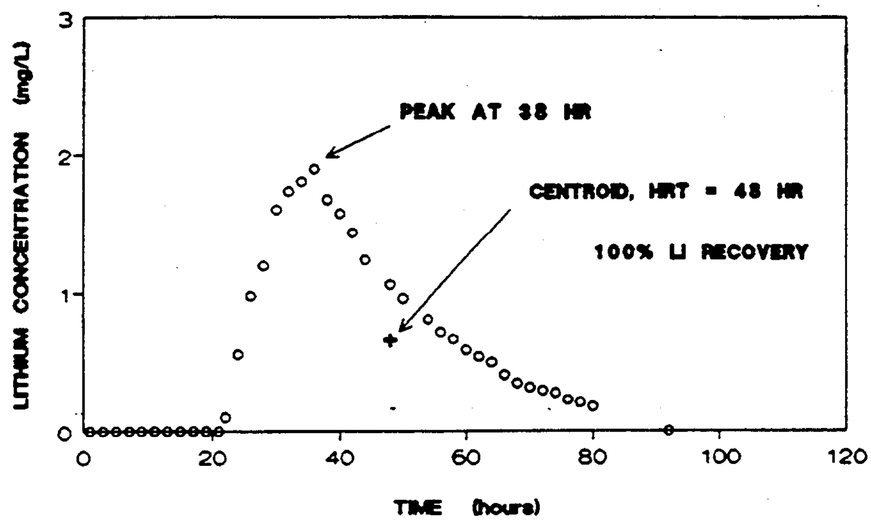

Figure 13. Lithium Tracer Study, Carville; LA.

The plug flow model is presently in general use. and it seems to provide a reasonable approximation of performance in these SF constructed wetlands. However, more sophisticated models have also been proposed for SF wetlands. One model includes a plug flow segment followed by three continuously stirred (complete mix) reactors in series (43). When sufficient data are available to validate these alternative models, they may replace the current approach. **In the interim, the use of the plug flow model is recommended for design.**

The “t” or hydraulic residence time (HRT) factor in equation (1) can be defined as:

$$t = \frac{nLWd}{Q} \qquad \textbf{(3)}$$

Where: n = effective porosity of media (see Table 5) % as a decimal
L = length of bed, m (ft)
W = width of bed, m (ft)
d = average depth of liquid in bed, m (ft)
Q = average flow through the bed, m^3/d (ft^3/d)

The Q value in equation (3) is the average flow in the bed to account for precipitation, seepage, evapotranspiration and other gains and losses of water during transit of the bed. This is the same value used in Darcy's Law for hydraulic design. Published values are usually available for an estimate of precipitation and ET losses at a particular site.

The d value in equation (3) is the average depth of liquid in the bed. If, as recommended previously, the design hydraulic gradient is limited to 10 percent of the potential available, -then the average depth of water in the bed will be. equal to 95 percent of the total depth of the treatment media in the bed.

The temperature dependence of the rate constant in equation (1) is defined as:

$$K_T = K_{20} (\Theta)^{(T-20^\circ)} \tag{4}$$

Where: K_T = rate constant at temperature T, d^{-1}
K_{20} = rate constant at 20°C, d^{-1}
= 1.104
Θ = 1.06

Combining equations (1), (3), and (4) produces:

$$\frac{C_e}{C_o} = e^{(-K_T LWdn)/Q} \tag{5}$$

Since the term LW in equation (5) is equal to the surface area of the bed, rearrangement of terms in equation (5) permits the calculation of the surface area (A,) required to achieve the necessary level of BOD_5 removal:

$$A_s = (L)(W) = \frac{Q[\ln(C_o/C_e)]}{K_T dn} \tag{6}$$

Where: A_s = bed surface area, m^2 (ft^2)

Other terms defined previously

The depth of media selected will depend on the design intentions for the system. If the vegetation is intended as a major oxygen source for nitrification in the system, then the depth of the bed should not exceed the potential root penetration depth for the plant species to be used. This will ensure availability of some oxygen throughout the bed profile, but may require management practices which assure root penetration to these depths. Table 6 presents results from the pilot system in Santee, CA (4) showing the relationship between root penetration and performance. The root depths shown in Table 6 are considered to be near the maximum practical limit to be expected. The design approach in Europe has also assumed a maximum depth of 0.6 m for *Phragmites* (15).

Table 6. Performance of Vegetated and Unvegetated SF Wetland Beds (4)

Bed Type	Root Depth, m	Final Effluent Quality, mg/L[a]		
		BOD_5	TSS	NH_3
Bulrush, *Scirpus*	0.8	5	4	2
Reeds, *Phragmites*	0.6	22	8	5
Cattails, *Typha*	0.3	30	6	18
No Vegetation	0.0	36	6	22

a. Primary effluent input (BOD_5 = 118 mg/L, SS = 57 mg/L, NH3 = 25 mg/L)

There is one operational system in the U.S. (Monterey, VA) with a media depth of 0.9 m (3 ft); the most commonly used depth is 0.6 m (2 ft). One system (Bear

Creek, AL) using *Typha* in fine gravel is obtaining excellent performance with a depth of 0.3 m, which matches the root penetration listed in Table 6 for that plant.

The final design and sizing of the SF bed for BOD_5 removal is an iterative process:

1. **Determine the media type, vegetation, and depth of bed to be used.**

2. **Determine by field or laboratory testing the porosity (n) and "effective" hydraulic conductivity (ks) of the media to be used.**

3. **Determine the required surface area of the bed, for the desired level of BOD_5 removal, with equation 6.**

4. **Depending on site topography, select a preliminary aspect ratio (L:W); 0.4:1 up to 3: 1 are generally acceptable.**

5. **Determine bed length (L) and width (W) from the previously assumed aspect ratio, and results of step 2.**

6. **Using Darcy's Law (equation 2) with the previously recommended limits (ks < 1/3 "effective" value, hydraulic gradient S < 10% of maximum potential), determine the flow (Q) which can pass through the bed in a subsurface mode. If this Q is less than the actual design flow, then surface flow is possible. In that case it is necessary to adjust the L and W values until the Darcy's Q is equal to the design flow.**

7. **It is not valid to use equation 5 with effluent BOD_5 (Ce) values below 5 mg/l. As previously discussed, these wetland systems export a BOD_5 residual due to decomposition of the natural organic detritus in the system.**

8. **In cold climates it is necessary to assume a design temperature for BOD_5 to first determine the required surface area. Thermal calculations are then necessary to determine the winter heat losses and bed temperature conditions during the design HRT. Further iterations of this procedure are necessary until the assumed temperature and the temperature determined by the heat loss calculations converge.**

Suspended Solids Removal

A kinetic model is not available for suspended solids removal; based on the data presentations in Figures 5 and 6, it is unlikely that such a model will be developed. It would appear that with an HRT of about 1 d the TSS will be removed to a level of about **10** mg/L. **As a rule of thumb it can be assumed that if the** system **is** designed **for a certain level of BOD_5 removal, the TSS removal will be comparable as long as significant long-term surface flow does not occur.** Long-term surface flow can result in short circuiting and the addition of TSS to the surface flow stream in the form of algae and plant detritus.

NITROGEN REMOVAL

As indicated in the discussion related to Figures 8 and 9, the major pathway for nitrogen removal in SF wetland systems is biological nitrification followed by denitrification (4,7,8). Based on the data presentations in Figures 8 and 9, it is clear that ammonia removal is not very effective in most of the operational SF systems studied. The limiting factor in ammonia removal via nitrification is believed to be the availability of oxygen in the media profile. This constraint is apparent regardless of the age or operational history of the system. The only two systems demonstrating excellent ammonia removal in Figure 9 have plant roots (and therefore available oxygen) throughout the profile, and sufficient HRT to complete the reactions. The majority of the systems constructed in the Gulf States in recent years have too brief an HRT (< 2d) and inadequate root penetration and development to depend on the plant roots as an oxygen source for significant nitrification. In these cases, other oxygen sources will be necessary.

There is no consensus on how much oxygen can be furnished by the vegetation in SF wetlands or on the oxygen transfer efficiency of various plant species. There is consensus that these emergent aquatic plants transmit enough oxygen to their roots to keep alive. The disagreement occurs over how much excess oxygen is then available to support biological activity in the root zone. Published estimates have ranged from zero to 45 gm $0_2/m^2/d$ of wetland surface area (4,7,34). This oxygen is not believed to be diffused throughout the subsurface profile, but is likely to be available-only on the surfaces of the smaller roots, within the root zone, in the bed. These aerobic microsites on the root hairs provide potential contact surfaces for the nitrification of ammonia. These microsites probably occur throughout the length of the bed, but the oxygen demand for BOD_5 removal is likely to limit the potential for nitrification in the front part of the system.

An alternative oxygen source, other than the plant roots, is not apparent in the completely submerged, but fully rooted, SF bed. Since these systems, as shown in Table 6, do provide significant ammonia removal (which is correlated to root depth), it seems likely that there is some oxygen available on the surfaces of the plant roots. Since it takes about 5 mg of oxygen to convert 1 mg of ammonia to nitrate, it is possible to estimate the oxygen that was provided in the Santee system to achieve the removals shown in Table 6. These results are summarized in Table 7.

The values in Table 7 do not define the actual oxygen production of the plants, but only indicate the amount of oxygen necessary to account for the known removal of ammonia at the Santee project. It seems likely that the plant roots were the source of this oxygen since no other source is apparent nor is any other alternative ammonia removal process likely. These values from Santee are also consistent with the performance of the Bear Creek system (see Figure 9 and related discussion). The average value of 7.5 gm $0_2/m^3/d$ is also consistent with, and near the low end of, the range reported by European investigators (7,341.

Table 7. Potential Oxygen from Vegetation at Santee, CA

Plant Type	Root Depth m	Available Oxygen $gm/m^3/d^a$	$gm/m^2/d^b$
Bulrush rush *(Scirpus)*	0.76	7.5	5.7
Reeds *(Phragmites)*	0.60	8.0	4.8
Cattails *(Typhal*	0.30	7.0	2.1
Average		7.5	

a. Available oxygen per unit volume of actual root zone.
b. Available oxygen per unit surface area of 0.76 m deep wetland bed.

The example below illustrates the application of these tentative relationships:

Assume: Q = 378 m^3/d, NH, in = 20 mg/L, NH, out = 2 mg/L
Plant type: cattails, Bed depth = 0.3 m
Average available oxygen 7.5 $gm/m^3/d$ (from Table 6)
BOD_5 in = 75 mg/L, BOD_5 level for start of nitrification = 20 mg/L
Media porosity = 0.4

HRT for BOD_5 = ln(75/20)/1.104 = 1.2 d (equation 3)
Area for BOD_5 removal to 20 mg/L = 0.4 ha (equation 6)

Nitrification:

Oxygen available = (0.3 m)(7.5 gm/m^3/d) = 2.25 gm/m^2/d

Oxygen required = (20 - 2)(378)(5) = 34,065 gm/d

Nitrification area required = (34,065)/(2.25) = 15,140 m^2 = 1.5 ha

Total HRT = 1.2 + 4.8 = 6.0 d Total Area = 0.4 + 1.5 = 1.9 ha (4.7 ac) (47 ac/mgd)

The example presumes a two-stage system in which the BOD_5 is reduced to about 20 mg/L followed by nitrification with oxygen supplied by the vegetation. The remaining BOD_5 in this second stage would be available for denitrification and the final effluent BOD_5 should be at background levels for these systems. The results of this example are consistent with the Santee performance and other locations where plant supplied oxygen is believed to be responsible for nitrification. **This suggests that nitrification via this pathway is possible when both fully developed root systems and sufficient detention time are available.** The limiting factor in this case is the rate at which the plant can provide oxygen. **However, there. are, at present, limited independent data** to **support this tentative procedure derived from the experience at Santee, which should be used with caution for design until further research results are available. It will also be necessary to identify the seasonal and temperature influences on the procedure to achieve successful application in cold climates.**

Adoption of this approach for nitrification would then require at least a six-day HRT, and the cost of the rock or gravel media could be prohibitive. Alternative. methods for oxygen supply and nitrification, therefore, deserve consideration.

These alternative methods may include: mechanical aeration after BOD_5 reduction, provision of open water zones for surface reaeration, use of the overland flow land treatment concept for nitrification, and the use of parallel nitrification cells operated on a batch type fill and draw basis to return atmospheric oxygen to the media profile. Another approach, which is under construction for both FWS and SF cells in Kentucky, will superimpose a vertical flow, recirculating bed, composed of fine gravel, for nitrification on top of the existing system. In this case the recirculating filter will be located at the influent end of the bed. The effluent from the nitrification bed will mix with the normal untreated flow in the wetland cell where the nitrates produced should be denitrified. With this approach the wetland cell need only be sized for BOD_5 removal requirements since nitrification is expected to occur in the recirculating bed. The concept offers promise for retrofit of the many existing systems which are having difficulty meeting their discharge limits for ammonia. The concept' may also be more cost-effective than a minimum of a six-day HRT for wetlands where plant-available oxygen is assumed as the sole source for nitrification. The hydraulic design of such a system will have to include consideration of the recycle flow within the horizontal portion of the bed. Data collection is planned for a 12-month period at the Kentucky site and should lead to optimization of the concept for application elsewhere.

Three other design models for estimating ammonia removal in constructed wetland systems are available in the literature. The WPCF Manual of Practice (8) presents a model for ammonia removal based on a regression analysis of both FWS and SF systems, for annual average conditions with no provision for temperature correction:

$$As = \frac{(0.01)(Q)}{[(1.527)(lnNH_e) - (1.050)(lnNH_o) + 1.69]} \qquad (6)$$

Where:

A_s = wetland surface area required for ammonia removal, ha

NH_e = effluent ammonia, mg/L

NH_o = influent ammonia, mg/L

Q = "average" flow through system, m^3/d

Based on pilot scale work with SF wetlands, Bavor (12) has presented the following equation for ammonia removal in those systems:

$$A_s = \frac{(Q)(\ln NH_o/NH_e)}{K_T dn} \qquad (7)$$

Where: A_s = required surface area, m^2
K_T = **temperature** dependent rate constant, d^{-1}
= $K_{20}(1.03)^{(T-}$
K_{20} = $0.107\ d^{-1}$

d = average depth of liquid in the bed, m
n = effective porosity of bed media, % as a decimal.

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

Based on a regression analysis of a limited number of systems, Hammer and Knight (41) have proposed the following equation for ammonia removal in constructed wetland systems:

$$A_s = \frac{(0.001831)\,(NH_o)\,(Q)}{NH_o + 0.16063} \qquad (8)$$

Where: A_s = required wetland surface area, ha
NH_0 = lnfluent ammonia concentration, mg/L
NHe = desired effluent ammonia concentration, mg/L
Q = average flow through the system, m^3/d

Equations 6, 7, and 8 produce comparative results down to an ammonia effluent concentration of about 2 mg/L. However, all three equations will predict a total treatment area at least twice that resulting from the initial procedure presented, which depends on the oxygen produced by the vegetation. The total HRT for the initial example was six days; the HRT for the same conditions, using equations 6, 7, and 8, ranges from 13 to 19 days. This may be because all three procedures were derived from site conditions where oxygen may have been slightly to significantly deficient in the water flowing through the system. None of these equations will provide an accurate prediction of ammonia removal for the SF systems shown on Figures 8 and 9. This is believed to be due to the almost complete lack of oxygen in most of the systems and, conversely, to the presence of available oxygen in the two systems which demonstrated excellent ammonia removal Equations 6, 7, and 8 may provide a reasonable order-of-magnitude estimate of ammonia removal performance of many of the existing SF constructed wetlands which do not have sufficient oxygen from the plants or other sources.

VEGETATION SELECTION AND MANAGEMENT

Most of the constructed wetlands in the U.S utilize one or more of the plant species listed in Tables 6 and 7. About 40 percent of the operational SF systems use only *Scripus. Phragmites* is the most widely used species in the European systems. A number of systems in the Gulf States also used a number of flowering plants for aesthetic reasons. These soft tissue plants decompose very rapidly and can affect water quality in the effluent. Many locations adopted a routine fall harvest to remove these plants before they died or suffered frost damage. There have been some attempts to create a plant diversity similar to that present in a natural marsh; this approach is more expensive and the intended diversity can be difficult to maintain.

Any of the three species listed in Tables 6 and 7 are suitable for use in SF systems. If the plant is expected to provide a significant treatment function, then the depth of the bed should not exceed the potential root penetration depth. ***The Phragmites* used in many European systems offer several advantages for a low** maintenance **treatment system.** They will grow and spread faster than bulrush; their roots should go deeper than cattails; and they are not a food source for muskrats and nutria which have been a problem for cattail and bulrush wetlands. However, the habitat values for a *Phragmites* system are probably less than for other plant species.

A number of systems in the Gulf States utilize an annual harvest, regardless of the plant species used. In contrast, routine annual harvesting is not practiced in Europe or at most other systems in the U.S. It may be useful to remove undesirable weeds during the early part of the growing season for the first few years of operation. Flooding of the bed surface after the initial planting can help reduce weed infestation. A routine annual harvest of the entire system provides minimal benefits and is not recommended. It is also suggested that the use of soft tissue flowering plants be avoided and thereby eliminate the need for their annual harvest and related maintenance.

Water level management in the SF bed is not only helpful for weed control, but can also be used to induce deeper root penetration. Based on experience in Europe, it is claimed that if the water level in the bed is gradually lowered in the fall of each year the roots will penetrate to greater depths. A three year period is considered necessary for *Phragmites* roots to reach their 0.6 m potential depth. Although this approach has not been tried in the U.S. it should be successful, but it may have to be repeated every year for the operational life of the system. The alternative is a root zone where the major mass of roots are limited to the top ± 0.25 m in the portions of the bed where nutrient concentrations are high.

The *Typha* roots penetrated to the full depth of the shallow bed (0.3 m) at the constructed wetland system in Bear Creek. This system was lightly Ioaded, and since it served a public high school it received essentially no wastewater flow at night, during the weekends, and during the summer months. These dormant periods when the nutrient concentrations would be at low levels in the bed are believed to be a contributing factor to the rapid penetration of the root system. It might be possible to replicate this experience at other locations with continuous flow if the system were divided into two or more parallel cells and if the initial flow were less than the ultimate design capacity. In this case, the wastewater flow could be alternated between the cells during the summer and fall months and result in enhanced root penetration in the temporarily dormant cell.

At the Santee pilot system, the plant roots penetrated to the depths shown in Table 6 without any special manipulation of water levels or other similar management activities. The six-day HRT combined with the warm climate and continuous growing season at this site probably ail contributed to low nutrient conditions in the latter part of the bed, thereby inducing root growth and penetration.

As root development commences, nitrification should be enhanced; this should be rapidly followed by denitrification (as long as a carbon source is available). The resulting loss of nitrogen to the atmosphere further reduces the availability of nutrients in the bed and may promote further progressive root development in the portions of the bed where the nitrifying organisms can successfully compete for the available oxygen. It is likely that root development would still be limited in the front part of such a bed where the oxygen demand for BOD_5 removal would limit the development of the nitrifiers. In this area much of the nitrogen would still be in the ammonia form and the plant roots would not have to penetrate deeply to obtain sufficient nutrients.

A critical difference between the Santee project and many of the operating systems in the Gulf States is the six-day HRT at the former versus the one- to two-day HRT at-the latter systems. The use of new plant species, harvesting and other vegetation management activities are unlikely to improve ammonia removal performance at these short detention time systems.

Deep penetration of the roots in these short detention time beds may be possible if most of the flow is actually occurring on top of the bed. In this case there would be minimal flow through most of the bed profile, resulting in low nutrient levels and deeper root penetration. The deeper root penetration in this case would not result in improved treatment since the roots are not in contact with the bulk of the wastewater. Hydraulic improvements are necessary for such systems to maintain flow throughout the full bed profile, but the short detention time will still be a limiting factor.

CHAPTER 5

CONSTRUCTION DETAILS

It is prudent for all but the smallest systems to divide the surface area required for treatment into two or more cells, in parallel, to provide flexibility for operation and maintenance. Each cell should be provided with an access ramp for maintenance equipment.

The hydraulic performance of- these constructed wetlands can be significantly influenced by improper construction activities. Initial excavation and grading must be carefully controlled to avoid low spots and preferential flow down one side of the cell, or erratic cross flow within the cell. Past experience has shown that even with careful initial grading, the cell profile can be disrupted by uncontrolled truck traffic bringing the gravel or rock media into the bed. It is suggested that the native soils at the bottom of the SF wetland cell be compacted to the same degree required for a highway subgrade to withstand damage from trucks and other equipment, and then construction vehicle access to the cell should be limited during wet conditions. The liner, or other impermeable barrier (if needed) goes on top of the compacted soil, and the bed media is placed on top of the liner. If a membrane liner is used, a layer of sand-is suggested to prevent puncture of the liner.

Selection of media type and size is critical to the successful performance of the system. Unwashed crushed stone has been used on a large number of projects. Truck delivery of such material during construction can lead to problems due to segregation of the fines in the truck during transit, and then the deposition of all of those fines in a single spot when the load is dumped. This can result in numerous small blockages in the flow path and internal short circuiting within the system. Washed stone or gravel is preferred. Coarse aggregates for concrete construction are commonly available throughout the U.S. and would be suitable for construction of SF wetland systems.

Appropriate inlet and outlet devices have been discussed in a previous section. It is essential that both devices provide for uniform distribution and/or collection. Some method for controlling the water level in the bed should be utilized following the effluent manifold.

The vegetation on most of the existing SF wetland systems has been planted by hand, with the initial spacing ranging from 0.3 to 1.2 m (1 - 4 ft). The use of individual root/rhizome material with a growing shoot at least 0.2 m (8 in) in length is suggested. The use of locally available plants is recommended since they have already adapted to the environmental conditions; however, these plants are also available from commercial suppliers. The root/rhizome material should be placed in the gravel or rock media, at a depth equal to the expected operational water level. The growing shoot should project above the surface of the media. If mature, locally available plants are used, they can be separated into individual root/rhizome/shoot units; in these cases the mature stem should be cut back to < 0.3 m. (< 1 ft) before planting.

The water level in the bed should be maintained slightly above the media surface during planting and for several weeks thereafter to suppress weed development and promote growth of the planted species. An initial, moderate application of commercial fertilizers will also enhance plant development. Wastewater applications can probably commence six to eight weeks after planting if vigorous growth is observed. An early spring planting is preferred whenever possible. The plants, and therefore system performance for nitrogen, may not begin to reach maturity and equilibrium until late in the second growing season..

C O S T S

A recent survey (13) indicates that the capital costs for SF wetland systems averaged around $200,000 per hectare ($87,000/ac) and the FWS systems were about $50,000 per hectare ($22,000/ac). The major cost difference of the two systems is in the expense of procuring the rock or gravel media, hauling it to the site, and placing it. Although the construction cost per hectare is higher for SF wetlands, the design flow rates at currently operating SF systems are also much higher than at the FWS type. As a result, for the systems included in the survey, the unit cost is $163/m^3 ($0.62/gal) of wastewater treated for the SF type, and $206/m^3 ($0.78/gal) for the FWS type. However, many of these early SF systems were much smaller than more recently designed SF systems, so the current unit costs are likely to be higher.

As shown in Table 3, the BOD_5 rate constant for SF wetlands is about double the value for FWS wetlands. However, that does not mean that FWS wetlands are double the size of SF wetlands for the same conditions and performance expectations. The two systems may operate at different water depths, and the increased “porosity” (space available for water flow) in the FWS case compensates to some-degree for the reduced rate constant. In the typical case, the FWS wetland might be about 70 percent larger than a SF wetland to achieve the same BOD_5 performance. Figure 14 presents the cost distribution for SF wetlands as derived by Conley, et al. (15).

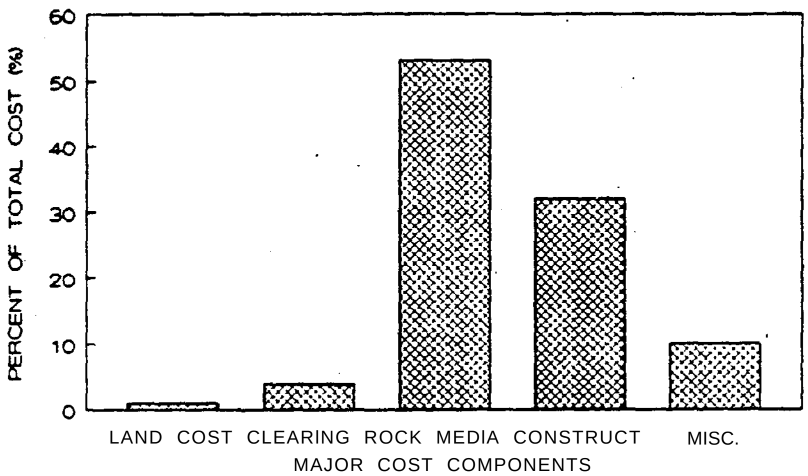

Figure 14. Cost Distribution for SF Constructed Wetlands.

The land costs are quite low in this case but procurement and placement of the rock media represents about 53 percent of the total construction costs. This is an expense not required by the FWS wetland alternative. This cost distribution does not include the cost for any collection or pumping systems in the community or any preliminary treatment.

Figure 15 presents the actual construction costs for a small (0.4 ha) SF system in southern Louisiana (35). The actual costs are referenced to the left axis and the percent of total costs to the axis on the right. The cost of the rock media is comparable to that shown on Figure 14. Rock is not locally available at this site so the material was barged from Arkansas to Louisiana and then trucked to the site. A lower cost might be expected if rock or gravel media were available locally. The land costs in this case were high since the system is located in and serves a large. residential subdivision.

The most cost-effective choice between SF and FWS constructed wetlands will then depend on the local costs for land and for the SF media. If the land costs are as low as shown in Figure 14, the FWS concept may be favored, assuming the additional land needed was available; if the land is as expensive as shown on Figure 16 the SF concept may be more economical, assuming a source of low-cost media is locally available.

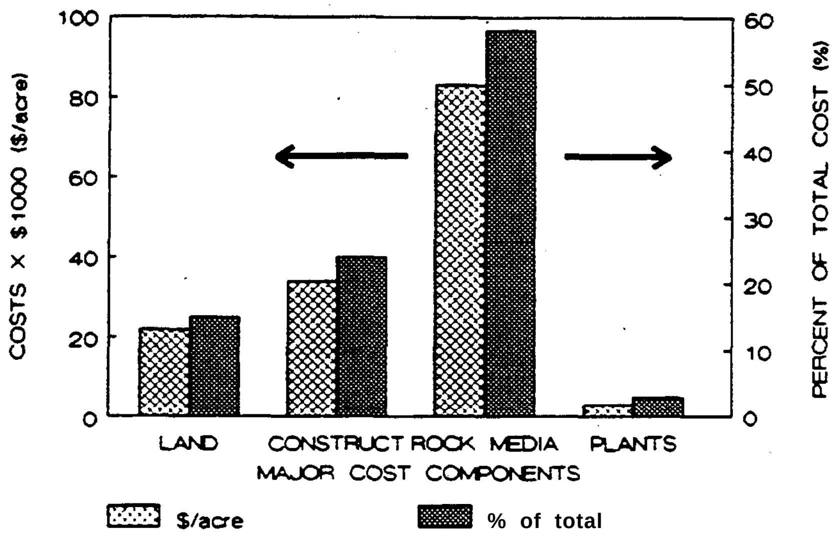

Figure 15. Actual Cost Distribution for an SF System in Louisiana.

If significant ammonia nitrification or nitrogen removal is required, then the FWS concept with an HRT of 8 to 10 d may still be more economical since about 6-d HRT is required for an SF system depending only on plant available oxygen. If a recirculating filter is used in combination with the SF concept, the total HRT might be in the range of 2 d (for warm climates) and it might then be the more cost-effective alternative.

The select/on of the more cost-effective alternative will depend on treatment goals, the availability and cost of land in the area, and on the cost of the media for the SF alternative. Other factors include mosquito and positive odor control offered by the SF concept, and less concern over public access. These factors would favor the use of the SF concept in close proximity to public facilities and dwellings.

CHAPTER 6

ON-SITE SF WETLAND SYSTEMS

The SF wetland concept is particularly well suited for on-site applications because of the advantages (odor and vector control, public access issues) of the process. Although there is no general consensus on design, a large number of on-site SF wetland systems have been constructed and placed in operation in Louisiana, Arkansas, Kentucky, Mississippi, Tennessee, Colorado, and New Mexico. These systems serve single-family dwellings, public facilities and parks, apartments, -and commercial developments.

In general these systems can provide better than secondary levels of treatment and in some states a surface discharge has been permitted (disinfection may be required depending on local conditions). A standard multi-compartment septic tank is the typical preliminary treatment device. Guidelines for design of these small SF wetland systems can be obtained from state agencies in Louisiana, Arkansas, Kentucky and the TVA (17,36). Many on-site systems have also been designed using the same first order plug flow model described previously. Systems of this type in the southwestern U.S. are typically designed for an effluent with < 10 mg/L BOD_5, < 10 mg/L TSS, and 10 mg/L total nitrogen, with an HRT in the wetland bed of > 6 d (42).

A review of the various design methods in current use suggests that the TVA method **and the plug flow model offer the most rational approach to design;** these two and the approach used in Louisiana will be discussed in detail.

LOUISIANA METHOD

The method used in Louisiana (36) is derived from the-work of Wolverton, et al. (3), and as applied to larger scale municipal systems in the region. The system typically involves a single narrow trench, excavated in clay, or lined in more permeable. soils. The HRT in the bed is one to two days (assuming the total specified bed volume is actively contributing to treatment). The performance in existing systems' has been somewhat variable, but effluent expectations are typically in the range of BOD_5 < 30 mg/L, TSS < 30 mg/L, and no limits on nitrogen.

The trenches in the Louisiana systems typically contain a 0.6 m (2 ft) depth of crushed stone, with a layer of smaller gravel on top. The trench bottoms are typically flat and the outlet does not allow adjustment of water level in the bed. Ammonia removal is not an issue with these on-site systems and is not required. Ornamental flowering plants are often used as the vegetation of choice in these systems. The bed outlet maintains the water level at mid-depth of the bed to provide a water source for the ornamental plants.

This constraint probably reduces the “active” volume in the bed which is potentially contributing to treatment. A better approach might be to use an adjustable outlet and more drought resistant plants such as *Phragmites.* The septic tank effluent is applied to these beds by a perforated pipe located at about the mid-point of depth, and extending about 1/3 of the length of the trench. It is likely that the actual HRT in these systems, at design flow, is less than the anticipated one to two days because of the hydraulic constraints imposed by the inlet and outlet devices.

The specific guidelines for these systems in Louisiana are (36):

1. Systems 1.5 m3/d (400 gpd) or less:

- Three-compartment septic tank, 1-.9 m3 (500 gal) in first compartment, 0.9 m3 (250 gal) each in second and third compartment.

- Wetland trench dimensions: 0.46 m (1.5 ft) deep, 0.61 m (2 ft) wide and 32 m (105 ft) long, or 0.9 m (3 ft) wide and 21 m (70 ft) long. The top 150 mm (6 in) is maintained in a dry state; therefore the required treatment volume is 5.9 m3 (210 ft3) for design flow < 1.5 m3/d.

- The rock in the treatment zone should be clean, washed rock ranging from 25 to 76 mm (1 to 3 in) in size. The top layer can be the same material or smaller in size.

- The trench should be lined with a plastic film at least 12 mil in thickness or other equivalent material.

- The effluent pipe for the bed will be a perforated pipe laid as a header across the full width of the bed, so that the high water level in the bed is maintained at a depth of 305 mm (1 ft) at all times.

- The influent pipe to the wetland bed will be a perforated pipe, extending 3 m (10 ft) into the bed on the centerline, with the invert of the pipe at the liquid level in the bed as defined above.

- Aquatic plants approved for use include:

Arrow Arum *(Peltandra virginica)*
Cattails *(Typha latifolia*
Pickerelweed *(Pontederia cordata)*
Calla Lily *(Zantedeschia aethiopica)*
Elephant Ear *(Calocafia esculenta)*
Arrowhead *(Sagittaria latifolia)*
Reed *(Phragmites)*
Canna Lily *(Canna flaccida)*
Bulrush *(Scirpus americanus)*
Ginger Lily *(Hedychium coronatum)*

- Typical plant density is about 60 plants per bed, evenly spaced, with a maximum of 0.46 m (1.5 ft) between individual plants. Soil should be washed from roots before planting and roots must be placed below the design liquid level in the bed. Dead vegetation must be regularly removed from the bed and additional plants installed as necessary.

- Adjacent surface runoff must be excluded from the bed by appropriate grading around the site.

- The bed should be 15 m (50 ft) from any well or potable water line, and at least 3 m (10 ft) from any property line.

2. Systems larger than 1.5 m3/d (400 gpd):

The three compartment septic tank will have a total volume equal to 2.5 times the design daily' flow. The wetland bed must be increased in volume by 1.4 m3 (50 ft3) for every 0.4 m3/d (100 gpd) of flow above 1.5 m3. Other requirements are the same as above.

TVA METHOD

The TVA concept (17) uses a design approach which considers the factors controlling the hydraulic performance of the bed, and the organic loading (kg $B0D_5/m^2/d$) on the entry zone cross sectional area to avoid potential clogging. The total area required for treatment is then divided into two equal cells in series. The transfer structure, between the cells, is equipped with an adjustable flow device to control the water level in the first cell. Both surface manifolds with adjustable outlets and buried manifolds have been used for the inlet structures.

The second cell in the TVA systems is the same size as the first cell and is unlined to permit seepage of the treated wastewater into the ground. The present procedure for design of these cells does not take into account the permeability of the soil beneath the unlined cell, but in the general case this second cell provides an infiltration area comparable to that required for conventional leaching beds or trenches.

The specific TVA guidelines are (17):

- Determine design flow (Q) from the home, a typical rate is 0.45 m3/d (120 gpd) per bedroom; state or local requirements will govern.

- Determine the daily organic loading (OL). A value of 0.045 kg BOD_5/person/d (0.1 lb/d) in the effluent leaving the septic tank is acceptable.

- Determine the total surface area (As) of the bed using the previously determined design flow and a surface loading of 31.9 m^2 of bed area per m^3/d of flow (1.3 ft^2/gpd) for a bed depth of 0.3 m (1 ft). A bed 0.46 m (1.5 ft) deep should be designed for a loading of 21.3 m^2/m^3/d (0.87 ft^2/gpd). The shallower depth is preferred if site conditions permit.

- The cross-sectional area (A,) is determined by comparing the impact of Darcy's Law, and organic loading criteria, and adopting the larger of the two cross sectional areas.

Organic loading factor: L_o = 4.097 m^2/kg BOD_5/d (20 ft^2/lb BOD_5/d)

Cross-sectional area: $A_C = (L_o)(OL)$

Darcy's Law: $A_c = Q / k_s S$

Where:
A_c = cross-sectional area of bed, m^2(ft^2).
Q = design flow, m^3/d (ft^3/d).
k_s = hydraulic conductivity
= 259 $m^3/d/m^2$ (850 $ft^3/d/ft^2$)
S = hydraulic gradient.
= 0.005 for flat bottom.
= 0.01 to 0.02 for 1 to 2 percent bottom slope.

- The bed width (W) can be determined since the depth has been selected. and the cross-sectional area determined above.

- The total bed length (L) can be determined by dividing the previously calculated surface area by the bed width. A two-cell system would have a length of L/2 for each cell.

- An impermeable liner *or* low permeability clay bottom is necessary for the first cell in a two-cell system. Synthetic liners using 20-30 mil polyethylene or polyvinyl chloride are acceptable.

- Washed river gravel is the preferred bed media, using pea gravel sizes up to 1.3 cm (0.5 inch) in diameter. The inlet and outlet zones of each cell should use 5 to 10 cm (2 to 4 inch) stone for a 0.6 m (2 ft) length around and over the influent and effluent manifolds.

- Schedule 40 PVC pipe, with a 10 cm (four-inch) diameter is acceptable for inlet and outlet manifolds, with 2.5 cm (one-inch) drilled holes.

- The outlet manifold should connect to either a swiveling standpipe or a flexible hose in a manhole to permit water level control in the bed.

- Surface water must be diverted away from the bed.

- The top of the bed should be mulched.

- Local plant species should be used on the bed. The preferred species include: cattail, sedge, rush, soft stem bulrush, and reeds. Decorative, flowering plants can be used around the edges of the bed.

An evaluation of this TVA design method reveals several concerns.. It does not take into account the temperature influence on performance when applied in cold climates. It assumes that infiltration and percolation into the ground will be acceptable but it does not include any field measurements of the actual hydraulic capacity of the in-situ soils. It does not provide a method for estimating nitrogen removal in the system for locations where ammonia or nitrogen control is required. The HRT in these systems will probably range from 3 to 4 days at design flow, which may not be sufficient to deal with the ammonia levels leaving most septic tanks. **In** summary, the **TVA approach is probably conservative and acceptable where in-ground disposal is the final discharge pathway and where nitrogen limits do not apply to such a discharge.** In these cases, some field measurements to determine the actual hydraulic capacity of the soil is also recommended. The TVA approach may not be sufficiently conservative for design of systems with a surface discharge in locations with cold winter climates and should be used with caution in these cases.

PLUG FLOW MODEL FOR ON-SITE SYSTEMS

The plug' flow model has been presented and discussed in detail in previous sections of this report. **A** summary listing of simplified criteria and procedures is given below to demonstrate the application to small scale on-site systems:

- Determine the design flow; 0.23 m^3/d (60 gpd) is a reasonable assumption for per capita flow for residential systems. State or local criteria will govern.

- Use a multi-compartment septic tank. Use one tank for single-family dwellings; use two or more tanks in series for larger scale (> 10,000 gpd) projects. The total volume of the tank(s) should be at least twice the design daily flow.

- Assume that the BOD_5 (C,) leaving the septic tank(s) is conservative > 100 mg/L.

- Assume that the wetland effluent BOD_5 (C,) will not exceed 10 mg/L.

- Use clean, washed gravel as the treatment media in the bed, size range 1.25 - 2.5 cm (0.5 - 1 inch), with a total depth of 0.6 m (2 ft). For design assume the "effective" water depth in the bed is 0.55 m (1.8 ft). Reasonable estimates are: hydraulic conductivity (k,) = 1500 m^3/m^2/d (5000 ft^3/ft^2/d; porosity = 0.38. If a large number of systems are to be installed using the same materials, field or laboratory testing for hydraulic conductivity (k,) and porosity (n) is recommended.

- Reeds *(Phragmites)* are the preferred plant species.

- Estimate the summer and winter water temperatures to be expected in the bed. In the summer and in year-round warm climates 2O°C is reasonable. In cold winter climates, a winter water temperature of = 6° C is a reasonable assumption.

- Determine the bed surface area with:

$$A_s = (L)\ (W) = \frac{Q[\ln(C_o/C_e)]}{K_T d n}$$

(6)

- As a safety factor, use a rate constant K_{20} which is 75 percent of the base value (1.104 d^{-1}). So, for design of small on-site systems K_{20} = 0.828 d^{-1}.

- At 2O°C, and with the other factors defined above, this equation reduces to:

  Metric: A_s = 13.31(Q) = m^2 (Q in m^3/d)

  U.S unit A_s = 4.07(Q) = ft^2 (Q in ft^3/d)

  At 6°C:

  Metric: A_s = 30.1(Q) = m^2 (Q in m^3/d)

  U.S units A_s = 9.2(Q) = ft^2 (Q in ft^3/d)

- **Adjustments for other temperatures, other media types, etc., should use the basic design equations.**

- Adopt an aspect ratio (L:W) of 2:1, calculate bed length (L) and width (W) since the surface area was determined above. In the general case, an aspect ratio of 2:1, or less, with a bed depth of 0.6 m (2 ft) will satisfy the Darcy's Law constraints on hydraulic design of the bed, so hydraulic calculations are not required. **If site conditions will not permit use of an L:W of 2:1 for the bed, and a 0.6 m bed depth, then hydraulic calculations as described previously will be necessary.**

This approach will give an HRT of about 2.8 d (at 2O°C) in the bed, which is more than adequate for BOD_5 removal to < 10 mg/L. If nitrogen removal to 10 mg/L is required, the size of the system should be doubled to produce an HRT of about six days. Nitrogen removal during the winter months, in cold climates may require an HRT of about 10 days. In these cases, heat loss calculations should be performed to assure that the bed is adequately protected against freezing.

- Construct the bed as a single cell for single-family dwellings. Use multiple cells (at least two) in parallel for larger systems.

- Use clay or a synthetic liner to prevent seepage from the bed.

- Construct the bed with a flat bottom and a perforated effluent manifold at the bottom of the bed. A perforated inlet manifold a few inches above the bottom of the bed is adequate for most small systems. These inlet and outlet zones should use 2.5 to 5 cm (one- to two-inch) washed rock for a length of about 1 m (3 ft), and for the full depth of the bed.

- The effluent manifold should connect to either a swiveling standpipe or a flexible hose for discharge, to allow control of the water level in the bed.

- The inlet and effluent manifolds should have accessible cleanouts at the surface of the bed.

The system as described above should produce an effluent with $BOD_5 < 10$ mg/L, TSS < 10 mg/L, and TN < 10 mg/L, and should therefore be suitable for either surface or in-ground discharge. Percolation tests, basin infiltration tests, or other field or laboratory techniques should be used to determine the hydraulic capacity of the native soils for in-ground discharge systems. The position of the groundwater table in the proposed disposal area must also be determined. The larger the system, the more sophisticated this testing should be. Because of the very clean nature of the wetland effluent, it should be possible to design the final disposal bed or trenches at about one third to one half the size that would be required for direct septic tank disposal

For example, a typical conventional on-site system for a family of four (1 m^3/d, 300 gpd) might include a 4 m^3 (1000 gal) septic tank and a 46 m^2 (500 ft^2) infiltration area in a sandy loam soil. Addition of a wetland component with a 6 d HRT would require about 28 m^2 (300 ft^2) of area. If appropriate credit for the higher level of treatment is allowed, the total area for the wetland cell and the infiltration bed could be less than 46 m^2 (< 500 ft^2).

An evaluation of this design approach suggests that it is both more conservative and more flexible than the other two. It allows adjustment in the design for different temperature conditions, bed configurations, and either surface or subsurface discharge options. It is appropriate for single-family dwellings and larger systems serving public buildings and commercial establishments. **As a result, this approach is recommended** for design of on-site systems.

CHAPTER 7

OTHER POTENTIAL APPLICATIONS FOR SF CONSTRUCTED WETLANDS

Constructed wetlands are in use or have been proposed for the treatment of landfill leachate, agricultural runoff, feed lot runoff, acid mine drainage, drainage from coal and ash piles, stormwater runoff, and combined sewer overflows (CSO). In most of the operational systems, the wetland concept in use is most often the free water surface type (FWS). This section will briefly examine the potential for the use of subsurface flow wetlands (SF) for these applications.

The SF concept involves submerged flow in a bed of gravel or crushed stone, and the design intent is to maintain that condition at all times to realize the treatment potential and to avoid surficial short circuiting. Implicit in that approach are the assumptions that the flow will be relatively steady state, and that the wastewater does not contain large quantities of inorganic solids which might lead to rapid clogging of the pore spaces in the bed.

STORMWATER SYSTEMS

The SF system can accommodate the diurnal flow pattern for domestic, municipal, and industrial wastewater flow but are not well suited hydraulically for treatment of stormwater or CSO discharges where the peak flow may be several orders of magnitude higher than the "average" flow. It would be uneconomical to provide a gravel bed large enough to contain, in a subsurface mode, the peak storm event. The use of sediment traps ahead of the wetland could reduce the impact of inorganic solids, and the construction of sufficient freeboard to contain the storm flow above the SF bed with subsequent steady-state discharge through the bed may sound possible. However, it is unlikely that the full bed volume would be involved in treatment of the ponded water; preferential discharge in a zone near the outlet is a more likely flow path. In this case the bulk of the media may not contribute to treatment during the major storm events. **As a result, there seems to be negligible** advantages **for the use of gravel media, so a FWS constructed wetland is probably the preferred choice for stormwater and CSO discharges.** A FWS wetland may tend to serve as a batch type plug flow reactor in response to storm events. The major treatment responses (other than sedimentation) will probably occur in the standing water between the storm events. That treated water will then be displaced by the incoming flow from the next storm event.

LANDFILL LEACHATES

SF constructed wetlands are being used for treatment of landfill leachates. In these cases the flow is relatively uniform **and** low in inorganic suspended solids and the SF wetland can be used to advantage. Collection of additional data is necessary for a better understanding of the capabilities of the wetland for removal of the complex organic and inorganic materials which may be found in leachates.

‘MINE DRAINAGE

SF wetlands can also be used for treatment of mine drainage, where again the flow is relatively uniform and the treatment does not result in the precipitation and deposition of large quantities of inorganic solids in the bed. FWS wetlands are more commonly used in the eastern states to treat drainage from coal mines, as well as coal and ash piles In these cases, the precipitation and storage of iron and manganese in the wetland cell is the major treatment goal.

AGRICULTURAL RUNOFF

Agricultural runoff is also subject to intermittent peak events and high inorganic sediment loads, so FWS wetlands would again seem best suited for these cases. Discharges from feed lots and other sources of high organic strength wastewater can be treated by either SF or FWS constructed wetlands. In both cases, preliminary treatment for solids and BOD_5 reduction is necessary.

In summary, the use of SF wetlands is probably best suited, because of the hydraulic constraints imposed by the media, for the treatment of wastewaters with relatively low solids concentrations, under relatively uniform flow conditions, and in locations where the advantages of the subsurface flow mode (ie: no insect vectors, higher rate of treatment, etc) are important considerations.

CHAPTER 8

RESEARCH NEEDS

Research needs to further advance and understand the SF wetland concept were discussed at an EPA workshop held in New Orleans, LA on September 24, and 25, 1992. Participants in that workshop are listed in Appendix A. These research needs were categorized as “high”, “medium” and “low” priority needs, and are summarized below.

HIGH PRIORITY RESEARCH NEEDS

A better understanding of the nitrogen removal and nitrogen transformations occurring in these SF systems is necessary. This should lead to the development of rational temperature and possibly seasonally dependent design models for nitrogen removal.

Additional data collection is necessary on the spatial responses to BOD_5 within the SF wetland bed to permit development and validation of improved design models to replace the interim procedures now in use.

Further research is needed on identifying the oxygen needs and sources in these SF systems. The role of the plant roots in providing this oxygen is especially important.

The use of plant types other than reeds, rushes, and cattails needs to be investigated to determine if optimum species exists. The need for routine plant harvest also deserves study.

Most operational SF wetlands demonstrating successful ammonia removal have an HRT of about six days or more. The system at Benton, LA has apparently demonstrated high ammonia removals with an HRT of < 1 d. The factors responsible for this performance at Benton need to be defined.

Although recent studies indicate minimal clogging in the beds investigated, the effort needs to be continued to determine the long-term risks of clogging.

Efforts should continue to collect reliable performance data from full-scale operating systems to confirm and supplement the results from laboratory, greenhouse, and pilot-scale research.

MEDIUM PRIORITY RESEARCH NEEDS

Intermittent or batch type flow to alternating beds might enhance the oxygen status and therefore the ammonia removal capability. The approach should be tested; and then demonstrated if promising.

A design model for pathogen removal in these systems would be useful, with the dependency on time and temperature defined.

The response to complex organic and inorganic compounds in industrial and agri-chemical wastes needs definition.

LOW PRIORITY RESEARCH NEEDS

The use of specialized media for improved phosphorus removal could be developed and demonstrated.

The removal mechanisms for metals in these systems could be better defined.

The role of the plant roots (other than serving as an oxygen source) in maintaining desirable bed conditions needs study.

The management of variable flows from communities which suffer from high infiltration/inflows in their sewer systems needs consideration.

Treatment of stormwater, CSO discharges, animal wastes, and leachates needs further consideration.

CHAPTER 9

CONCLUSIONS

1. The subsurface flow (SF) constructed wetland concept can offer high performance levels for BOD_5 and TSS at relatively low costs for construction and operation and maintenance. It is particularly well suited for small to moderate sized installations where suitable land and media are available at a reasonable. cost.

2. The odor and vector control offered by the SF concept make it attractive for systems which are in close proximity to the public. These uses range from single family dwellings to larger developments and public facilities.

3. The cost effectiveness of SF wetland systems as compared to free water surface (FWS) wetlands for the same water quality goals will depend on the local availability of land and the cost for land and for the media used in the SF concept.

4. Ammonia removal in most of the present generation of operating SF systems is deficient. The reason is believed to be the lack of oxygen in the bed profile and a too brief HRT to complete the nitrification reactions.

5. Effective ammonia removal has been reliably established in a few SF wetland systems. The common elements in those systems are full penetration of the plant roots and an HRT > 3 d.

6. Removal of BOD_5 and TSS is not related to the aspect ratio (L:W) of the system.

7. Surface flow has been observed at a number of operational SF systems. This is believed to be largely due to inadequate hydraulic design of the systems and not to clogging of the pore spaces in the media. The water level in the bed can be effectively controlled with adjustable outlet structures.

8. It is likely that some oxygen is available from the plant roots to support nitrification reactions. Effective use of that oxygen source requires complete development of the root zone in the bed profile and sufficient detention time. Neither condition is present in most operational SF systems. Further research is necessary to optimize these relationships.

9. Methods appear to be available to induce and maintain root penetration in order to enhance this oxygen source for nitrification. About a six-day HRT would be necessary for significant nitrification with a fully developed root zone and warm weather conditions. This approach cannot be used as a retrofit for most existing systems since there is not enough area available to increase the HRT to 6 d.

10. Use of a recirculating nitrification filter in combination with the SF wetland bed seems to 'offer promise for successful ammonia control and continued high levels of BOD_5 and TSS removal. This combination may be more cost-effective than an FWS wetland designed for the same performance level.

11. The removal of BOD_5 in SF wetlands shows a linear relationship to the BOD_5 mass loading up to levels of at least 140 kg/ha/d (125 Ib/ac/d).

12. A first order plug flow kinetic model provides a reasonably accurate estimate of BOD_5 removal performance, and is recommended for use as an interim approach until more sophisticated models can be developed and validated.

13. Darcy's Law provides a reasonable approximation of the hydraulic performance in these SF systems as long as the limitations are recognized and accommodated.

14. The hydraulic conductivity (ks) and porosity (n) of the media to be used in these systems should be tested in the field or laboratory prior to final design.

15. To provide an adequate safety factor, no more than one-third of the measured "effective" hydraulic conductivity, and no more than 10 percent of the maximum potential hydraulic gradient should be used for the hydraulic design of these systems. These constraints will tend to limit the aspect ratio to < 3:1 for beds 0.6 m deep.

16. SF systems of all sizes should include a final adjustable outlet to permit control of the water level in the bed.

17. Larger systems (Q = > 5,000 gpd) should consider the use of multiple wetland cells in parallel to improve control and flexibility in operations.

18. The limited data available on removal of fecal coliforms indicate that these systems are capable of about a one- or two-log reduction with sufficient HRT. In most cases that will not be sufficient to reach the commonly applied limit of 200/100 ml, so some form of final disinfection may be necessary.

19. Some form of preliminary treatment, at least to the primary level, is typically used for SF wetland systems in both the U.S. and Europe. Primary treatment using septic or Imhoff tanks is suitable for small to moderate sized systems. Many existing SF systems follow facultative lagoons since they were typically added as a polishing step. Facultative lagoons are an acceptable form of preliminary treatment but can add large concentrations of algae. In these cases a variable level draw-off in the, lagoon may help reduce the algal load on the wetland component.

CHAPTER 10

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Site Details for the Systems Listed in Table 2

(Water Quality Data Are Average Values *Over* the Period of Record at Each Site)

GREENLEAVES SUBDIVISION (Mandeville, LA)

Flow: 0.149 mgd, Area: 1.1 ac, L = 457 ft, W = 105 ft, L:W = 4.4:1
bed depth = 2 ft, HRT = 1 d, Preliminary treatment: 2 cell lagoon, floating aerators in first cell
Wastewater type: domestic, plant type: bulrush.

	Input mg/L	output mg/L
BOD_5	36	12
TSS	42	1 0

DEGUSSA CORPORATION (Mobile, Al)

Flow: 1.78 mgd, Area 2.2 ac, L = 475 ft, W = 28 ft, L:W = 17:1
bed depth = 2 ft, HRT = 1 d, Preliminary treatment: oxidation ditch
Wastewater type: organic industrial, plant type: bulrush

BOD_5	5	4
TSS	23	4
COD	287	245
TKN	22	18
NH3	4.2	2.3

PHILLIPS HIGH SCHOOL (Bear Creek, AL)

Flow: 0.0155 mgd, Area: 0.502 ac, L = 175 ft, W = 125 ft, L:W = 1.4:1 bed depth = 1 ft, HRT 3.9 d, Preliminary treatment: extended aeration Wastewater type: school/domestic, plant type: cattails.

BOD_5	13	1
TSS_5	60	3
TKN	22	2.6
NH3	10	2
NO_3	26	6
TN	48	9
TP	5	0.23
FC	80,000	10

MONTEREY, VA (Monterey, VA)

Flow: 0.022 mgd, Area: 0.056 ac, L = 74 ft, W = 33 ft, L:W = 2.2:1, bed depth = 3 ft, HRT = 0.9 d, Preliminary treatment: lmhoff tank, Wastewater type: municipal (high I & I), plants: cattails, bulrush.

BOD_5	39	15
TSS	32	7
NH3	9.3	8.7

DENHAM SPRINGS, LA (Denham Springs, LA)

Flow: 1.73 mgd, Area = 15.2 ac, L = 1050 ft, W + 210 ft, L:W = 5:1 bed depth = 2 ft, HRT = 1 d, Preliminary treatment: Facultative lagoon, Wastewater type: municipal, plants: bulrush, Canna lillies.

	Input mg/L	output mg/L
BOD_5	25	10
TSS	48	14
NH_3	0.7	10
FC	52,000	3,800

BENTON, LA (Benton, LA)

Flow: = 0.100 mgd, Area: 1.5 ac, L = 900 ft, W = 58 ft, L:W = 15.5:1
bed depth = 2 ft, HRT = 21 d, Preliminary treatment: Facultative lagoon + recirculation
Wastewater type: municipal, plants: bulrush, canna lillies.

BOD_5	18	6
TSS	57	4
NH3	0.6	2.8

HAUGHTON, LA (Haughton, LA)

Flow: = 0.100 mgd, Area: 1.5 ac, L = 934 ft, W: = 72 ft, L:W = 13:1,
bed depth = 2.5 ft, HRT = 4.5 d, Preliminary treatment: facultative lagoon,
Wastewater type: municipal, plants: bulrush, canna lillies.

BOD_5	12.5	2
TSS	47	14
NH3	1.1	7.2

CARVILLE, LA (US PHS Hospital, Carville, LA)

Flow: 0.1228 mgd, Area: 0.64 ac, L = 528 ft, W = 62 ft, L:W = 8.5:1
bed depth = 2.5 ft, HRT = 1.4 d, Preliminary treatment: 3 cell lagoon, air in 1 st cell
Wastewater type: hospital/domestic, plants: arrowhead.

BOD_5	20	8
TSS	93	17
VSS	65	8
COD	107	44
TKN	8.6	7.1
NH3	4.8	5.1
NO_3	0	0
TP	2.3	2.3

MANDEVILLE, LA (Mandeville, LA)

Flow: 1.224 mgd, Area: 4.56 ac, L = 470 ft, W = 207 ft, L:W = 2:1 bed depth = 2 ft, HRT = 0.7 d, Prelimtreat= 3 cell aerated (Hinde tubing) lagoon, Wastewater type: municipal, plants: bulrush.

BOD_5	4 1	10
TSS	59	7
VSS	39	5
COD	79	53
TKN	5	3
NH3	1.4	2.1
NO_3	4.4	0.8
TP	3	4

BENTON, KY, CELL 3 (Benton, KY)

Flow: 0.1811 mgd, Area: 3.6 ac, L = 1092 ft, W = 144 ft, L:W = 7.6:1 bed depth = 2 ft, HRT = 5 d, Prelim treat.: facultative lagoon, Wastewater type: municipal, plants: bulrush & weeds.

BOD_5	26	9
TSS	56	4
TKN	14.1	9.5
NH3	5.1	7.4
NO_3	0.3	0.4
TN	14.4	9.8
TP	4.4	3.4

HARDIN, KY, *phragmites side* (Hardin, KY)

Flow: 0.0624 mgd, Area = 0.79 ac, L = 475 ft, W = 72 ft, L:W = 6.6:1, bed depth: 2 ft, HRT = 3.3 d, Preliminary treatment: contact stabilization plant, Wastewater type: municipal, plants: reeds.

BOD_5	51	9
TSS	118	17
TKN	20.7	12.1
NH3	10.1	9.9
NO,	0.5	0.3
TN	21.2	12.5
TP	4.9	2.2

HARDIN, KY, *scirpus side* (Hardin, KY)

Flow: 0.0492 mgd, Area = 0.79 ac, L = 475 ft, W = 72 ft, L:W = 6.6:1, bed depth: 2 ft, HRT = 4.2 d, Preliminary treatment: contact stabilization plant, Wastewater type: municipal, plants: bulrush.

	Input mg/L	output mg/L
BOD_5	51	4.1
TSS	118	9.4
TKN	10.7	9.7
NH3	10.1	8.3
NO_3	0.5	0.3
TN	21.2	10
TP	4.9	2.4

UTICA, MS, NORTH SYSTEM (Utica, MS)

Flow: 0.05 mgd, Area: 1.5 ac, L = 280 ft, W = 140 ft, L:W = 2: 1, bed depth = 2.1 ft, HRT = 5 d, Preliminary treatment: Facultative lagoon, Wastewater type: municipal, plants: bulrush & cattails.

BOD,	38	14
TSS	52	23
NH3	6.7	2.9
NO_3	. 0.3	0.2
TP	5.8	2.4
FC	2,308	700

UTICA, MS, SOUTH SYSTEM (Utica, MS)

Flow: 0.11 mgd, Area: 2 ac, L = 315 ft, W = 158 ft, L:W = 2:1, bed depth = 2.1 ft, HRT = 3.7 d, Preliminary treatment: Facultative lagoon, Wastewater type: municipal, plants: bulrush & cattails.

BOD,	31	11
TSS	32	11
NH3	5.6	3.1
NO_3	0.3	0.2
TP	4.3	2.6
FC	1,272	628

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