EPA/600/A-94/167

PERFORMANCE OF GRAVEL BED WETLANDS IN THE UNITED STATES

by

Sherwood C. Reed* and Donald S. Brown**
*Environmental Engineering Consultants
RR 1, Box 572, Norwich, VT 05055
**Risk Reduction Engineering Laboratory
U.S. Environmental Protection Agency
Cincinnati, OH 45268

Contract No. 68-C0-0027

Work Assignment Manager

Donald S. Brown
Water and Hazardous Waste Research Division
Risk Reduction Engineering Laboratory
Cincinnati, OH 45268

RISK REDUCTION ENGINEERING LABORATORY
OFFICE OF RESEARCH AND DEVELOPMENT
U.S. ENVIRONMENTAL PROTECTION AGENCY
CINCINNATI, OH 45268

To Be Presented at the IAWPRC Specialist Conference
"Wetland Systems in Water Pollution Control"
Sydney, Australia, Nov. 30 - Dec. 3, 1992

DISCLAIMER

The work described in this paper has been funded by the United States Environmental Protection Agency through the Agency's Risk Reduction Engineering Laboratory, Cincinnati, Ohio. However, this paper has not been subject to the Agency's review and therefore does not necessarily reflect the views of the Agency, and no official endorsement should be inferred.

PERFORMANCE OF GRAVEL BED WETLANDS IN THE UNITED STATES

by: Sherwood C. Reed, P.E.
Environmental Engineering Consultants
RR 1, Box 572
Norwich, VT 05055 U.S.A.

Donald Brown
US EPA RREL
Cincinnati, OH 45268 U.S.A.

INTRODUCTION

Several hundred gravel bed, or subsurface flow (SF) wetland systems exist in the United States ranging in size from single family dwellings to municipal systems designed for flows up to 11,000 m^3/d. Most of these systems have been constructed in the period 1988 to 1992 without the benefit of a consensus on design, construction or operational procedures. The U.S. Environmental Protection Agency (EPA) commenced a continuing series of studies in 1989 to identify the critical issues and to develop appropriate process criteria for this concept. These efforts have included a detailed survey, on-site visits and performance evaluations, and special data collection and evaluation programs at selected sites. Two of these special studies were completed during the summer of 1991, and two are presently underway. This paper is based on the results of those special studies as well as data from other systems which had available long term performance data.

The focus of the paper is on the capability of these systems to remove biochemical oxygen demand (BOD_5), total suspended solids (TSS), and ammonia nitrogen (NH_3 as N) since these are the major water quality parameters controlled by the regulatory agencies in the U.S. A total of 14 full scale operational systems were used to develop this analysis. These data sources are not identified in the graphical presentations but are listed in Table 1 below. All of the data shown in the graphs are long term averages over the entire period of record at each site.

Table 1. Data Sources for Performance Evaluation

Location	Wastewater Type	Design Flow m^3/d	Treatment Area ha
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	Municipal	380	0.61
Carville, LA	Hospital	465	0.26
Mandeville, LA	Municipal	4633	1.85
Benton, KY	Municipal	685	1.46
Hardin, KY*	Municipal	236	0.32
Hardin, KY$^{\Phi}$	Municipal	186	0.32
Utica, MS$^{\theta}$	Municipal	189	0.61
Utica, MS$^{\Omega}$	Municipal	416	0.81

* *Phragmites bed*, Φ *Scirpus bed*, θ North system, Ω South system

PERFORMANCE EVALUATION

BOD Removal

Input versus output BOD_5 data for the 14 systems listed in Table 1 are shown in Figure 1. All effluent values are well below the typical 20 mg/L effluent standard and this has been achieved regardless of the input concentration (within the range shown). The low values in the lower left corner of the graph illustrate a minor limitation of these systems. These systems will typically export an effluent BOD in the range of 5 mg/L due to decomposition of natural organic materials in the system.

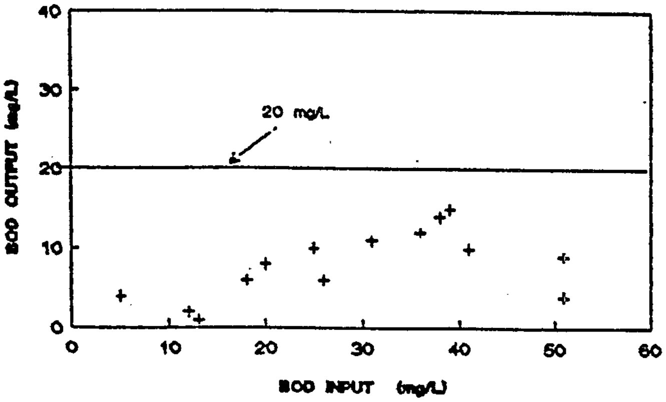

Fig. 1 BOD_5 input vs output

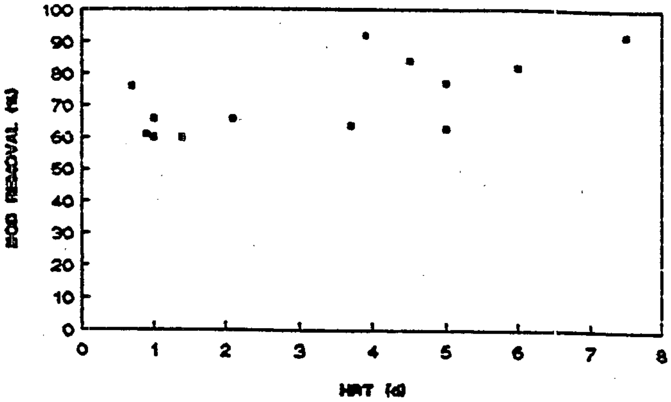

Fig. 2 BOD removal vs HRT

Figure 2 presents the removal of BOD versus the actual hydraulic residence time (HRT) in these systems. The removal of BOD is strongly dependent on HRT up to about 1 d but improves only slightly thereafter, up to an HRT of 7.5 d. The 60 to 65 percent removals at about 1 d HRT are not due to ineffective removal capability but rather to relatively low input levels.

It has been suggested that these SF wetland systems should be constructed with a high aspect ratio (L:W) to insure plug flow conditions and high levels of performance. Figure 3 tests that hypothesis with L:W from less than 2:1 to over 17:1. As demonstrated by the figure there does not appear to be any relationship between aspect ratio and BOD removal. The very low removal associated with the highest L:W in the plot is due to the very low BOD input at this system (< 5 mg/L) and not due to any relationship with aspect ratio.

Figure 4 illustrates the relationship between mass organic loading rates and mass removal rates in SF constructed wetlands. A linear relationship is confirmed by the relatively high r^2 value.

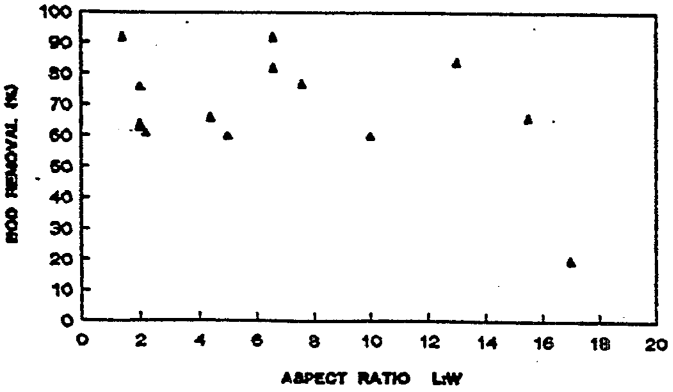

Fig 3. Bod Removal vs Aspect Ratio

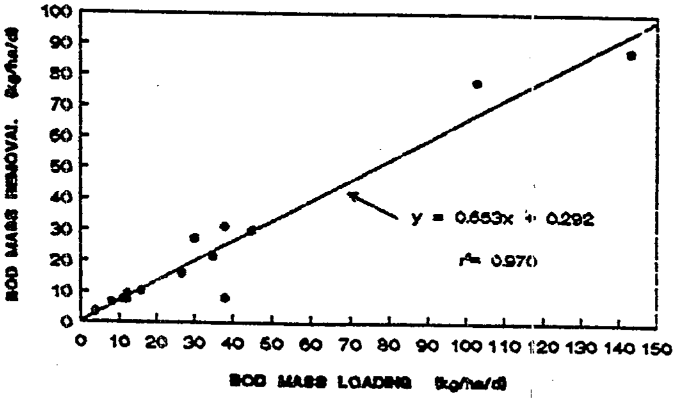

Fig 4. BOD removal vs BOD loading

A first order plug flow model for BOD removal has been suggested for design of these systems in Australia, Europe and the U.S. (Bavor, H.J. et al, 1988, *Reed, 1988*, Boon, A.G. 1985, US EPA, 1988, WPCF, 1990, Conley, L.M., et al., 1991).

A plug flow rate constant was calculated for each of the systems listed in Table 1. These results are plotted versus organic loading on Figure 5. The relationship between the rate constant and the organic loading has an r^2 of 0.95 indicating an excellent correlation.

For comparative purposes, the same relationship for facultative lagoons, as derived by Neel, et al (1961) is also shown on Figure 5. Preliminary work with data from free water surface wetlands (FWS) indicates a similar relationship with the curve falling about midway between facultative lagoons and SF wetlands. An extension of this preliminary analysis indicates that an FWS wetland might be up to 75% larger than an SF wetland for comparable flow and BOD removal goals. The choice between the two concepts may then depend on the availability and cost of land and the cost for the SF media in the local area.

It is believed that the rate constant for SF wetlands is higher than that for FWS wetlands or facultative lagoons because the media in the SF wetland provides more specific surface area and opportunity for retention of the organisms which contribute the biological treatment responses. In open water, continuous flow reactors, it is usually necessary to provide for sludge recycle and a high "sludge age" to obtain comparable reaction rates. The relationship for SF wetlands shown on Figure 5 suggests that these systems can be reliably designed for organic loadings up to at least 100 kg/ha/d. This provides support for use of a rate constant in the neighborhood of 1.0 d^{-1} which has been used in the plug flow models mentioned above.

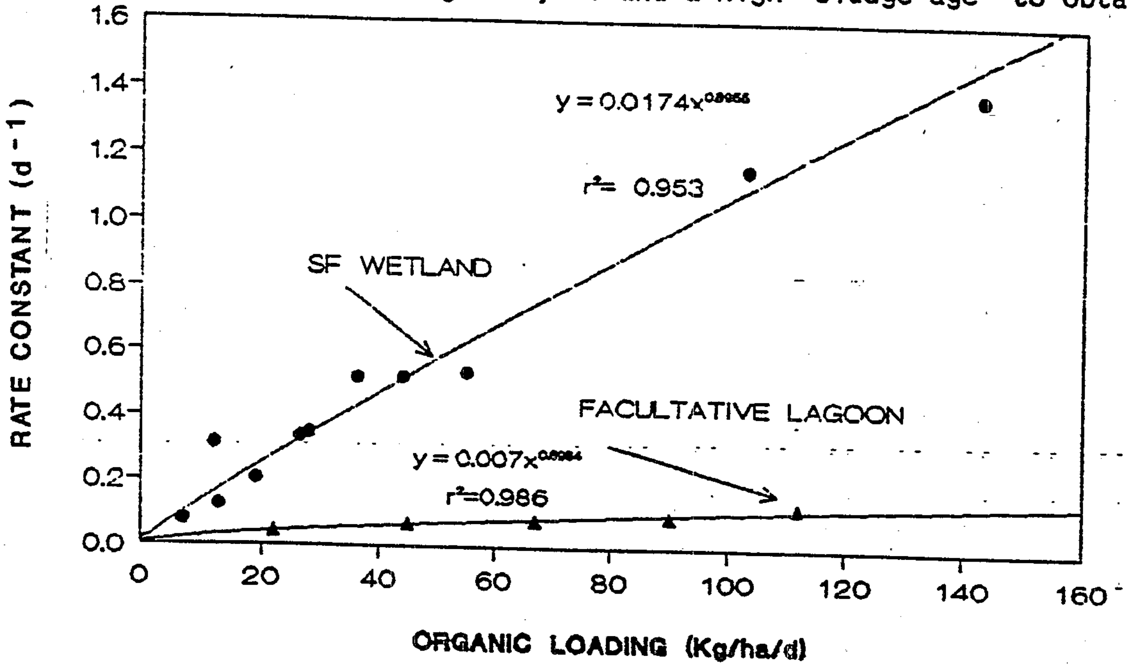

Fig. 5 Plug Flow Rate Constant vs Organic Loading

The relationship shown on Figure 5 also suggests that many of the existing systems are larger than they need to be for effective BOD removal. This has probably occurred because the designers made conservative estimates for input BOD and the actual input BOD has been significantly less.

Figure 6 presents the results of a tracer study, using lithium chloride, conducted in 1990 at the operational SF wetland system in Carville, LA. This is representative of the four such studies conducted under the EPA program. Essentially 100 percent of the tracer was accounted for in the effluent so it can be considered a valid study. The centroid of the curve at 48 hr is identical to the theoretical HRT for the system. It clearly does not exhibit ideal plug flow responses but is similar to curves for lagoons and other reactors designed with plug flow kinetics. The plug flow model seems to give reasonably accurate estimates of system performance. As additional data is obtained the use of alternative models such as a series of continuously stirred reactors may be adopted, but the plug flow model is likely to continue in use for the near term.

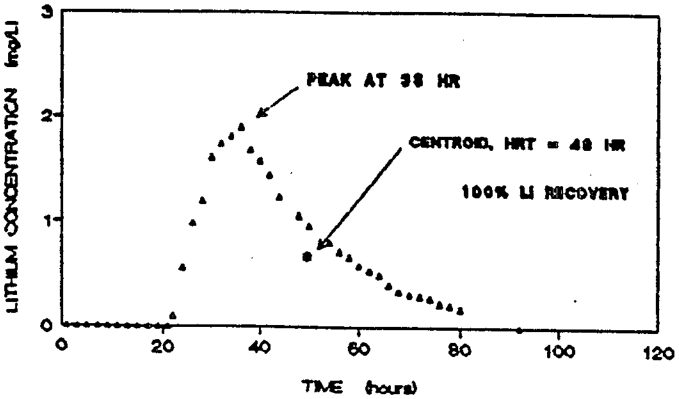

Fig 6. Lithium Tracer Study

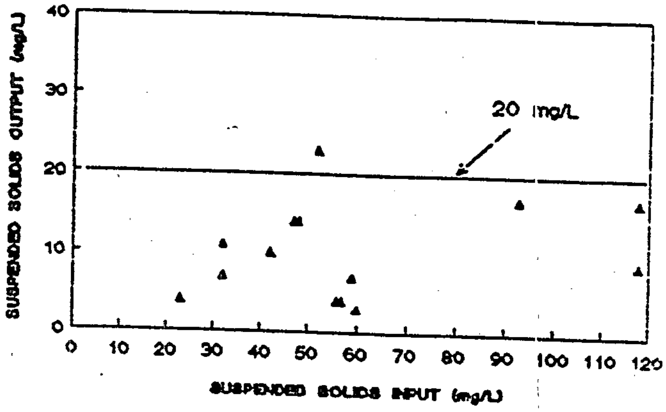

Fig 7. TSS Input vs Output

TSS Removal

Figure 7 compares input to output TSS for the systems studied, with 20 mg/L shown as the typical reference index. Except for one excursion, all of the systems produce a final effluent less than 20 mg/L regardless of the input level (up to 118 mg/L). The systems with high TSS input values (≥ 50 mg/L) have facultative lagoons for preliminary treatment and the high solids are due to algal carry-over from the lagoon. The removal and decomposition of these solids in the SF bed has an impact on the NH_3 status in the system as discussed in the next section of this paper.

The relationship between TSS removal and HRT in the system is similar to the BOD results shown on Figure 2. After about 1 day HRT there is little significant improvement. The relationship between TSS removal and aspect ratio is also similar to the BOD results shown on Figure 3, indicating that there is no correlation between system aspect ratio and TSS removal.

Concerns have been expressed over the potential for gradual clogging of these beds with TSS or dead root/rhizome material. The special EPA studies have excavated pits in four systems in Louisiana ranging in age from 2 to 5 years and minimal clogging has been observed in all cases. In three cases, the solids represent less than two percent of the available void spaces; in the worst case the solids approached six percent of the available void spaces. In all cases, these trapped solids were composed of at least 80 percent inorganic matter. Discussions with the operators and review of the construction histories indicate that these materials may have been delivered to the site during construction either as fines with the media or as soil on the tires of the trucks and other equipment, therefore, further accumulation may be minimal

The surface flow which can be observed on many of these systems has been attributed to clogging of the media. A more likely explanation is provision of an inadequate hydraulic gradient in the hydraulic design of the system. Many of these early systems had L:W ratios approaching 10:1, had a flat bottom on the bed, and had the outlet ports in the effluent manifold near the top of the bed.

Nitrogen Removal

The major concern is the removal of unoxidized ammonia (NH_3) to protect water quality and aquatic life in receiving waters. Figure 8 presents NH_3 (total as N) input versus output for the systems included in this study. The sloping line on the graph is the condition where input equals output. About half of the systems are at or above that line indicating there is a net production and export of NH_3 in these systems. The source of this "extra" NH_3 is believed to be from the anaerobic decomposition of the organic nitrogen trapped in the bed as particulate matter. There is then insufficient oxygen to oxidize this ammonia to nitrate. Support for this hypothesis is shown on Figure 10 which presents ammonia removal versus HRT.

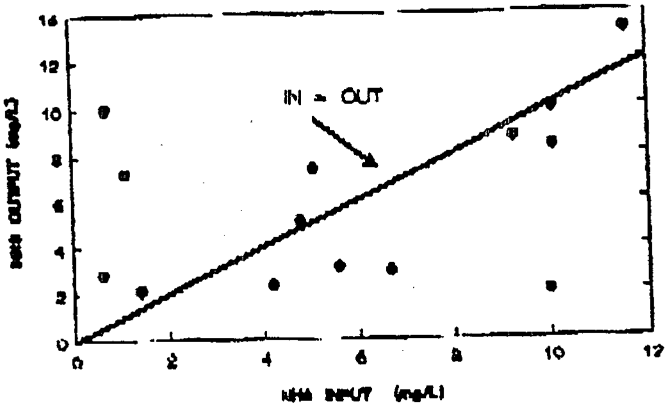

Fig 8. NH_3 Input vs Output

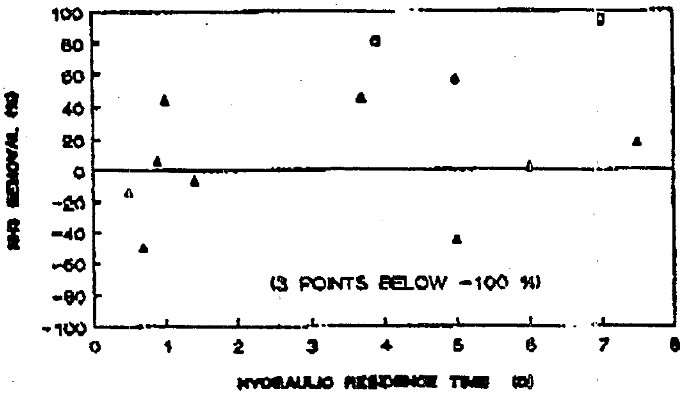

Fig 9. NH_3 Removal vs HRT

Most of the systems shown on Figure 9 display a marginal or a negative ammonia removal rates regardless of detention time. Two of the systems, shown as open squares in the figures display very high removal rates at comparable HRT levels. The difference for these two data sets is that the root zone was fully developed in the bed so that plant available oxygen could support nitrification. In one case (Bear Creek, AL) the bed is only 0.3 m deep and supports a stand of *Typha*, with the roots penetrating to the bottom of the fine gravel bed (Watson, 1990). Ammonia removal at this system 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 bed and ammonia removals of 94 percent were achieved with an HRT of 7d with primary treated wastewater as input (Gersberg, et al, 1985).

Many of the early investigators and designers assumed that the plant roots would always penetrate to their full potential depth so the entire depth of the bed would be an active root zone with at least some oxygen available. The recent EPA studies indicate that root/rhizome penetration is limited to about 0.3 m regardless of the plant species used. Deeper roots can be found near the edges of the bed and other "dead spots" for flow but in general the active root zone is 0.3 m or less in most systems in the U.S. As a result, a significant portion of the flow at these systems never comes in contact with the root zone and has no opportunity to utilize the available oxygen for nitrification. At the Bear Creek and Santee systems shown on Figure 9 all of the flow comes in contact with the root zone and there is sufficient residence time to complete the nitrification reactions. There is no consensus on how much oxygen might be available at the plant roots to support nitrification. Estimates range from zero to about 45 gm $O_2/m^2/d$ (Cooper, et al 1990). Based on the experience at Santee and at the Bear Creek site it seems likely that some plant produced oxygen is available in the root zone, on the surfaces of the root hairs. It can be calculated from the ammonia removal data at these sites that the available oxygen is about 7.5 $gm/m^3/d$ in the active root zone for *Typha, Scirpus*, and *Phragmites*. Assuming a 0.6 m deep active root zone this would translate to 4.5 gm $O_2/m^2/d$ of wetland surficial area, which is near

the low end of available oxygen values in the literature.

Based on the work at Santee and in Europe the maximum potential root zone depths are: *Typha* 0.3 m, *Scirpus* 0.8 m, *Phragmites* 0.6 m. Using these values and the 7.5 $g/m^3/d$ oxygen availability, it is possible to show that nitrification to low levels of ammonia (≤ 2 mg/L) will require at least 5 to 6 days residence time in the system during the warm weather growing season.

Other models are available for predicting ammonia removal (Bavor, 1988, WPCF, 1990) in SF constructed wetlands, but these predict even larger land areas than the previous case. The reason is believed to be that these models were derived from systems which were generally deficient in oxygen and do not reflect the potential nitrification if the root zone is fully developed. These models may adequately describe the performance of the present generation of systems in the U.S. which tend to have short detention times and inadequate root zone development.

However, the costs of the land and the media for the SF bed in a system with six days or more residence time may be prohibitive. As a result, alternative methods are under consideration to supplement the oxygen available from the plants. These include mechanical aeration in open water zones after most of the BOD is removed, utilization of shallow overland flow or grass filtration for nitrification, and the use of cells in parallel operated sequentially on a fill and draw basis to introduce atmospheric oxygen into the bed profile on a frequent daily basis.

The use of unsaturated vertical flow reactors for nitrification such as trickling filters and recirculating sand filters has been common practice in wastewater treatment for a number of years. The use of vertical flow wetland cells has been demonstrated and discussed in the literature (Cooper, et al 1990).

A vertical flow recirculating filter composed of fine gravel has been designed for a constructed wetland system in Kentucky which was having problems meeting the discharge limits for ammonia. Fine pea gravel (0.2 to 0.6 mm size) was selected for use because of the high specific surface area per unit volume ($55\ m^2/m^3$ to $70\ m^2/m^3$) and the high hydraulic conductivity. Based on nitrification experience with other attached growth processes it appears that about 1230 m^2 of this specific surface area is required to oxidize 1 kg of ammonia nitrogen per day with a possible recycle ratio of up to 3:1. The depth of the bed to be used is 0.6 m. The nitrification bed will be superimposed on top of the existing SF constructed wetland, at the head of the cell. Sprinklers will apply the recirculated effluent from the SF cell to the top of the filter. The nitrified percolate is expected to drain through the media and mix with the untreated wastewater in the bed. Denitrification of the nitrate is then expected to occur. This system was under construction in April 1992 and long term data collection is planned, with the intent of optimization of design and operational factors for application elsewhere.

This recirculation component combined with a normal SF bed should need less land area and less media than a system designed for total reliance on plant available oxygen for nitrification. This approach may be the most cost effective method for achieving high levels of nitrogen removal in these constructed wetland systems.

Phosphorus Removal

Phosphorus removal was not consistent in the systems included in this study. In two cases, the phosphorus input equaled the output. In most cases a 40 to 70 percent removal was achieved producing an effluent of 2 to 3 mg/L. In one case (Bear Creek, AL) phosphorus removal to less than 0.1 mg/L was achieved. Flow in this case was through a shallow depth of fine textured gravel and the soil and iron and aluminum oxides associated with the gravel may have provided the adsorption sites needed for phosphorus

removal.

Role of Vegetation

Based on the investigations described in this paper it appears the major contribution from the vegetation in these SF systems is service of the root/rhizome structure as a substrate for microbial activity and as a limited oxygen source for nitrification. This suggests that if the plant is expected to play a major role the depth of the bed should not exceed the potential root development for the plant species selected. It also suggests that a management plan will be necessary to induce and maintain root penetrations below the 0.3 m depth commonly found in most systems in the U.S.

The need for routine harvesting is somewhat controversial in the U.S. Some systems in the southern U.S. conduct a routine annual harvest regardless of the plant species used. It is the author's opinion that such a program is not necessary when plants such as *Scirpus*, *Typha*, and *Phragmites* are used, but may be necessary when soft tissue flowering plants are the dominant species.

Preliminary Treatment Requirements

Some form of preliminary treatment is necessary for these SF wetland systems. The most common form in the U.S. is facultative lagoons since in many cases the wetland component was added to an existing lagoon as a polishing step. Septic tanks and Imhoff tanks are suitable for small to moderately sized systems. Larger size systems may need some form of mechanical preliminary treatment but only to the equivalent of primary effluent.

COSTS

The cost of these systems are presented in other papers at this conference, and elsewhere (Reed, S.C., D. Brown, 1992). The major issue of concern is the relatively high cost to procure and place the media in the SF wetland bed. This one factor may represent 50 to 60 percent of the total construction costs. A cost comparison between SF and FWS wetland systems will depend on the cost of the land and on the cost of the media for the SF concept. Even though the FWS system is likely to require a larger land area it is not clear which concept will be the more cost effective because of these two cost factors.

The SF concept offers other advantages over the FWS alternative in that the subsurface flow provides positive control over odors and insect vectors and lessens public access concerns. These factors are particularly important when systems are to be located adjacent to habitations or at public facilities.

CONCLUSIONS

The SF constructed wetland concept offers high performance levels for BOD and TSS at relatively low costs for construction, and very low costs for operation and maintenance. It is particularly well suited for small to moderate sized communities where suitable land is available.

Ammonia removal in most of the present generation of operating SF systems in the U.S. is deficient. The reason is believed to be the short detention time and the lack of oxygen in the bed profile to support nitrification.

Methods seem to be available to induce and maintain root zone development and the related oxygen source. This will require water level management in the bed utilizing an adjustable outlet mechanism. A system with a fully developed root zone might still require up to six days to produce low levels of ammonia.

The surface flow observed on many systems in the U.S. is believed due to inadequate hydraulic design and not to clogging of the bed. The use of an adjustable outlet mechanism should correct these problems.

A recirculating nitrification filter in combination with an SF wetland seems to offer promise as a cost effective method for achieving effective nitrogen removal in these systems.

The removal of BOD in these systems displays a linear relationship to mass loading up to levels of at least 140 kg/ha/d.

A first order plug flow kinetic model seems to provide a reasonably accurate estimate of BOD removal capability in SF constructed wetlands.

REFERENCES

Bavor, H.J. et al. (1988). *Joint Study on Sewage Treatment Using Shallow Lagoon - Aquatic Plant Systems*, Water Research Laboratory, Hawkesbury Agricultural College, Richmond, NSW, Australia.

Reed, S.C., et al (1988) *Natural Systems for Waste Management & Treatment* McGraw Hill, New York, NY.

Boon, A.G. (1985) *Report of a Visit by Members and Staff of WRC to Germany to Investigate the Root Zone Method for Treatment of Wastewaters*, Water research Center, Stevenage, England.

US EPA, (1988) *Design Manual - Constructed Wetlands and Aquatic Plant Systems for Municipal Wastewater Treatment,*, EPA 625/11-88/022, US EPA CERI, Cincinnati, OH.

WPCF, (1990) *Natural Systems for Wastewater Treatment*, Manual of Practice FD-16, Water Pollution Control Federation, Alexandria, VA.

Conley, L.M., et al. (1991) *An Assessment of the Root Zone Method of Wastewater Treatment*, Jour. WPCF (63)3,239-247.

Neel, J.K., et al (1961). Experimental Lagooning of Raw Sewage, *Jour. Water Pollution Control Fed.*, 33(6)603-641.

Watson, J.T. (1990) *Design and Performance of the Constructed Wetland Wastewater Treatment System at Phillips High School, Bear Creek, AL*, TVA/WR/WQ-90/5, TVA, Chattanooga, TN.

Gersberg, R.M., et al, (1985). Role of Aquatic Plants in Wastewater Treatment by Artificial Wetlands, *Water Research*, 20:363-367.

Cooper, P.F., Findlater, B.C., Editors, (1990) *Constructed Wetlands in Water Pollution Control*, Pergamon Press, New York, NY.

Reed, S.C., D. Brown (1992). *Constructed Wetland Design - The First Generation*, Jour. WEF, (in press).