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SMALL SCALE WASTE MANAGEMENT PROJECT

Treatment Systems Required For Surface Discharge of Onsite Wastewater

by

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TREATMENT SYSTEMS REQUIRED FOR SURFACE DISCHARGE OF ON-SITE WASTEWATER

David K. Sauer[1]

INTRODUCTION

Many on-site wastewater disposal systems for homes in the United States are constructed on soil types that have contributed to the failure of the conventional septic tank-soil absorption system. Many other homes cannot be built because of poor soil conditions for waste disposal. Home construction where good soil conditions exist also leads to the depletion of prime agricultural land. Surface discharge is an alternative disposal method. However, to avoid environmental and health problems, discharge of on-site wastewater to the surface may require extensive treatment and disinfection.

There are numerous alternative schemes which may be considered by the homeowner to achieve surface water discharge requirements. To make a proper decision on these alternatives, the homeowner must examine both the in-house processes (5), as well as treatment options which might best meet the local water quality objectives. A general matrix depicting these choices might be similar to that shown in Table 1.

Many of the options listed above have been examined or are being evaluated by the SSWMP. Among those treatment options considered most effective include the intermittent sand filtration of either septic tank or aerobically treated effluent followed by disinfection. The major emphasis of this paper will be to examine the effectiveness of these two alternatives to meet a water quality objective primarily concerned with low BOD, suspended solids and fecal coliform concentrations.

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TABLE 1. Suggested Processes for On-Site Surface Water Treatment Systems

IN HOUSE PROCESSING	TREATMENT PROCESSES	WATER QUALITY OBJECTIVES
Water Conservation Fixtures	Septic Tank Systems	BOD
Waste Segregation-Grey/Black	Aerobic Processes	Suspended Solids
Household Product Selection	Intermittent Sand Filtration	Pathogens
Appliance Selection	Activated Carbon Filtration	Phosphorus
Waste/Water Recycle-Reuse	Wastewater Lagoons	Nitrogen Species
	Clinoptiolite Ion Exchange	
	Nitrification	
	Denitrification	
	Chemical Precipitation	
	Chlorination, Iodination	
	UV Irradiation, Bromination	

DESCRIPTION OF EXPERIMENTAL STUDIES

Laboratory and field investigations of the intermittent sand filtration process following septic tank or aerobic treatment processes have been underway since 1973. In addition, disinfection studies in the laboratory and field have been undertaken. A brief description of this work is presented below.

Laboratory Experiments

The objective of the laboratory study was to determine the effect of applied wastewater quality, media size and hydraulic loading rate on sand filter effluent quality, length of filter run, filter clogging and crust development (7). Tensiometric measurements were also taken to characterize and possibly predict filter run lengths. Various maintenance techniques, such as resting, raking or removing the crusted sand surface were also studied as methods of rejuvenating clogged sands.

The laboratory experiments were performed at the Park Street Laboratory and involved the application of septic tank effluent and aerobic unit effluent to 24-4 inch (10.2 cm) sand columns. The study has been conducted since June, 1975.. Three types of sand, two hydraulic loading rates and two effluent types were employed in the study. The effective size and uniformity coefficient of the sands are listed in Table 2. The hydraulic loading rates used in the study were 5 gal/day/sq ft (0.2 m/day) and 10 gal/day/sq ft (0.4 m/day). The wastewater applied to the columns was a simulated household waste treated by a septic tank or an aerobic treatment unit (1). Wastewater effluents from these units were monitored and selected parameters are shown in Table 3. Wastewater flow rates through the units were designed to follow the characteristic flow pattern of single households.

TABLE 2. Sands Used for Laboratory Experiments

Effective Size (mm)	Uniformity Coefficient
0.19-0.22	3.3-4.0
0.43-0.45	3.0-3.3
0.65	1.4

TABLE 3. Wastewater Characteristics at the Park Street Laboratory

	BOD_5(mg/ℓ) Mean	TSS(mg/ℓ) Mean
1000 Gallon Septic Tank	53	25-54
Extended Aeration Aerobic Treatment Unit	20-34	50-85

The sand columns contained 30 inches (76.2 cm) of sand underlain by 6 inches (15.2 cm) of pea gravel and 6 inches (15.2 cm) of coarse gravel. A freeboard space of 18 inches (45.7 cm) existed above the sand to allow intermittent ponding of wastewater above the sand. The columns were dosed on the average 6 times per day.

Field Experiments

The objective of the field study was to determine whether household wastewater applied at hydraulic loading rates of 2 to 10 gal/day/sq ft (0.08-0.4 m/day) could be adequately treated by sand filters and to determine the length of filter runs. Various types of maintenance, such as raking the sand surface, removing the top layer of sand or resting the filter bed for some period of time were also investigated. Disinfection of the sand filter effluents via dry-feed chlorination and ultra-violet light irradiation was also investigated in the field experiments.

The field experiments have been conducted since September, 1973 (3). Three systems have been constructed at three rural homes located on University of Wisconsin experimental farms. The system at the Ashland Experimental Farm employed the sand filtration of septic tank effluent while the system at the Electric Research Farm involved the sand filtration of an aerobic unit effluent. Disinfection of the sand filter effluent at both of these sites used gravity flow through dry-feed chlorinators. The system at the Arlington Experimental Farm included the sand filtration of both septic tank effluent and of aerobic unit effluent. Disinfection of the sand filter effluents was performed by an ultraviolet light irradiation unit.

All field sand filters ranged from 14 to 16 square feet (1.3-1.5 m^2) in area and contained 24 to 30 inches (61.0-76.2 cm) of washed sand. The sand was underlain by 6 inches (15.2 cm) of pea gravel and 6 inches (15.2 cm) of coarse gravel. The effective size and uniformity coefficient of the sand used at each site are listed in Table 4. It is important to note that the sands used at the Ashland Experimental Farm and the Electric Research Farm were also tested in the laboratory column studies. The sand was washed pit run sand which was locally available and relatively inexpensive.

TABLE 4. Sands Used for Field Experiments

	Effective Size (mm)	Uniformity Coefficient
Ashland Experimental Farm	0.43-0.45	3.0-3.3
Electric Research Farm	0.19-0.22	3.3-4.0
Arlington Experimental Farm	0.28	2.8

The sand filters were enclosed in concrete block basins and were placed below ground level to prevent freezing problems. The top 4 inches (10.2 cm) of the basins were above ground level to allow an insulated and removable cover to be fastened to the top of the filter. The covers prevented the accumulation of debris on the sand surface, reduced odor, eliminated freezing problems and allowed easy access to the sand surface. An open space of approximately 16 inches (40.7 cm) existed above the sand surface to allow intermittent ponding of wastewater above the sand.

The distribution system for the sand filters consisted of a 2 inch (5.1 cm) plastic pipe with an up-turned elbow located in the center of the bed. A splash plate was placed underneath the outlet elbow to reduce erosion of the sand surface. The collection system at the filter bottom consisted of a 4 inch (10.2 cm) perforated pipe which was vented above the sand surface. Further description of the sand filters can be found in the literature (2).

The hydraulic loading rates employed for the sand filter studies ranged from 2-40 gal/day/sq ft (0.08-1.6 m/day) with the rates primarily between 2-10 gal/day/sq ft (0.08-0.4 m/day. Excessive groundwater infiltration sometimes caused the rates to become exceedingly high. The sand filters were dosed (4 to 13 times/day) by a submersible pump with a controlled volume of wastewater. This was dependent upon the amount of wastewater generated each day.

RESULTS AND DESIGN RECOMMENDATIONS

<u>Septic Tank-Intermittent Sand Filter</u>

Sand Clogging and Maintenance

Throughout the field and laboratory studies a record was maintained of the infiltration rate of sands loaded with septic tank effluent.

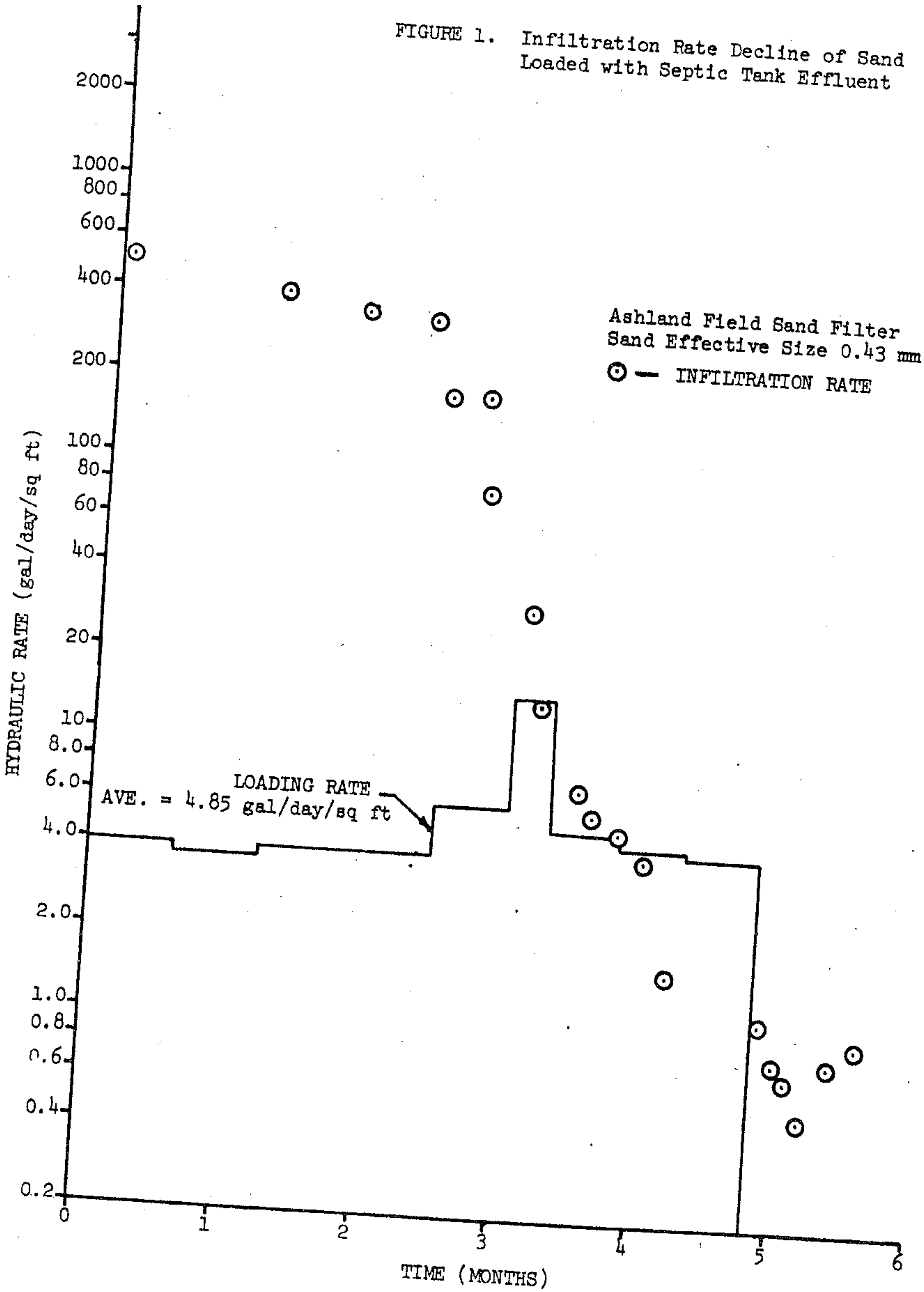
Since there were both field and laboratory studies, many curves were generated representing the decline of infiltration rate over a given time period. A detailed discussion concerning this subject will be presented in a forthcoming publication. In this paper, a general discussion highlighting significant findings will be presented.

Figure 1 shows the infiltration rate decline of a sand loaded with septic tank effluent. The data in this figure was generated at the Ashland field sand filter. The sand had an effective size of 0.43 mm and an initial saturated hydraulic conductivity of 565 gal/day/sq ft (23 m/day). The average hydraulic loading rate was 4.85 gal/day/sq ft (0.2 m/day). This figure is shown because it represents the characteristic infiltration rate reduction that was found in both the field and laboratory studies.

The length of this filter run was approximately 150 days; however, it was found from other filter runs and the laboratory sand columns that run lengths varied depending upon the loading rate, the strength of the wastewater, maintenance performed to the sand surface and the sand size. An important finding was that the pattern of infiltration rate reduction was similar in all of the studies. As shown in Figure 1, a logarithmic decline in infiltration rate occurred leading to continuous ponding of the sand and an ultimate infiltration rate between 0.5 and 1.0 gal/day/sq ft (0.02-0.04 m/day).

An initial time period at the start-up of the sand filters did not show this logarithmic infiltration rate decline. The length of this time period largely influenced the length of the filter runs. Larger sand size, lower hydraulic loading rates and lower wastewater organic strengths increased filter run lengths.

FIGURE 1. Infiltration Rate Decline of Sand Loaded with Septic Tank Effluent



Although the true mechanisms involving the purification of septic tank effluent using sand filters have not been delineated, it is speculated that during the initial start up period a large increase occurs in the microbial activity within the sand. Products of microbial growth, such as cells, slimes and polysaccharides accumulate throughout the sand bed. This growth is especially high in the top 2-4 inches (5.0-10.2 cm) of sand. Eventually these growth products reduce the volume of the sand pores resulting in the logarithmic decline in the infiltration rate.

The ultimate infiltration rates of 0.5 to 1.0 gal/day sq ft (0.02-0.04 m/day) were found in nearly all the experimental studies. To regenerate the clogged sand beds, various maintenance techniques were tested. Results showed that a physical breakup of the crust which forms in the top 2 to 4 inches (5.0-10.2 cm) of sand along with a resting period was required to adequately restore the infiltration capacity of the sand. The breakup of the crust was performed by raking the top 2 to 4 inches (5.0-10.2 cm) of sand or by replacement of the crust with clean sand. The resting period was essential to allow the lower portion of the sand bed to aerate and regenerate.

Based on results from both laboratory and field studies with septic tank-sand filter systems, a hydraulic loading rate of 5 gal/day/sq ft (0.2 m/day) is recommended for determination of the required surface area of the sand filter. Due to the clogging effect of septic tank effluent it is recommended that an additional sand filter of equal size be installed. Application of effluent onto the sand filters would alternate between the two beds. Time periods of loading and resting has been found to be dependent upon the effective size of the sand. These values are listed in Table 5.

TABLE 5. Septic Tank-Sand Filter Operation Schedule

Sand Effective Size (mm)	Sand Uniformity Coefficient	Loading and Resting Period (months)
0.2	3-4	1
0.4	3	3
0.6	1.4	5

For example, if a sand with an effective size of 0.4 mm and uniformity coefficient ≈ 3 is used, the operation and maintenance schedule would be as the following. The entire wastewater load is applied to the first sand filter for 3 months. The flow is then switched to the second sand filter and the first filter is raked to a depth of 2 to 4 inches (5.1-10.2 cm). Wastewater is reapplied to the first filter after 3 months of rest. After the second 3 month loading of the filter the top 4 inches (10.2 cm) of the sand is replaced with clean sand. The first filter then rests for 3 months while the second filter is in operation. Total maintenance to each filter involves raking the sand surface and removing the top sand crust once each year.

Effluent Quality

A detailed analysis of the effluent quality from field sand filters loaded with septic tank effluent at an average rate of 5 gal/day/sq ft (0.2 m/day) has been published previously (3). More recent data has been similar, so only selected parameters are presented in Table 6.

TABLE 6. Septic Tank-Sand Filter Effluent Quality Data

	Septic Tank Effluent	Sand Filter Effluent	Chlorinated Effluent
BOD_5 (mg/ℓ)	123	9	3
TSS (mg/ℓ)	48	6 - 9	6
Ammonia -N (mg/ℓ)	19.2	0.8 - 1.1	1.6
Nitrate -N (mg/ℓ)	0.3	19.6 - 20.4	18.9
Orthophosphate (mg/ℓ)	8.7	6.7 - 7.1	7.9
Fecal Coliforms (#/100 ml)	5.9×10^5	$(0.5 - 0.8) \times 10^3$	2
Total Coliforms (#/100 ml)	9.0×10^5	1.3×10^3	3

NOTE: Loading rate average: 5 gal/day/sq ft (0.2 m/day). Numbers listed are mean values.

Concentrations of BOD_5 and TSS were significantly reduced by the sand filters. Almost complete nitrification of the septic tank effluent was also achieved by the sand filters. Orthophosphate concentrations were reduced approximately 20%. One to two log reductions of total and fecal coliforms were obtained through the sand filters; however, effluent concentrations will not meet current surface discharge recommendations of 1000/100 ml and 200/100 ml respectively. Chlorination of the sand filter effluents reduced the coliform levels below the recommendations.

Aerobic Unit-Intermittent Sand Filter

Sand Clogging and Maintenance

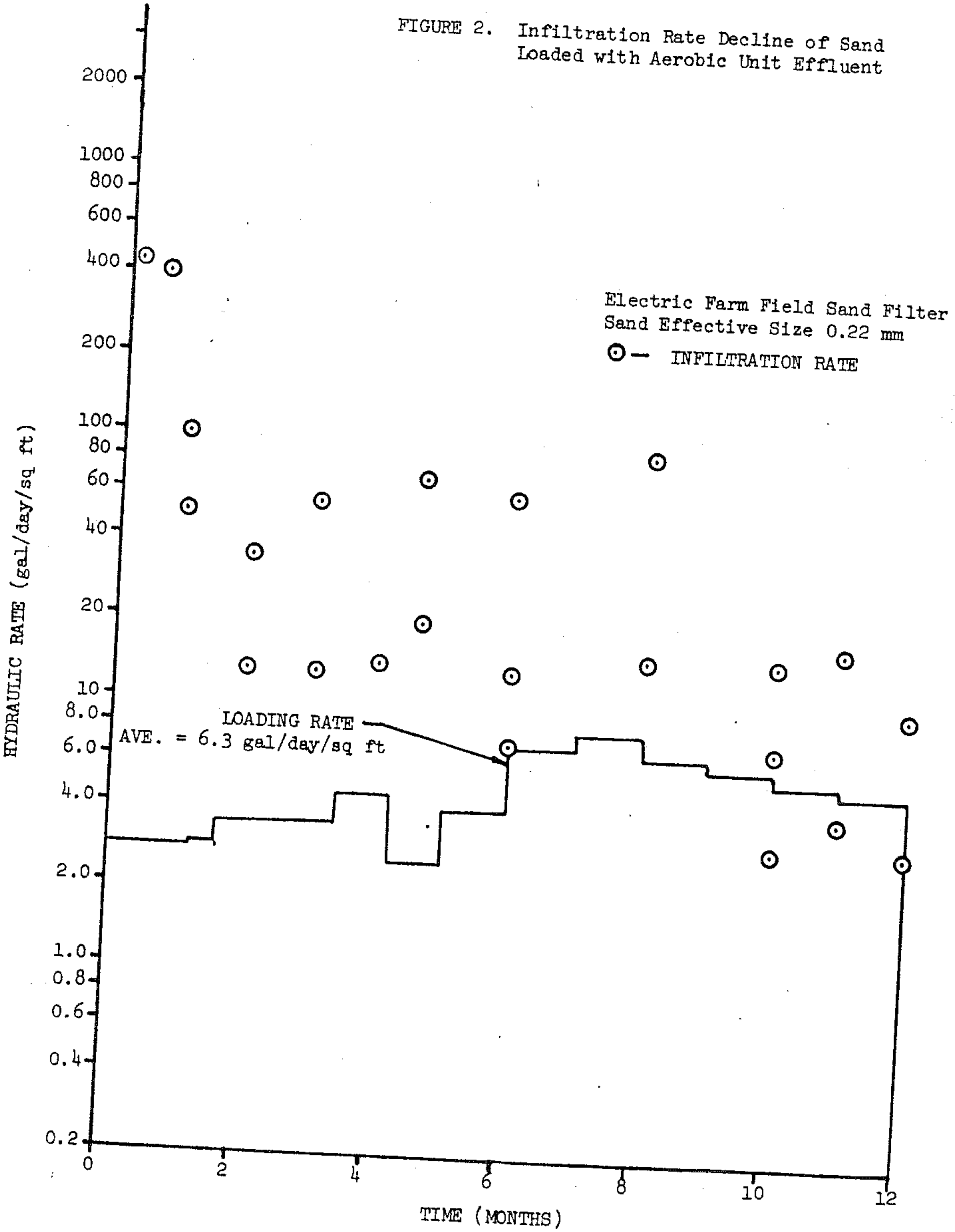
A record of the infiltration rate of sands loaded with aerobic unit effluent was also maintained for the field and laboratory studies. Numerous curves were generated and a detailed discussion on the subject will be presented in a forthcoming publication. A general discussion covering the major findings will be presented here.

Figure 2 shows the decline in infiltration rate with time when an aerobic unit effluent is applied to a sand. The data in this figure was generated at the Electric Farm field sand filter. The sand effective size was 0.22 mm and the initial saturated hydraulic conductivity was 418 gal/day/sq ft (17 m/day). The average hydraulic loading rate was 6.3 gal/day/sq ft (0.25 m/day). The infiltration rate reduction in this figure represents the general pattern found in both the laboratory and field studies.

As shown in Figure 2, an immediate logarithmic decline in infiltration rate occurred during the first month of application. This was due to an accumulation of suspended solids, which were strained from the wastewater. This mat of accumulated solids formed on top of the sand surface and did not penetrate into the sand. The formation of the solids mat continued throughout the filter run indicating a non-degradable nature of the suspended solids in the effluent of the aerobic treatment unit. Flow rates through the crust ranged form 50 to 100 gal/day/sq ft (2-4 m/day) at this time.

During the remaining filter run, infiltration rates ranged from 3 to 100 gal/day/sq ft (0.12-4 m/day). The range of infiltration rates was dependent upon the amount of time the crusted material on top of the sand remained unponded with wastewater. When continuous ponding occurred, infiltration rates decreased to as low as 3 gal/day/sq ft (0.12 m/day); however, when the crust was allowed to dry and crack open, infiltration rates were as high as 100 gal/day/sq ft (4 m/day). Eventually, continuous ponding predominated causing infiltration rates to lower below the 5 to 6 gal/day/sq ft (0.20-0.24 m/day) loading rate. When wastewater ponding above the sand reached 12 inches (30.5 cm), the filter run was ended.

FIGURE 2. Infiltration Rate Decline of Sand Loaded with Aerobic Unit Effluent



At this time, an accumulation of suspended solids on top of the sand had become 3/4 to 1-1/2 inches (1.9-3.8 cm) in depth. It is important to note that even at failure, the ultimate infiltration rate was $\geq$ 3.0 gal/day/sq ft (0.12 m/day).

Sand columns studied in the laboratory also showed an infiltration rate reduction as shown in Figure 2. The total run length, however, was shorter due to a higher suspended solids concentration and higher hydraulic loading rates. Larger sand sizes offered little increase in total run lengths; however, they appeared to have a higher ultimate infiltration rate at the end of the filter run. Ultimate infiltration rates in the laboratory ranged from 3.2 to 9.4 gal/day/sq ft (0.13-0.38 m/day).

Various maintenance techniques were studied to regenerate the clogged sand filters and columns. Results showed that removal of the solids mat from the top of the sand surface restored the infiltrative capacity to 50-100 gal/day/sq ft (2-4 m/day). This capacity was approximately equal to the sand capacity after the initial logarithmic decline in infiltration rate at start-up. This allowed succeeding filter runs to be run without the need for resting the sand bed, thus eliminating the need for alternate filters. Apparently little active biological decomposition occurs below the sand surface, due to the low soluble organic material in the aerobic unit effluent. This statement is only true when the sand surface remains aerobic and at a high oxidation reduction potential. It also assumes that the aerobic treatment unit is properly operated and maintained.

The length of filter runs appeared to be dependent upon the mass of suspended solids applied to the sand surface. The mass is determined by the hydraulic loading rate and the concentration of suspended solids in the aerobic unit effluent. At loading rates ranging from 4-6 gal/day/sq ft (0.16-0.24 m/day) and suspended solids concentration ≃ 50 mg/ℓ, filter run lengths of one year have been experienced. At the same loading rate and suspended solids concentrations ≃ 100 mg/ℓ filter run lengths of six months have been experienced.

Based on these findings a hydraulic loading rate of 5 gal/day/sq ft (0.2 m/day) is recommended for determination of the required surface area of the sand filter. Maintenance to the sand filter is recommended at six month intervals and involves the removal of the solids mat from the top of the sand surface along with 1 inch of sand. After replacing 1 inch of top sand, wastewater may be reapplied to the filter. No alternate filter appears to be necessary, if the aerobic treatment unit is properly operated and maintained. Periodic biological and hydraulic upsets can be assimilated by the sand filters; however, extended time periods of these upsets have led to shorter filter runs.

Effluent Quality

A detailed analysis of the effluent quality from sand filters loaded with aerobic unit effluent at an average rate of 3.8 gal/day/sq ft (0.15 m/day) has been published previously (3). A few selected parameters are presented in Table 7. Further data from sand filters loaded at an average rate of 6.3 gal/day/sq ft (0.25 m/day) has shown similar findings.

An important finding to note is that there is little difference in the effluent quality of the aerobic unit-sand filter effluent and the septic tank-sand filter effluent listed in Table 6. This is especially true after chlorination of the sand filter effluents, which reduced the coliform levels below recommended standards for primary contact recreational waters.

TABLE 7. Aerobic Unit-Sand Filter Effluent Quality Data

	Aerobic Unit Effluent	Sand Filter Effluent	Chlorinated Effluent
BOD_5 (mg/ℓ)	26	2 - 4	4
TSS (mg/ℓ)	48	9 - 11	7
Ammonia -N (mg/ℓ)	0.4	0.3	0.4
Nitrate -N (mg/ℓ)	33.8	36.8	37.6
Orthophosphate (mg/ℓ)	28.1	22.6	23.4
Fecal Coliforms (#/100 ml)	1.9×10^4	1.3×10^3	8
Total Coliforms (#/100 ml)	1.5×10^5	1.3×10^4	35

NOTE: Loading rate average: 3.8 gal/day/sq ft (0.15 m/day). Numbers listed are mean values.

Disinfection Alternatives

Dry Feed Chlorinators

The evaluation of commercially available dry feed chlorinators was performed at two home sites located on University experimental farms. Experiments involved the chlorination of sand filter effluent over an 18 month time period. The sand filtered effluent was collected and flowed by gravity through a dry feed chlorinator and finally into a chlorine contact chamber. The filtered effluent dissolved the tablets in the chlorinator as it trickled past the tablets, while the contact chamber provided detention times ranging from 3-21 hours (4).

Residual chlorine concentrations are equally important in the evaluation of chlorinator performance. A range of residual chlorine concentrations of 0.1 to 1.0 mg/ℓ were measured after 4.5 to 18 hours of contact time. Since residual chlorine concentrations > 0.05 mg/ℓ can be toxic to aquatic plant and animal life (8), acceptable discharge of this effluent will be dependent upon the dilution effects of the receiving water.

A hypochlorite tablet uptake rate of 0.6 to 0.9 tablets/1000 gal. (3.8 m^3) was measured for the aerobic unit-sand filter effluent at a flow rate of 100-150 gal/day (0.38-0.56 m^3/day). For the septic tank-sand filter effluent at a flow rate of 280 gal/day (1.06 m^3/day), a rate of 1.0 tablet/1000 gal. (3.8 m^3) was found. Using these uptake rates, an approximate cost of chemical for chlorination was determined. For flow rates of 100 to 280 gal/day (0.38-1.06 m^3/day) the cost would range from $15 to $54 per year.

A major problem associated with the use of dry feed chlorinators was the lack of control of the hypochlorite dose to the wastewater. This was due to the periodic improper dissolving of the hypochlorite tablets. Various operation and maintenance techniques have been suggested to correct this problem (4).

Ultraviolet Light Irradiation

Initial studies on the use of ultraviolet irradiation as a disinfection method of sand filter effluent have also been performed. A commercially available U.V. water purifier was installed and is being tested at a home site located on a University experimental farm. Experiments have examined the U.V. irradiation of aerobic unit-sand filter effluent and of septic tank-sand filter effluent. The capacity of the U.V. water purifier was specified at 4-16 gal/min; however, initial tests were performed at a rate of 4 gal/min based on previous laboratory studies. The unit was also equipped with automatic wipers, which periodically clean the quartz jacket surrounding the U.V. lamp. In an attempt to arrive at a simple and practical unit, the cleaning system was not used in the study.

The U.V. lamp was operated continuously; however, sand filter effluent was pumped through the unit on an intermittent batch basis dependent upon the flow rates from the sand filters. A summary of four months of operating data are presented in Table 8. During the initial two months, U.V. irradiation of aerobic unit-sand filter effluent was performed while in the latter two months U.V. irradiation of septic tank-sand filter effluent was performed. Results from these tests show that no detectable numbers of fecal or total coliform per 100 ml of U.V. effluent were present.

It should also be noted that cleaning of the quartz jacket surrounding the U.V. lamp has not been required in four months of operation. Power requirements to operate the lamp continuously have been approximately 1.5 kwhr/day. At 4¢/kwhr, this represents an operation cost of approximately $22/yr.

A large drawback in the use of U.V. irradiation as a method of disinfection for small on-site wastewater systems is the large initial capital investment. From the initial results of this study, it appears that a much smaller and simpler U.V. unit could be built to adequately disinfect sand filter effluent. By doing this, initial capital costs could be reduced considerably, thus making the process more economically attractive.

Cost Analysis

The cost-effectiveness of any on-site wastewater treatment and disposal system is an important consideration. For the surface discharge disposal system proposed here, the costs are largely dependent upon the amount of wastewater to be treated, the availability of quality filter sand and the amount of maintenance required by the system. A cost

TABLE 8. Coliform Analysis of Sand Filter and U.V. Water Purifier Unit Effluent

	Aerobic Unit-Sand Filter Effluent Loading Rate = 2.0 gal/day/sq ft (0.08 m/day)	U.V. Effluent	Septic Tank-Sand Filter Effluent Loading Rate = 3.2 gal/day/sq ft (0.13 m/day)	U.V. Effluent
Fecal Coliform (#/100 ml) Mean	11 - 13	0	$(2.6-4.4) \times 10^3$	0
Total Coliform (#/100 ml)	64 - 75	0	$(3.6-5.1) \times 10^3$	0

analysis involving the application of septic tank effluent and aerobic unit effluent onto sand filters has been performed. A summary of this analysis is presented in Table 9. Assumptions in the analysis include a three bedroom home, a family size of five, wastewater production of 50 gal/cap/day (0.19 m^3/cap/day) and the availability of a sand with effective size ≈ 0.4 mm and uniformity coefficient of ≈ 3.5. It is noted that sampling costs are not included in the cost analysis. Since discharge is to surface waters, state regulatory agencies may require some type of monitoring program.

The cost ranges presented in Table 9 suggest that the two alternatives examined in this paper have similar, albeit high costs, when compared with septic tank-soil absorption fields, or with sewered waste treatment systems for small communities. These costs could likely be reduced if water conservation was practiced and quite possibly if waste segregation were employed. It must be recognized, however, that isolated systems can only be evaluated on a case by case basis and conclusions on cost effectiveness cannot be drawn by examining national averages.

APPLICATION OF SURFACE DISCHARGE TREATMENT SYSTEMS

Individual Home Sites and Community Systems

Although it has been shown technically that a high quality effluent can be produced at individual home sites, numerous regulation, maintenance and institutional problems must be solved before widespread use of such systems occurs (6). Unlike conventional subsurface disposal systems, surface discharge systems must be concerned with a point of final disposal. Whether the disposal point is a stream, lake, ditch, underground drain tile or an open field, potential environmental and health problems must

TABLE 9. Initial Capital Costs and Annual Operation and Maintenance Costs

Unit	Cost, in Dollars
Septic Tank (1000 gal)	
Equipment and Installation Cost	350-450
Maintenance Cost	10/yr
Aerobic Treatment Unit	
Equipment and Installation Cost	1300-2000
Maintenance Cost	35/yr
Operation Cost, 4 kwhr/day at 4¢/kwhr	60/yr
Wet Well Pumping Chamber	
Equipment and Installation Cost	250-350
Operation Cost[1], 1/4 kwhr/day at 4¢/kwhr	4/yr
Sand Filter	
Equipment and Installation Cost	10-15 dollars/sq ft
Maintenance Cost	1 dollar/sq ft/yr
Chlorination and Settling Chamber	
Equipment and Installation Cost	700-1000
Operation Cost[1] (Chemical)	40/yr
Ultraviolet Irradiation Unit	
Equipment and Installation Cost	1100-1500
Operation Cost[1], 1-1/2 kwhr/day at 4¢/kwhr	20/yr
Maintenance Cost, Cleaning and Lamp Replacement	*

[1]Does not include pump replacement

*Undetermined

be considered. These problems are in turn dependent upon the number of systems within a prescribed area. Another problem is to insure the performance of maintenance to the treatment system, which is required for proper operation.

There is, however, a rational approach to the solution of these problems. The first step would involve the installation of a few experimental systems. These systems would be maintained by an installer or other trained personnel who are bonded and under contract with the homeowner. Monitoring and regulation of surface discharge systems should be performed by some regional or local governing agency. Such agencies may include state regional offices, county offices or sanitary districts. These people would insure that the system is operating properly and also that proper maintenance is being performed. Undoubtedly unforeseen management problems will occur during the experimental period. Solutions to these problems can only be determined if experimental systems are installed and tested.

If step number one proves successful, regulatory officials should carefully outline a policy on the future use of surface discharge systems. Due to the relative complexity of such systems, a suggested policy outline includes a basic set of regulations followed by a case by case approval. The basic set of regulations should specify the following conditions and requirements:

1. the type of home where a surface discharge system may be used, i.e., an existing home, a seasonal home, new homes installed with low flow fixtures, etc. (Sand filter systems are ideally suited for seasonal homes or homes installed with low flow fixtures. The periodic use of seasonal homes would reduce the

amount of periodic maintenance to the sand filters. Waste segregation and low flow devices would reduce the probability of hydraulic upsets and lower the nutrient discharge concentrations of nitrogen and phosphorus.);

2. the number of homes having surface discharge systems allowable within a defined region, i.e., a housing complex surrounding a lake or a small town on a trout stream may have to be concerned with nitrogen, phosphorus and bacteriological concentrations; hence, only a limited number of systems may be allowable;
3. the procedure for properly designing the surface discharge system;
4. a contracted agreement between the homeowner and a separate entity to insure the system is properly operated and maintained;
5. the discharge requirements and the amount of monitoring required by the governing agency; and
6. the legal authority of the governing agency when violation of the regulations occurs.

A more detailed and comprehensive discussion concerning regulatory methods for on-site sewerage systems can be found in the literature (6).

In conclusion, this paper has examined only one series of on-site wastewater treatment to achieve surface water quality standards. The impact of in-house waste segregation, water conservation, product and appliance selection and recycle will all be significant. In addition, the need for other treatment processes within the flow sheet to provide for nutrient removal or other effluent polishing will greatly alter the cost effectiveness of the system. Many of these alternatives are currently being evaluated by SSWMP and future reports will be forthcoming.

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