

ON-SITE DISPOSAL
OF
SMALL WASTEWATER FLOWS
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ON-SITE TREATMENT AND DISPOSAL OF SMALL WASTEWATER FLOWS

Small Scale Waste Management Project
University of Wisconsin-Madison/University of Wisconsin-Extension

INTRODUCTION

In 1970 approximately 19.5 million households or nearly 30 percent of all housing units in the United States disposed of their wastewaters by some form of private sewerage facilities (1). This number is growing at an increasing rate, due to an emerging trend of population movement to rural areas where community sewage treatment facilities are not usually available. Retired persons are moving back to rural areas, as well as young families who are following the growth of industries on the outlying fringes of metropolitan centers (2). Most of these rural households utilize septic tank systems to dispose of their wastewater. Because of poor design, construction or maintenance, however, a large number of these systems are failing to provide adequate treatment and disposal of their sewage.

Many households, while located in rural areas, are situated in small communities or subdivisions ranging in size from a few households to a hundred or more. In such instances, failing septic tank systems which allow raw or poorly treated sewage to reach the ground surface, surface body of water or even the groundwater, create a severe public health hazard and nuisance because of the close proximity of homes. Public wastewater facilities are often the only solution to abate the problem.

Assessment of wastewater facility needs of small rural communities is difficult because of the lack of information. The last known published status report is a survey conducted by the U.S. Department of Agriculture in 1962 (3). The results of the survey are presented in Table I. At that time, 92 percent of the communities with populations less than 1000 had no public facilities as compared to 19 percent for all communities with populations above 1000 people.

Since 1962, there have been several governmental programs initiated, namely the Federal Water Pollution Control Act (PL92-500) and various state programs, which attempt to abate water pollution by providing grants in aid of construction for community sewerage facilities.

Consequently, the data presented in Table I needs to be updated. However, these figures do serve to indicate that few small communities have public wastewater facilities.

Table I. Number of communities with and without public sewerage facilities in 1962 (3)

Size of community population	United States	
	With	Without
26-999	3,803	42,837
1000-2499	3,079	1,391
2500-5500	2,027	349
Over 5500	2,926	142
Total	11,835	44,709

Certainly the need for improved facilities exists in many of these communities. The communities often were established long before sound design and installation criteria for septic tank systems were enforced. Some homeowners merely installed a pipe to discharge their wastewater into a ditch or stream away from the house. More conscientious homeowners installed septic tank systems, but without good design criteria and proper maintenance, many of these systems have failed. Nuisance and public health hazards have resulted, often impeding or halting economic development in the area.

Conventional Public Facilities

The traditional method of providing public wastewater facilities is to construct a system of gravity collection sewers which convey all the wastewaters to a single community treatment plant. This "central" system is preferred by governmental authorities, engineers and the public alike for several reasons. First, the gravity sewer system is tried and proven. There is much technical expertise in the theory, design and operation of central sewerage which has led to great confidence in the system. Second, central sewerage is usually more cost-effective because of economies of scale. It is less costly to serve many people with one system rather than each one individually. Third,

central sewerage allows ready application of central (and usually public) management which is responsible for the proper functioning of the system. The availability of a single entity to manage the system is quite desirable from a regulatory authority's viewpoint because the authorities have an entity against whom they can bring administrative or judicial action to abate water pollution problems. Central management is also favored by the homeowner who no longer has the responsibility for his private system.

For smaller communities and subdivisions, however, such a conventional collection and treatment facility is impractical because of the financial burden it places on the residents or developer. This is largely due to the high cost of collecting wastewater from each home or business. Smith and Eilers (4) computed the 1968 national average of total annual costs of municipal wastewater collection and treatment facilities which showed that 65 percent of the total annual cost is for amortization and maintenance of the collection system. A more recent study by Sloggett and Badger (5) of 16 small communities in Oklahoma showed a similar distribution (see Table II). It is clear from this breakdown of the total annual costs that the collection system is the most expensive component of any facility.

Table II. Distribution of total annual costs for municipal wastewater collection and treatment facilities

	Amortization cost		Current expenses			Total
	Collection	Treatment	Operation & maintenance Collection	Overhead Treatment		
Smith & Eilers (1968)(4)	60.3%	15.3%	4.7%	8.4%	11.3%	100.0%
Sloggett & Badger (5)	—	72.6%	—	14.2%	3.2% (lagoons)	10.0%

In small communities, homes are typically scattered, which cause the costs of sewerage to rise dramatically. In their study of 16 wastewater collection and treatment systems, Sloggett and Badger (5) ...

showed that the costs per customer rise as the number and density of customers declines. Construction costs per customer were compared to the density and number of customers served (see Tables III and IV). Both factors were shown to have a significant effect but the density of customers was shown to have the largest impact on per capita construction costs.

Sloggett and Badger (6) made similar comparisons using the total annual costs. They found both number and density of customers to be significant (see Table V). In 1972, average annual costs per customer ranged from \$76.90 to \$43.36 for communities with populations less than 100 and 300-400 respectively. The national average for all municipalities large and small was \$19.80 in 1968 (7).

Table III. Cost of construction per customer relative to density of customers for 16 community wastewater facilities in Oklahoma (5)

	Customers per mile of sewer			
	Under 30	30-39	40-49	Over 50
Number of systems	5	5	1	5
Average cost/customer (1972 dollars)	\$1,100	\$847	\$696	\$575
Average number of customers	96	119	310	256

Table IV. Cost of construction per customer relative to number of customers for 16 community wastewater facilities in Oklahoma (5)

	Number of customers served			
	Under 100	100-199	200-299	300-400
Number of systems	6	4	3	3
Average cost/customer (1972 dollars)	\$1,000	\$798	\$594	\$434
Customers/mile of sewer	28.3	37.8	49.4	55.2

Because of the prohibitive costs of extending sewers, outlying members of the community may not be served. In 30 percent of the communities with public facilities surveyed in 1962 by the U.S. Department of Agriculture (3) at least one-third of the residences were not accommodated. In small communities this number would be much higher. Thus, central sewerage often does not abate the pollution problems as intended.

Table V. Total average annual cost per customer for 16 community wastewater facilities in Oklahoma (6)

No. of customers	Total average annual cost (1972 dollars)		
	0% construction grant	75% construction grant	100% construction grant
Under 100	\$76.90	\$33.06	\$18.44
100-199	57.55	25.39	14.63
200-299	52.10	24.09	14.75
300-400	43.36	20.72	13.17

In smaller communities, where homes tend to be more scattered, the cost of conventional facilities can become prohibitive. Costs can exceed \$10,000 per household for the capital portion alone and may be even higher if treatment beyond secondary is required to meet water quality standards. It is not unusual for the cost of the complete system to approach the total equalized value of the community (8).

To help communities meet the water quality goals of the Federal Water Pollution Control Act Amendments of 1972, the federal government was authorized by a provision in the Act to provide grants in aid of construction for 75% of the grant eligible portions of the wastewater facility. The availability of these grants would help offset the high per capita costs in small communities but, unfortunately, small communities have difficulty in obtaining them.

The federal funds are allocated to the individual states on the basis of need, but each state is given the power to determine how the funds are to be spent. Only minimum requirements are set out by the Act for states to follow in preparing a priority list of projects. For example, the Act requires that consideration be given to the severity of the pollution

problem, the population affected, the need for preservation of high quality waters and national priorities. The federal regulations seem to give the states some discretion by not requiring strict adherence to their rankings of pollution discharges. Thus, the priority lists usually work to the disadvantage of small communities, in that many of them are near the bottom, preceded by communities with larger populations and larger pollution discharges. This emphasis denies small communities any expectation of receiving badly needed funding for public facilities in the near future. It is obvious from this discussion that it is impractical to expect many small communities to construct conventional public wastewater facilities to eliminate failing private systems.

Non-Central Wastewater Facilities for Small Communities

A "non-central" facility of several treatment and disposal systems serving isolated individual residences or clusters of residences may offer a less costly alternative to the conventional central facility in the non-urban setting. As Table II indicates, approximately two-thirds of the total annual cost of a conventional facility is due to the collection system. In a community of scattered homes this proportionate cost could be even higher. If the central treatment plant could be eliminated, long sewer extensions collecting wastes from widely spaced homes would not be necessary. Instead, treatment and disposal could be provided where the wastes are generated. Individual or jointly used septic tank systems or other treatment and disposal methods could be used. Such a non-central facility of disperse systems could result in a substantial savings because of the following advantages (9):

1. Existing functional septic tank-soil absorption systems can be utilized rather than providing new service. Often, homeowners who are not having trouble or who have recently installed new septic tank systems do not wish to support community action that will cost them more money unnecessarily. Incorporating existing systems into the public system minimizes this opposition, as well as reducing the total cost of the public facility.

2. Isolated single homes and clusters of homes can be served individually instead of extending costly sewer lines out to them. This could be equally advantageous to existing communities, as well as newly platted subdivisions. Where future growth is not expected to be great enough to warrant sewer extensions, individual septic tank systems could be used. In cases where substantial growth is expected, such as in newly platted subdivisions, the first few homes built could be served by holding tanks which would be pumped and maintained by the management entity. When the number of homes increased to the point where a

common disposal system is warranted, it could be built on land reserved for that purpose. This would delay construction until the time there are enough contributors available to pay for it.

3. Less costly treatment facilities can usually be constructed. In addition, subsurface disposal can often be employed which requires minimal treatment and avoids the necessity of upgrading the treatment plant to meet changing standards for effluent discharges to surface waters. Where subsurface disposal is not possible, the smaller flows may allow other simple treatment methods to be used. In addition, by limiting the area served the necessary excess capacity required for future growth is accurately known providing a more optimal design.

4. A more cost-effective facility may encourage smaller communities to proceed with construction rather than waiting for federal construction grants. This would speed abatement of water pollution problems. Where financial aids are necessary, a greater number of community facilities could receive construction grants because of the fewer dollars required for each project.

5. More rational planning of community growth is possible. Linear development, which is encouraged by the construction of interceptor sewers used to collect wastes from outlying clusters of homes could be avoided. Growth could be encouraged in the more desirable areas by providing public service in those areas only.

6. Non-central facilities are more ecologically sound since the disperse systems dispose of the wastes over wider areas. Through this practice the environment is able to assimilate the waste discharge more readily, which reduces the need for mechanical treatment and the associated energy consumption.

Though relatively untried, the use of individual or several jointly used on-site treatment and disposal systems does not exclude the use of central management. There are several methods of exerting public (or in some cases private) central management over such facilities. The powers needed by an entity to properly manage a non-central facility are similar to those powers needed to manage a conventional community system.

Powers Needed by a Management Entity Any management entity which endeavors to effectively administer on-site wastewater disposal systems must have the power and authority to perform vital functions (9). The entity should be able to:

1. Own, operate, manage and maintain all wastewater systems

within its jurisdiction. The entity must be empowered to acquire by purchase, gift, grant, lease or rent, both real and personal property. It must also have the authority to plan, design, construct, inspect, operate and maintain all types of on-site systems whether the system is a typical individual septic tank system or a more complex system serving a group of residences. The entity should have at least these "ownership and operation" powers within its boundaries but it should not be limited to providing services only within its boundaries. The entity may be given extra territorial jurisdictional authority to operate, maintain and perhaps own such systems outside of the entities boundaries by state statute, by case law, or as terms under a contract.

2. Enter into contracts, to undertake debt obligations either by borrowing and/or by issuing bonds and to sue and be sued. These powers are more than mere legal niceties because without them the entity would not be able to acquire the property, equipment and supplies and services necessary to construct or operate the individual or jointly used on-site systems.

3. Raise revenue by fixing and collecting user charges and levying special assessments and taxes. The power to tax is limited to various public or quasi-public management entities. In lieu of taxing powers, the non-government management entities must have the authority implied or directly granted to set and charge user fees to cover administrative costs.

4. Plan and control how and at what time wastewater facilities will be extended to those within its jurisdiction.

Though not necessary to provide adequate management of a non-central facility, two additional powers are desirable. These are that the entity be able to:

a. Make rules and regulations regarding the use of on-site systems and provide for their enforcement through express statutory authorization. To promote good public sanitation, the entity should be empowered to require the abatement of malfunctioning systems and to require the replacement of all such systems, all according to the plans of the entity. This power, however, may already be inferred from the statutory authorization to operate a system.

b. Meet the eligibility requirements for both loans and grants in aid of construction from both the federal and state governments. While it is obvious that a management entity can function without being eligible for these loans and grants, the viability of the "non-central" system is strengthened when grant money is used to offset some or

most of the costs to the families served by the entity. This is especially true considering that low-income families typically can ill-afford to finance the entire cost of their sewerage system. Experience has shown that low-income families cannot pay wastewater bills in excess of \$7.00 per month or a combined water-sewage bill of \$14.00 per month (10). This rate is difficult to reach without benefit of public subsidy. The inequity should be especially obvious to most non-rural residents who typically pay considerably less than this amount.

Types of Acceptable Management Entities The types of entities which could manage a non-central facility vary from state to state. The various state constitutions, state statutes, administrative agency rules and regulations must be examined on a state by state basis, to determine which types of entities are authorized to manage on-site systems. In addition, the case laws (essentially interpretations of state laws made by the courts) must be checked to determine if the courts have construed the constitution, statutes or regulations to give or to remove the authority to manage such a system from a possible entity. Those entities which may have the necessary powers include municipalities, counties and townships, special districts, private non-profit corporations, rural electric cooperatives and private profit-making businesses. Each state would have to be checked to see which are permitted.

While there are disadvantages, the potential of non-central facilities seem to warrant further investigation. Many of the possible shortcomings of this alternate facility may vanish as some are constructed and experience gained.

Collection and Treatment Alternatives for Non-Central Facilities Proper facilities planning involves a systematic comparison of all feasible alternative methods of dealing with a wastewater treatment and disposal problem. The purpose of this comparison is to identify the most "cost-effective" solution which will minimize total costs to the community and the environment over time.

The commitment by regulatory agencies and engineers to conventional gravity sewers with a common central treatment plant, however, has eliminated many worthy alternatives from consideration. If this bias can be changed, the utilization of the non-central concept has the potential of significantly reducing the environmental and monetary costs of wastewater facilities in many communities by either reducing the size or eliminating the collection system altogether and by simplifying the treatment facility.

The most extreme non-central system would be one where each

home and other establishment were served by an individual septic tank system. However, the most cost-effective community system would probably lie somewhere between the two extremes of central sewerage and individual systems. Either because of economies of scale or because site conditions are unfavorable for individual disposal systems, joint systems serving several homes may be constructed. The end result may be a mix of several individual and joint systems.

The number of alternative methods that can be considered for dealing with a wastewater treatment and disposal problem are endless. To evaluate which method is most cost-effective for a particular community would seem to be a monumental task. However, this task can be greatly simplified by selecting the proper beginning point from which to design the facility.

The objective which must be met by the wastewater facility is to produce an effluent which will not accumulate harmful pollutants to dangerous levels in the environment. The environment, of course, is part of the treatment system, providing final purification. If the pollutant load in the wastewater received by the environment is too great for the environment's assimilative capacity, the pollutants will accumulate. Therefore, to properly design a wastewater facility it is necessary to first evaluate the physical characteristics of the local environment. These characteristics will dictate the type and degree of treatment required before the wastes are discharged. The receiving environment may either be surface waters, land or the atmosphere. Usually, surface waters are used as the receiving environment because discharging large volumes of water into rivers and streams is an easy matter. However, this practice requires that rather high degrees of treatment be provided prior to discharge to prevent the degradation of the stream. On the other hand, if soil is considered as the disposal medium, lower levels of treatment are required before disposal, because of the soil's greater assimilative capacity. The trade-off is that large areas of land are required for absorption. However, when operation and maintenance costs of high levels of treatment for surface water disposal are compared to land costs for land disposal, land disposal may be a more cost-effective alternative. A similar situation may exist for atmospheric disposal as in evapotranspiration.

Thus, the point of beginning in designing community wastewater facilities is to characterize the local environment. Once it is determined what disposal media are available, then treatment systems can be designed to fit for cost-effective comparisons.

This requires that the capabilities of the receiving environment for waste assimilation be known. Federal and state regulatory agencies have

already set effluent standards for surface waters. However, the assimilative capacities of soil and evapotranspiration systems are poorly understood. Therefore, it is necessary to review these areas.

THE USE OF SOIL AND SOIL MATERIALS FOR TREATMENT AND DISPOSAL OF WASTEWATER

Liquid Movement Into and Through Soil and Soil Materials

Proper performance of on-site wastewater disposal systems depends upon the ability of the soil or a soil material to absorb and purify the wastewater. Failure occurs if either of these functions are not performed. Both are directly related to the hydraulic conductivity characteristics of the soil, which are largely controlled by the pore geometry of the material.

Soil Porosity and Permeability Soil is a complex arrangement of solid particles and air and water filled pores. The size and shape of these pores is a function of the structure or arrangement of the solid particles. In single grained soils such as sands, the voids are simply packing pores that exist between the individual grains. The size and shape of these pores is a function of the texture (particle size distribution) of the soil and the shape and packing of the individual grains. When significant amounts of clay and organic matter are present, soil particles become cemented together and form aggregates or peds. Planar voids will form separating the peds. Tubular channels formed by plants and animals living in the soil and irregularly shaped discontinuous pores called vughs are also found in soils (see Figure 1).

Soil permeability or capability to conduct water is not determined by the soil porosity but rather the size, continuity and tortuosity of the pores. A clayey soil is more porous than a sandy soil, yet the sandy soil will conduct much more water, because it has larger, more continuous pores. These twisting pathways with enlargements, constrictions and discontinuities through which the water moves are constantly being altered as well. The soil structure which helps to maintain the pores is very dynamic and may change greatly from time to time in response to changes in natural conditions, biological activity and the soil-management practices. Repeated wetting, drying and freezing help to form peds while plants with extensive root systems and soil fauna activity promote soil aggregation and channeling. On the other hand mechanical compaction and the addition of soluble salts can cause the breakdown of the peds reducing the capacity of the soil to conduct water.

Characterization of Water in Soils Under natural drained conditions, some pores in soil are filled with water. The distribution of this water

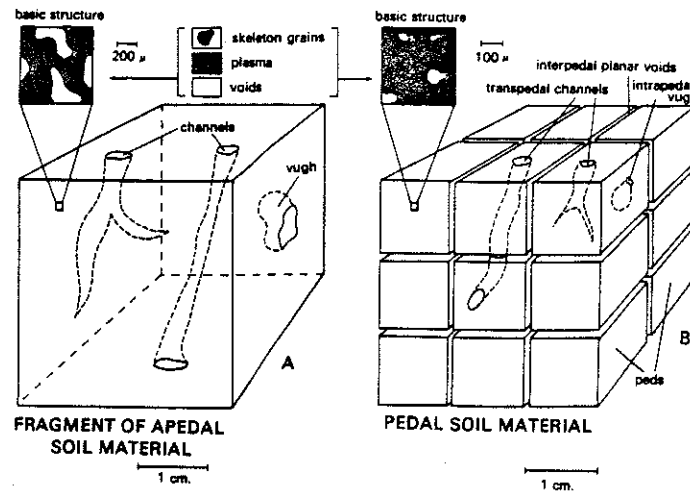


Figure 1. Schematic representation of a single-grained (left) and an aggregated soil material (right) (11)

depends upon the characteristics of the pores while its movement is determined by the relative energy status of the water. Water flows downhill, but more accurately, it flows from points of higher energy to points of lower energy. The energy status is referred to as the moisture potential.

The moisture potential has four components of which the gravitational and the matric potential are the most important. The gravitational potential is the result of the attraction of water toward the center of the earth by a gravitational force and is equal to the weight of the water. To raise water against gravity, work must be done and this work is stored by the water in the form of gravitational potential energy. The potential energy of the water at any point is determined by the elevation of that point relative to some reference level. Thus, the higher the water, the greater its gravitational potential.

The matric potential is produced by the affinity of water to the soil particle surfaces. The pores and surfaces of soil particles hold water due to forces produced by adsorption and surface tension. Individual molecules within the liquid are attracted to other molecules equally in all directions by cohesive forces. Molecules at the surface of

the liquid, however, are attracted more strongly by the liquid than by air. To balance these unequal forces, the surface molecules pull together causing the surface to contract creating surface tension. When solids come in contact with the surface of the liquid, the water molecules are more strongly attracted to the solid than other water molecules and hence the water climbs up the surface of the solid. This is referred to as capillary rise. The upward movement ceases when the weight of the raised water equals the force of attraction between the water and the solid. As the ratio of solid surface area to liquid volume increases, the capillary rise increases. Therefore, water rises higher in smaller pores.

For example, a cylindrical pore radius of 100 microns corresponds with a relatively low capillary rise of 28 cm water (pressure below meniscus equals -28 cm water) while a pore radius of 30 microns results in a relatively high rise of 103 cm (pressure equals -103 cm water) as illustrated in Figure 2. The water within the tube is at less than atmospheric pressure as noted because it is "pulled" downward by gravity as it is being "pulled" upward by the forces of capillarity. The water is under tension as the tube essentially "sucks" the water into it. This negative pressure in soil is called soil tension or soil suction and is measured in millibars (mbar). This implies that it takes more energy to remove or pull water from a small pore than a large one.

In addition to the capillary phenomenon, adsorption forces also contribute to the matric potential. Molecular forces between the surface of the soil particles and the water form envelopes of water over the particle surfaces retaining the water in the soil (Figure 2).

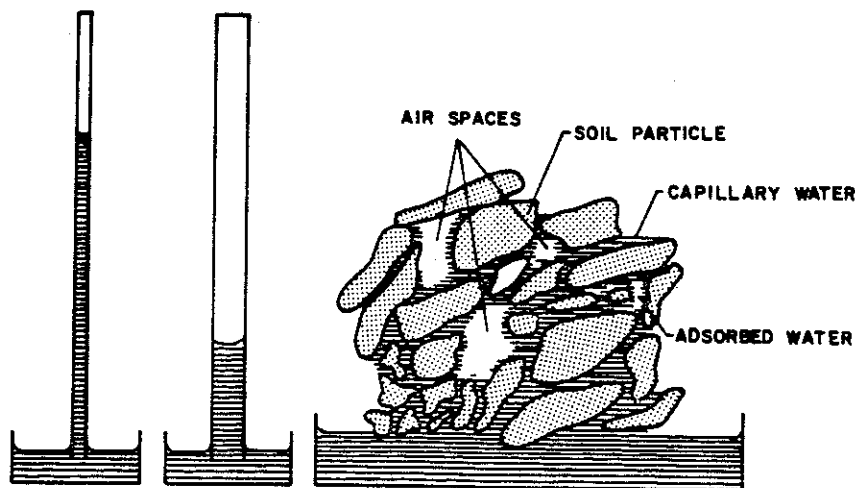


Figure 2. Upward movement by capillarity in glass tubes as compared with soils (after Brady (12))

When the soil is saturated all the pores in the soil are filled with water and no capillary suction occurs. The soil moisture tension is zero. If the soil drains, the largest pores empty first, because they have the least tension to hold water. As drainage continues, progressively smaller pores empty and the soil moisture tension increases because smaller pores have a stronger pull to hold water. Thus, the tension represents the energy state of the largest water filled pores. Finally, with further drainage, only the very narrowest pores are able to exert sufficient capillary "pull" to retain water. Hence, increasing tension or suction is associated with drying.

The rate of decrease of moisture in soil upon increasing tension is a function of its pore-size distribution, and is characteristic for each soil material or type. Figure 3 shows the soil moisture retention curves for a sand, silt loam, sandy loam and a clay soil. The sand has many relatively large pores that drain abruptly at relatively low tensions, whereas the clay releases only a small volume of water over a wide tension range because most of it is strongly retained in very fine pores. The silt loam has more coarse pores than does the clay, so its curve lies somewhat below that of the clay. The sandy loam has more finer pores than the sand so its curve lies above that of the sand.

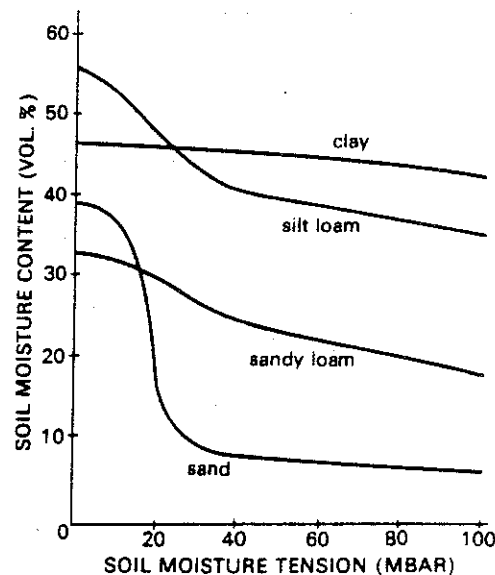


Figure 3. Soil moisture retention for four different soil materials (11)

Liquid Movement in Soils Water will flow from a point where it has a higher potential to a point of lower potential. The gravitational potential acts to move water downward while the matric potential attracts water in all directions but only if the soil is not saturated. The rate of flow increases as the potential difference of potential gradient between points increases. The ratio of the flow rate to the potential gradient is referred to as the hydraulic conductivity or K defined by Darcy's Law.

$$Q = KA \frac{dH}{dZ}$$

where: Q = flow rate
K = hydraulic conductivity
A = cross-sectional area of flow
 $\frac{dH}{dZ}$ = hydraulic gradient

This parameter accounts for all the factors affecting flow within the soil including tortuosity and size of the pores. Thus, the measured K values for different soils vary widely due to differences in pore size distributions and pore continuity.

The hydraulic conductivity often changes dramatically with changes in the soil moisture tension. At a tension equal to or less than zero, the soil is saturated and all the pores in the soil are conducting liquid. When the tension is greater than zero, air is present in some of the pores and unsaturated conditions prevail. This condition grossly alters the flow channel because the forces which cause flow are now associated with capillarity. As the water content decreases or tension increases, the path of the water flow becomes more and more tortuous since the water travels along surfaces and through sufficiently small pores to retain water at the prevailing water potential. Therefore, the unsaturated hydraulic conductivity is usually much lower.

To illustrate this, three different soil materials can be considered with pore size distributions schematically represented in Figure 4. One "soil" is a coarse, porous material (like a sand), one is a fine porous material (like a clay) and a third (like a sandy loam) has both large and fine pores. With an open infiltrative surface and with a sufficient supply of water, all the soil pores are filled and each pore will conduct water downward due to gravity. The larger pores will conduct much more water than the smaller ones. If a weak barrier or crust forms over the tops of the tubes to restrict flow some of the larger tubes will drain. Only the pores with sufficient capillary force to "pull" the water through the crust will conduct water. The larger the pore, the smaller the

capillary force so that progressively smaller pores empty at increasing crust resistance. This crusting leads to a dramatic reduction in the hydraulic conductivity of the soil (see Figure 5).

If no crust is present, similar phenomena occur when the rate of application of water to the capillary system is reduced. With abundant supply, all pores are filled. If the supply is decreased, there is not enough water to keep all pores filled during the downward movement of the water. The larger pores empty first, since the smaller pores have a greater capillary attraction for water. Thus, larger pores can fill with water only if smaller pores have an insufficient capacity to conduct away all the applied water.

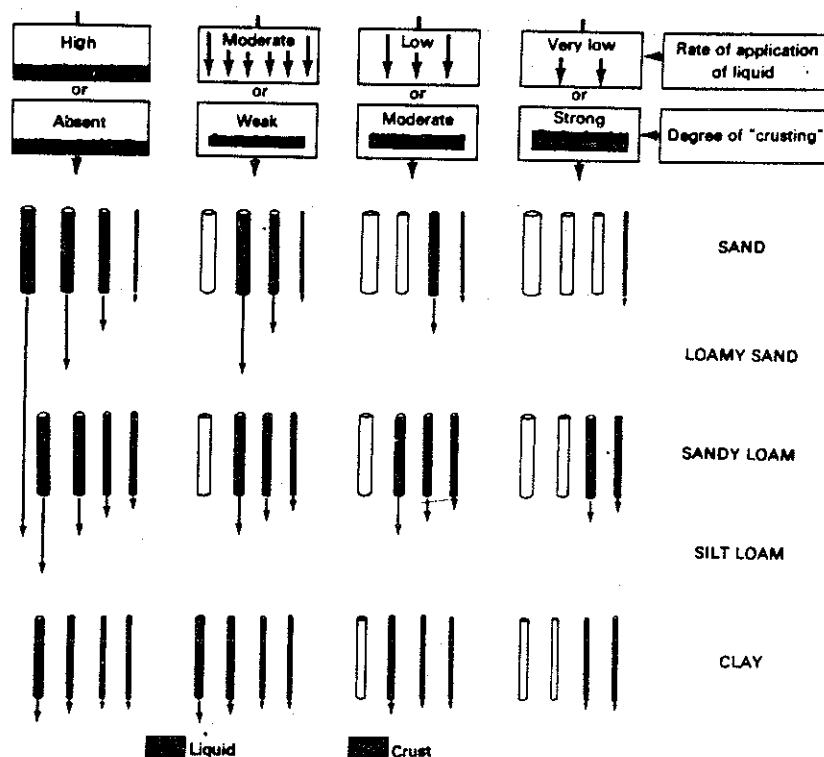


Figure 4. Schematic illustration of the effect of increasing crust resistance or decreasing rate of application of liquid on the rate of percolation through three "soil materials" (11)

The reduction in K upon increasing soil moisture tension is, therefore, characteristic for a given soil texture and structure. Coarse soils with predominantly large pores have relatively high saturated hydraulic conductivities (K_{sat}), but K drops rapidly with increasing soil moisture tension. Fine soils with predominantly small pores have relatively low K_{sat} , but their hydraulic conductivity decreases more slowly upon increasing tension. Hydraulic conductivity or K curves, determined in situ show such patterns for natural soil (see Figure 5).

The K curves for the pedal silt loam and clay horizons demonstrate the physical effect of the occurrence of relatively large cracks and root and worm channels. The fine pores inside peds contribute little to flow. The large pores between peds and root and worm channels give relatively high K_{sat} values (25 cm/day for the silt loam), but these pores are not filled with water at low tensions and K values drop dramatically between saturation and 20 cm tension (1.5 cm/day for the silt loam).

The Process of Pore Clogging When liquid wastes are applied to the soil, a clogging zone often develops at the infiltrative surface. This restricts the rate of infiltration, preventing saturation of the underlying soil even though liquid is ponded above. The soil is then able to conduct liquid only if the water is able to penetrate the clogged zone under the forces of hydrostatic pressure and capillary pull.

Several phenomena contribute to the development of a clogging zone at the infiltrative surface of soil absorption systems. These include (1) compaction, puddling and smearing of the soil during construction; (2) puddling caused by the constant soaking of the soil during operation; (3) blockage of soil pores by solids filtered from the waste effluent; (4) accumulation of biomass from growth of microorganisms; (5) deterioration of soil structure caused by exchange of ions on clay particles; (6) precipitation of insoluble metal sulfides under anaerobic conditions; and (7) excretion of slimy polysaccharide gums by some soil bacteria.

Studies by several investigators indicate that the physical and biological mechanisms are the primary causes of soil clogging in an absorption field not smeared and compacted during construction (11, 13-27). In these instances, clogging seems to develop in three stages: (1) slow initial clogging, (2) rapid increase of resistance leading to permanent ponding, and (3) a final leveling off towards equilibrium. Initial development of the clogging zone seems to be due largely to the accumulation of suspended solids from the wastewater so that the liquid seeps away more and more slowly between loadings. Aerobic bacteria decompose many of the organic solids helping to keep the soil pores open but they can function only when the infiltrative surface drains

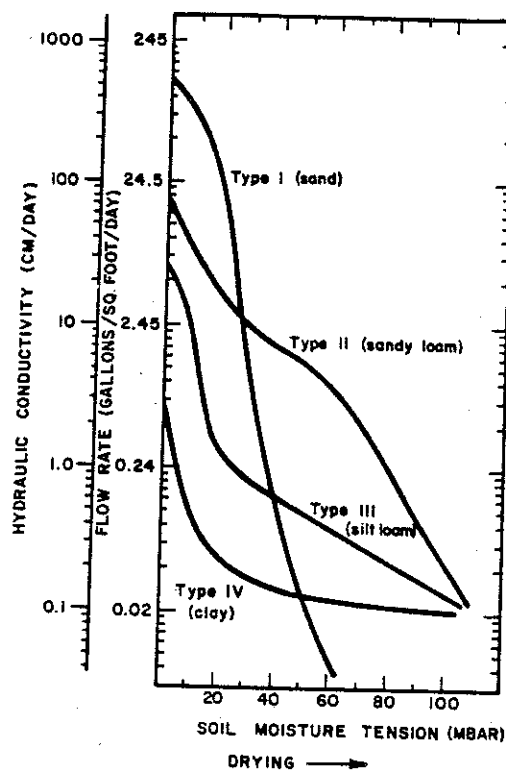


Figure 5. Hydraulic conductivity (K) as a function of soil moisture tension measured in situ with the crust-test procedure (28)

between doses to allow the entry of air. As the clogging zone begins to form, decreasing the aerobic periods between ponding, the aerobic bacteria eventually are unable to keep up with the influx of solids. Permanent ponding finally results, leading to anaerobic conditions where oxygen is no longer present. Any dissolved oxygen in the water is inadequate to maintain the aerobic environment necessary for the rapid decomposition of the organic matter. Clogging then proceeds more quickly due to the less efficient destruction of soil clogging organics by anaerobic bacteria. Sulfides produced by reduction of sulfate by these bacteria bind up trace elements as insoluble sulfides, causing heavy black deposits in the clogging zone. Some anaerobic and facultative organisms which grow in such an environment produce gelatinous

materials (bacterial polysaccharides slimes or gums) which clog the soil pores very effectively. At this point the clogging mat seems to reach an equilibrium state where the resistance to flow changes little. Failure of an absorption field will not occur, however, if the rate of application does not exceed the equilibrium rate. The process can be reversed to restore much of the original infiltrative capacity if the ponded surface is allowed to drain and rest to permit aerobic biological decomposition and the drying and cracking of the clogging materials.

This description of soil clogging assumes that the native soil structure is left relatively intact at the infiltrative surface during construction of the system. However, many systems fail, usually within a year or two, because of poor construction techniques. Absorption of water by soils depends upon preservation of a suitable soil structure, but soil structure can be partially or completely destroyed by compaction and smearing during construction. Extensive damage does not occur in soils with a single-grained structure (sands) but can be very serious in aggregated soils with high clay contents. When mechanical forces are applied to a moist or wet soil, the water around clay particles acts as a "lubricant" causing the soil to exhibit plasticity where individual soil particles move relative to one another. Such movements, referred to as compaction, puddling, or smearing, close the larger pores. Structural damage increases as soil wetness and clay content increase. Compaction may result from frequent passes over the field by heavy machinery, smearing of the soil surface by excavating equipment and puddling by exposure of the infiltrative surface for a day or more to rainfall or wind-blown silt that seals off the soil pores. The result is that the absorption field may be clogged before it is put into service.

Infiltration is not only dependent upon the resistance of the clogging zone but also on the capillary properties of the underlying soil (28). For example, an identical "crust" with a resistance of 5 days (the length of time for 1 cm^3 to pass through 1 cm^2 of barrier with a head of 1 cm) and ponded with 5 cm of liquid would induce flow rates of 8 cm/day in a sandy loam; 7 cm/day in a sand; 4 cm/day in a silt loam; and 1.8 cm/day in a clay (28). Crusts with very high resistances would conduct more liquid when overlying a clay than when overlying a sand. Thus, similar clogging zones developed in different soils have different conductivities.

The Significance of Unsaturated Flow Flow of liquid in unsaturated soil proceeds at a much slower rate than in saturated soil because flow only occurs in the finer pores. This slows the rate of infiltration into the soil but enhances purification. Wastewater effluent is purified by filtration, biochemical reactions and adsorption, processes which are more

effective in unsaturated soils because average distances between effluent particles and the soil particles decrease while the time of contact increases. This flow phenomenon can be illustrated by an example (11). Figure 6 shows a thin section of the C horizon of a Saybrook silt loam, which is a stony sandy loam till with a saturated hydraulic conductivity of 80 cm/day. The flow velocity of water in the soil pores can be estimated from its moisture retention curve (Figure 3). This velocity can be used to derive the time for water to travel one foot (30 cm), assuming a hydraulic gradient of 1 cm/cm due only to gravity. Successively smaller pores empty at increasing tensions and K decreases correspondingly (see Figures 4 and 5). Calculated travel times increase from 3 hours at saturation to 30 hours at 30 mbars and 8 days at 80 mbars of soil moisture tension.

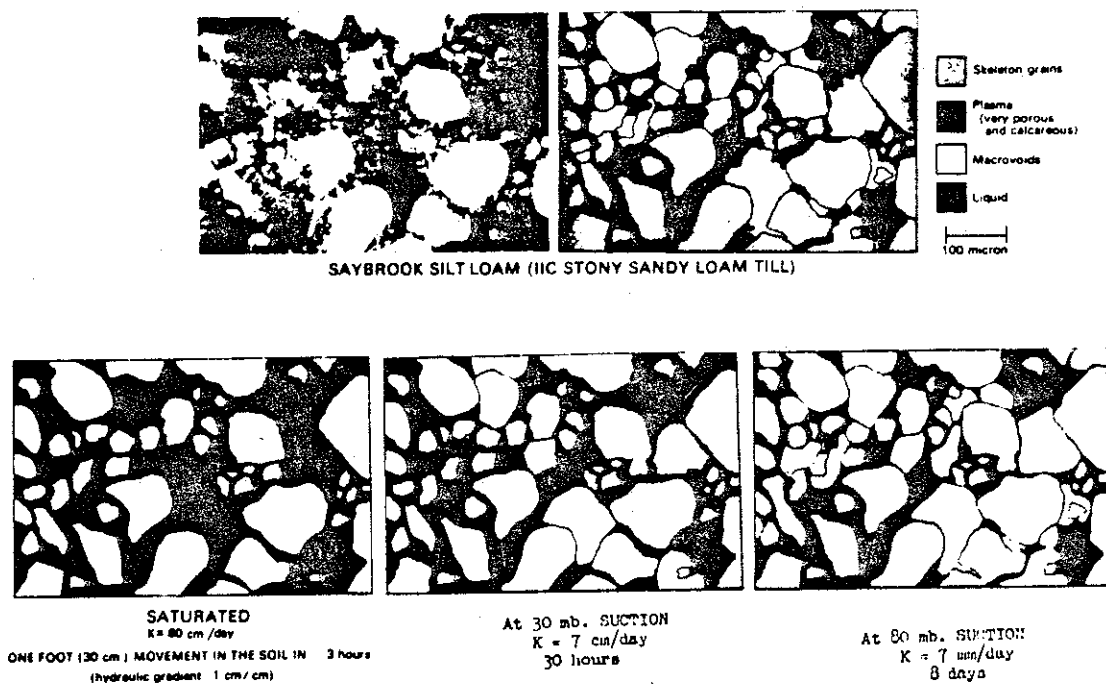


Figure 6. Occurrence and movement of liquid in a saturated and unsaturated sandy loam till C horizon of Saybrook silt loam (11)

In structured soils it is possible to have flow predominantly through the planar voids, thus bypassing the interior of the peds. High liquid applications may result in high dispersion where the water passes through the planar voids without displacing the water already in the peds. In such

instances, short-circuiting of liquid through the soil occurs with associated low retention times. Low applications of water would displace more of the water in the peds and have low dispersion. Differences in dispersion related to different structures while following chloride movement in soil columns has shown short circuiting to be a particular problem on drained soils dosed at relatively high rates (29).

Short circuiting in a structured soil is schematically illustrated in Figure 7. If the large planar voids are drained and air filled, a high application rate of liquid applied at the surface will quickly pass through the large pores before much can enter the fine pores of the peds. Therefore the retention time of the bulk of the liquid is low and only a portion of the entire soil volume is used to transmit the fluid.

If the application rate is low or if there is a barrier to flow, the dispersion is low. The large pores will not fill with liquid and flow will be through the fine pores in the peds. In this case the retention time will be long and flow will only be through the portion of the soil most effective in renovation.

Long liquid travel times are desirable to adequately purify the wastewater. The design of absorption systems may be critical to achieve this in some soils. Travel times are sufficiently long under all moisture tensions to affect adequate purification in clay, but are too short in sand and sandy loams when the soil is near saturation. Once a clogging zone has developed in such permeable soils, moisture tensions reach a level where sufficiently long travel times result. However, when an absorption system constructed in a highly porous or dry structured soil is first put into service without a developed clogged zone, there is danger that adequate purification may not be achieved unless precautions are taken in design to insure unsaturated soil conditions are maintained.

Wastewater Treatment Capabilities of Soil Materials

The principal goal in liquid waste disposal for homes in unsewered areas is the purification of the liquid before it reaches potable or recreational waters. Organic matter, chemicals and pathogenic organisms and viruses that are not removed prior to application to the soil must be removed or transformed by the soil material. Numerous studies have shown that under proper conditions, the soil is an extremely efficient purifying medium.

Bacteria and Virus Removal by Soil From the standpoint of public health, removal of disease organisms and viruses is the most critical function of a soil disposal field. Many field and laboratory studies have examined the efficiency of the soil for pathogen removal and the various parameters that

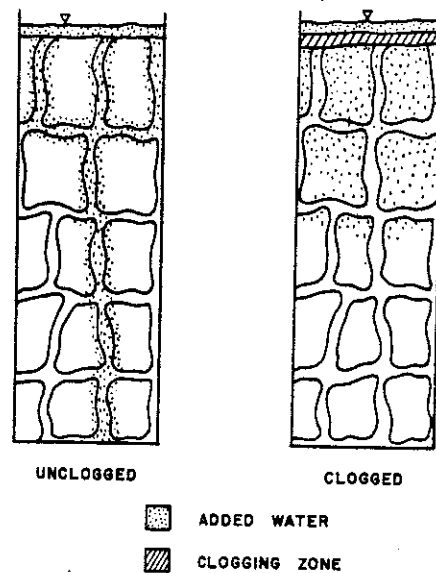


Figure 7. Influence of clogging zone on short circuiting in structured soils

affect its efficiency. Factors important in removal of pathogens by soil include soil type, temperature, pH, organism adsorption to soil and soil clogging materials, soil moisture and nutrient content and biological antagonisms (30). Another key factor is the liquid flow regime in the soil. As shown previously, unsaturated flow, induced by either a clogged zone or application rate, enhances purification because liquid movement is through only the smaller pores of the soil.

Figure 8 shows removal of fecal coliforms and fecal streptococci from septic tank effluent by two columns packed with 2 feet of Plainfield loamy sand (effective size 0.14 mm, uniformity coefficient 1.99) (31, 32). Both columns were loaded well below their saturated hydraulic conductivity rates of nearly 400 cm/day (96 gpd/ft²) but one was loaded at twice the rate of the other. During the first 100 days of application, the number of bacteria discharged from both columns reached a plateau and then began to decline. Fewer bacteria passed through the column with the lower loading rate. Column 1, loaded at 10 cm/day (2.4 gpd/ft²), removed approximately 92% of the fecal coliforms applied per day while column 2, loaded at 5 cm/day (1.2 gpd/ft²), removed 99.9%. Fecal streptococci

and *Pseudomonas aeruginosa* were also found in the effluent from the more heavily loaded column 1. These organisms were not detected in effluent from the more lightly loaded column 2. During this period a clogging zone developed on the infiltrative surface of each column and the fecal coliform count in the effluents from both columns eventually dropped to between 10 and 100 FC/100 ml (32).

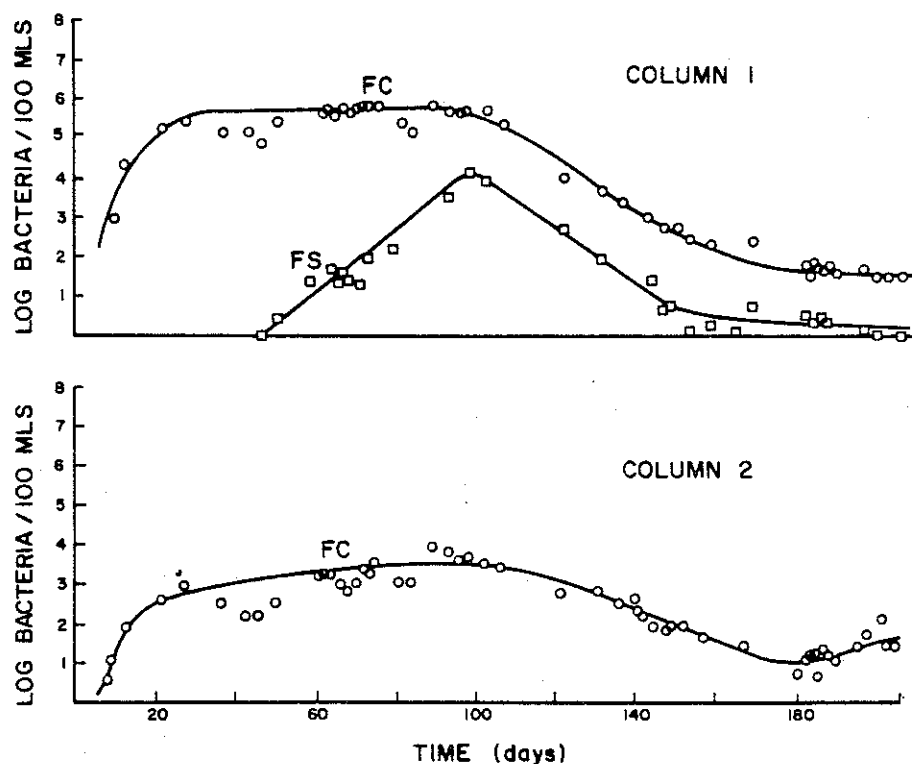


Figure 8. Bacteria counts in effluents from sand columns loaded with septic tank effluent. Column 1 loaded at 10 cm/day (1.5 hours retention time) and Column 2 loaded at 5 cm/day (25 hours retention time)
 FC = fecal coliforms FS = fecal streptococcus (31)

Septic tank systems installed in sands also exhibit the effects of the clogging zone in removing indicator bacteria. Figure 9 shows the bacterial counts obtained while monitoring several points around an absorption trench in an unsaturated medium sand soil. The kinds and numbers of bacteria found in the liquid 1 foot (30 cm) below and 1 foot (30 cm) to the side of the trench were similar to natural soil flora (11,31,32).

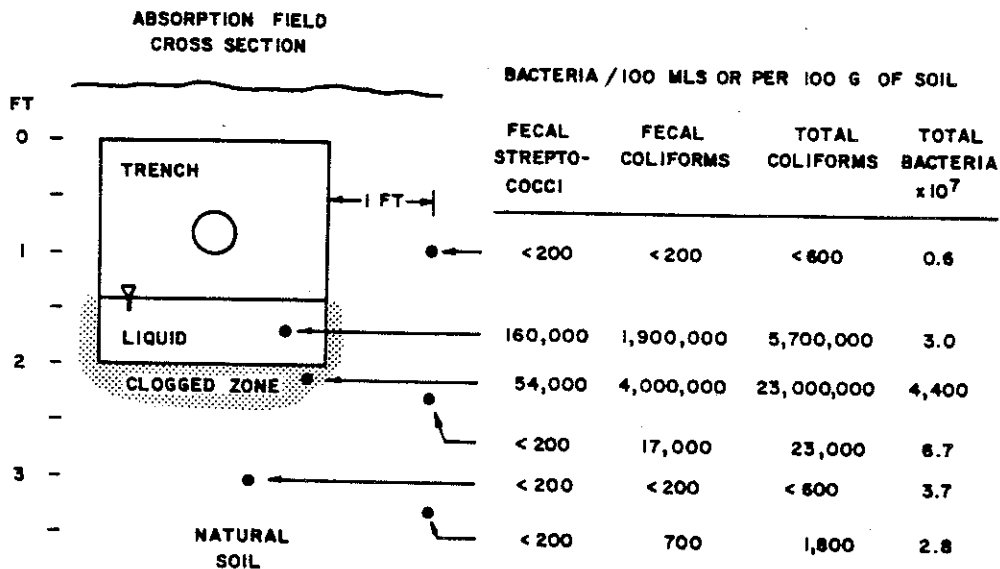


Figure 9. Cross-section of seepage trench in sand showing bacterial counts at various points near the trench (18, 20)

Concurrent studies of Almema silt loam were also conducted (32). This soil has a lower capacity to conduct liquid than the unstructured sands and the majority of flow is through the larger pores between soil peds. Undisturbed cores, 2 feet deep, of Almema silt loam were loaded with septic tank effluent at a rate of 1 cm/day (0.24 gpd/ft²). At this loading, effluent short-circuited through large pores and channels and significant numbers of bacteria were found in the column effluents. When the loading rate was reduced from 1 cm/day to 3 mm/day to promote slow flow through the soil peds rather than through the larger cracks around the peds (Figure 7), bacterial counts decreased dramatically to below 2/100 ml of fecal coliforms, fecal streptococcus and *P. aeruginosa*. When the loading was restored to 1 cm/day, high counts of these organisms were again observed (see Figure 10).

Virus adsorption and inactivation in soils have been of considerable interest to scientists and engineers over the years. When virus enter the septic tank or other treatment process, they are likely associated with cells in fecal material. These masses settle releasing some virus depending upon turbulence within the process (up to 89% of the poliovirus added in fecal material was released by vigorous shaking in laboratory studies (34)). Secondary adsorption on wastewater solids may occur in

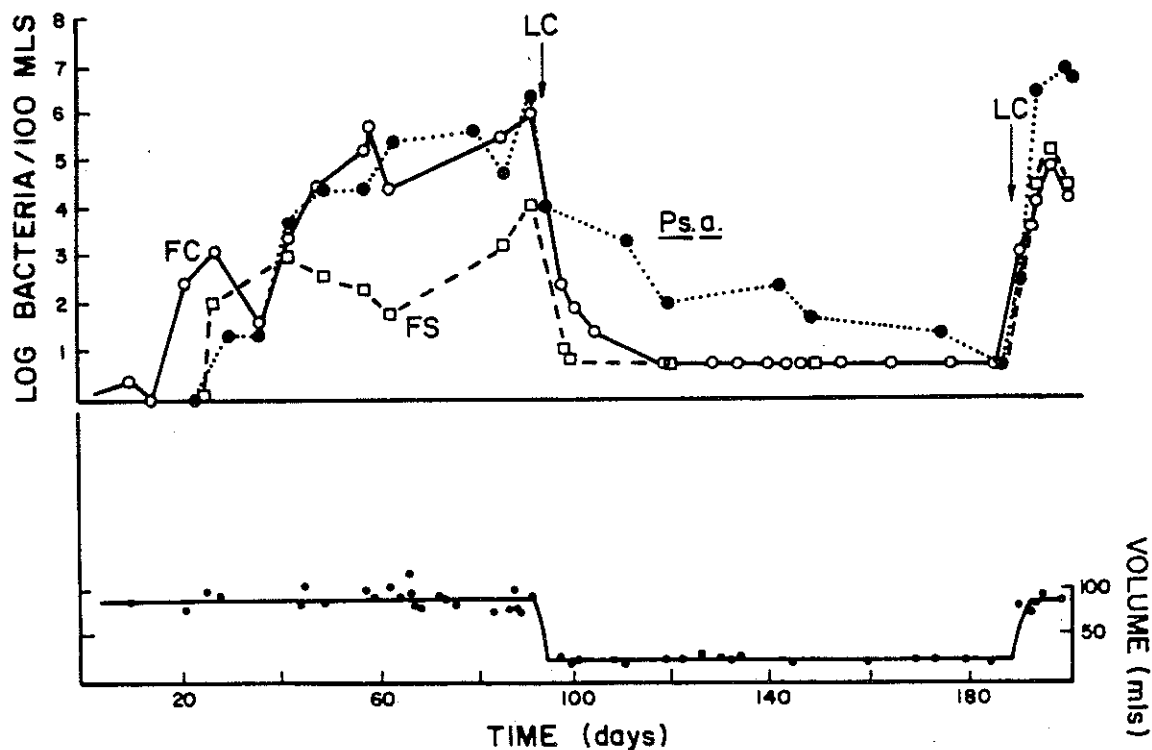


Figure 10. Bacteria counts in effluent from an undisturbed core of Almena silt loam loaded with septic tank effluent (LC = loading change from 1 cm/day to 3 mm/day and return to 1 cm/day) (31)

treatment processes. The free and particle adsorbed virus will then be discharged to subsequent treatment processes or the soil absorption field.

Removal of virus in soils occurs as the result of the combined effects of sorption, inactivation and retention. Upon entry into the soil, virus are rapidly adsorbed to solid surfaces. Desorption appears to be strongly related to the ionic strength of the applied fluid, increasing as the ionic strength decreases (36). In the adsorbed position, the virus are inactivated in a spontaneous process which is temperature dependent, being greater at the higher temperatures (34). Virus detention within the soil is affected by the degree of saturation of the pores through which the virus laden effluent flows. The more saturated the pores, the less opportunity there is for virus contact with surfaces to which it can adsorb.

In laboratory studies with packed sand columns, septic tank effluent was inoculated with more than 10^5 plaque-forming units (PFU) per liter of poliovirus type I (34, 35). All viruses were removed in the 24-inch columns at a loading rate of 5 cm/day (1.24 gpd/ft²) over a period of more than one year. At a loading rate of 50 cm/day (12.4 gpd/ft²), virus breakthrough occurred (Figure 11). Analysis of the sand residue following virus application, indicated that adsorbed virus within the column were inactivated at a rate of 18% per day at room temperature and at 1.1% at 6 to 80°C (35).

In contrast to these results, virus were detected approximately 60 inches within columns packed with calcareous loamy sand and fed secondary effluent containing 3×10^4 PFU polio virus type I at a rate of 15 cm/day (3.70 gpd/ft²) (36). Most of the virus was adsorbed in the top 2 inches of the soil and virus removal was not appreciably affected at application rates of between 15 and 55 cm/day (3.70 and 13.6 gpd/ft²). Only deionized water desorbed the virus but drying for five days prevented desorption even with deionized water.

Laboratory tests with ground soil from a Batavia silt loam reduced virus in septic tank effluents by 5.4 logs per cm and Almena silt loams material produced 7.9 logs of reduction per cm (35). It should be emphasized, however, that soils in the field do not exist in a finely ground state. Channels in natural soil will reduce opportunity for virus adsorption and travel over long distances may occur when loading rates are high.

Chemical Transformations and Removals by Soil Domestic wastewaters may contain a few chemicals hazardous to public health or the environment. Nitrogen and phosphorus compounds are discharged in household wastewater which can enter ground or surface waters in sufficient quantities to cause concern. Nitrogen, in the form of nitrate or nitrite has been linked to cases of methemoglobinemia in infants (37). A safety limit for nitrate of 10 mg/l as nitrogen is recommended by the U.S. Public Health Service (38). There are many reports of nitrate concentrations above 10 mg/l N limit in wells near septic tank systems (37, 39-41). Accelerated eutrophication of surface waters is also attributed to nitrate contributed from waste discharge (40).

In solution, nitrate moves freely through the soil though some denitrification (reduction of nitrate to nitrogen gas) can occur where organic material and an anaerobic environment occur together. Nitrogen in septic tank effluent is about 80% ammonium and 20% organic nitrogen, but much of it is converted biologically to nitrate as it moves through the aerated unsaturated soil immediately below the clogging zone in

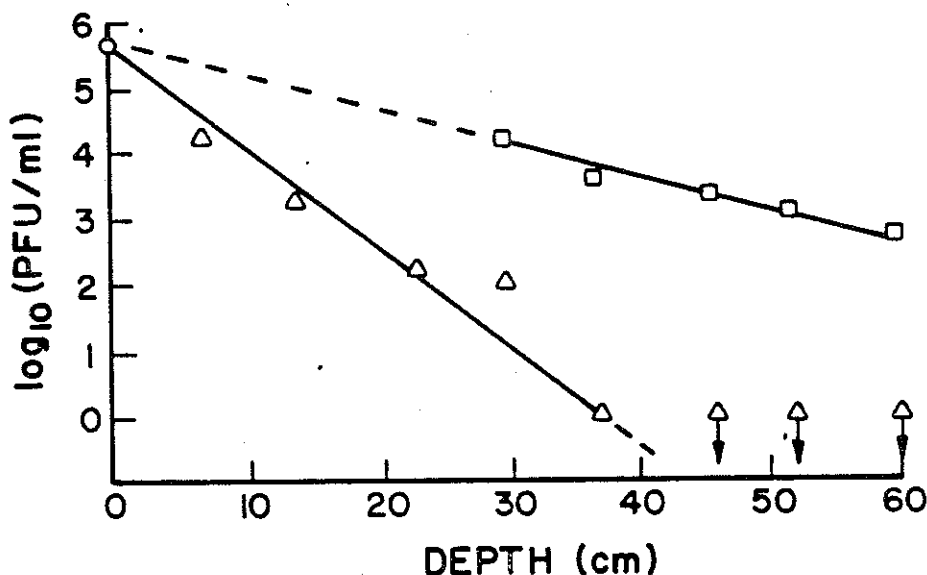


Figure 11. Penetration of polio virus into a 60 cm conditioned sand column at room temperature. Fluid samples (Δ 5 cm (850 ml) dose, \square 50 cm (8.5 l) dose, \bullet input titer, \downarrow <indicated value) (35)

the seepage field (42). This is illustrated in Figure 12 where concentrations of the various forms of nitrogen are plotted against depth below a soil trench in a sandy soil. If anaerobic conditions prevail in the subsoil, nitrification will not occur and the nitrogen then remains in the form of ammonium. Ammonium is readily adsorbed by soil materials of high clay content and hence migrates much more slowly (39, 42).

Phosphorus is also of environmental concern. If allowed to reach surface waters, it can accelerate eutrophication because it is an essential nutrient of algae and aquatic weeds. However, phosphorus enrichment of groundwater seldom occurs below septic tank systems because phosphorus is fixed in soil by sorption reactions or as phosphate precipitates of calcium, aluminum or iron. Calcium is usually found in the wastewater itself and aluminum and iron are abundant in most soils (43). Phosphorus leakage to the groundwater may occur, however, where high water tables exist, very coarse sand and gravel occur, or where the seepage bed has been loaded heavily for a long period of time. In such instances, concentrations of phosphate above 5 mg/l P have been observed (40). Phosphorus

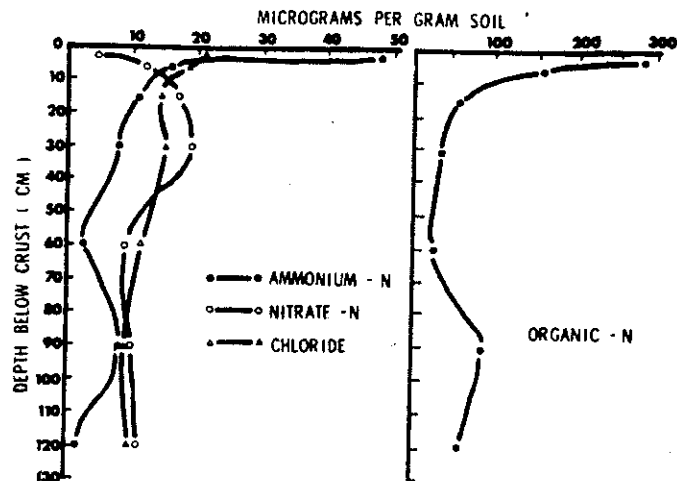


Figure 12. Concentrations of $\text{NH}_4\text{-N}$, $\text{NO}_3\text{-N}$, Organic N and Cl in unsaturated soil below the clogged zone in sand (31)

can move downward 50-100 cm per year through "clean" silica sand (44) but, movement in loams, silt loams and clays is much slower (5-10 cm per year). Thus, except in coarse soils, over 10 years would be required for the phosphorus to move as much as 3 feet (44).

Heavy metals and complex organic compounds are also effectively adsorbed by soil and therefore removed from the percolating wastewater.

ESTIMATION OF THE INFILTRATIVE AND PERCOLATIVE CAPACITY OF THE SOIL

Criteria for site selection for on-site systems vary from folk knowledge and experience to various empirical methods of site testing, often codified into rules. The U.S. Public Health Service has developed a general reference manual (45), which has provided guidelines for many state, regional and local manuals or codes of practice.

Several factors are usually considered in the selection of a site for a septic tank system. The ability of the soil to absorb liquid usually estimated by the percolation test, is a common requirement. Other factors include slope, depth to groundwater, nature and depth to bedrock, likelihood of seasonal flooding and distance to well or surface water (45). These traditional factors have several limitations and vary widely between codes.

Estimation of Soil Permeability

The Percolation Test In 1926, Henry Ryon developed a test to obtain field data on failing seepage systems (46). He dug a hole one foot square to the depth of the failed systems, filled it with water, allowed the water to seep away, refilled the hole with water and recorded the time required for the water level to drop one inch ("percolation rate"). To calibrate the test, Ryon inspected several failing or near-failing systems and noted the loading of the system, the soil characteristics and the percolation rate measured in nearby soil. Ryon plotted curves relating permissible loading rates versus the percolation rate from these data. It was later proposed that these curves could be used to size new soil absorption systems. Adoption of the procedure by the New York State Health Department led to its wide acceptance though slight changes have been made over the years. Today it is used by nearly every state to size on-site systems.

The percolation test is based on the assumption that the ability of a soil to absorb sewage effluents over a prolonged period of time may be predicted from its initial ability to absorb clear water (46). From Ryon's data comparing absorption rates of existing septic tank systems to the percolation test, the measured rate is reduced by a factor ranging from 20 to 2500 in order to size the absorption area (11). However, the results of the test are highly variable and its use for system sizing relies on an empirical relationship between the measured percolation rate and

the actual loading rate. Tests run in the same soil vary by as much as 50% (47, 48). Thus, the procedure is unreliable, requiring a more accurate test.

The 'Crust Test' The soil below most operating absorption systems is unsaturated because of the clogging mat which develops at the infiltrative surface. To properly size an absorption system, therefore, the unsaturated hydraulic conductivity characteristics of the soil must be known. Since the standard percolation test does not provide conductivity data of this type the "crust test" was developed (11, 47, 49-52).

The crust test is performed in situ to avoid disturbing natural pores and to maintain continuity with the underlying soil. A carved soil column is fitted with a ring infiltrometer (an impermeable collar with a tight fitting lid) to control water addition to the column. A tensiometer is installed in the column just below the infiltrative surface to determine the degree of saturation in the soil by measuring the soil moisture tension as water is applied (see Figure 13). To maintain unsaturated conditions in the soil column, a "crust", made of gypsum and sand is placed over the soil surface. When water is introduced through the infiltrometer, flow into the soil is restricted by the crust. This establishes a constant steady-state flow rate which induces a nearly uniform moisture tension in the soil beneath the crust. The measured soil moisture tension and the equilibrium flow rate locate one point on the hydraulic conductivity curve. Additional tests run with crusts of different hydraulic resistances define the K-curve as shown in Figure 5. This curve can be used for design if the range in soil tensions under the clogged zones of mature absorption systems in similar soils is known.

Though this procedure offers a direct measurement of K, it is time consuming and requires a skilled operator. It is not a test which can be run economically at each site. However, since the hydraulic conductivity of a soil is dependent upon the pores in the system, the conductivity of a soil at various sites in the soil map unit can be defined within statistical limits (see Figure 14) (53). Also, it has been found that variability curves of different soils in the same textural groups have similar conductivity curves (see Figure 15 and Table VI) (53). Therefore, by defining families of K-curves for groups of soils the hydraulic conductivity characteristics of a particular soil or site can be predicted.

In Wisconsin, four major hydraulic conductivity types have been suggested based on the texture of the soil materials (28). These textural groupings include the sands; sandy loams and loams; silt loams and some silty clay loams; and the clays and some silty loams. (In other regions

Table VI. Summary of morphological characteristics
Soil series from Figure 14 (53)

Soil identification	Horizon selected	Texture	Structure	Frequency of biopores
Group A				
Ontonogon series	B2	heavy loam	moderate, medium	few coarse,
Very fine, illitic,			angular blocky	common
frigid				medium
Typic Eutroboralf				
Magnor series	IIB	heavy loam	moderate, medium	few coarse,
Fine-loamy, mixed			subangular blocky	common
Aquic Glossoboralf				medium
Group A				
Planb series	B2	silty clay	moderate, medium	few coarse,
Fine-silty, mixed,		loam	subangular blocky	common
Mesic				medium
Typic Arguidoll				and fine
Batavia series	B2	silty clay	moderate, medium	few coarse,
Fine-silty, mixed		loam	subangular blocky	common
Mesic				medium
Mollic Hapludalf				and fine
Morley series	IIB3	heavy, silty	moderate, medium	few coarse,
Fine, illitic,		clay loam	prismatic, too	common
Mesic			coarse, blocky	medium
Typic Hapludalf			structure	

similar groupings might be made but they must be based on field data since differences in soil mineralogy may affect these groupings.) Typical hydraulic conductivity curves were developed from field measurements for each of these conductivity types (see Figure 5).

To make these curves useful in designing soil absorption fields for septic tank systems, soil moisture tensions were measured under the clogging zones of several operating absorption fields (28). This information provided a design point on the curve for proper field sizing. This same procedure could be used to select design points for other types of soil systems. (see Table VII).

The application rates for various soils presented in Table VII represent the best estimates available to date. Because of the unstructured nature of the sands and sandy loams the rates are reasonably accurate. However, because of the nature of the flow which occurs in finer textured, structured soils there is more variability in the tensions measured under operating fields (28). In these soils the design rates must be used with care particularly where expandable clays are present.

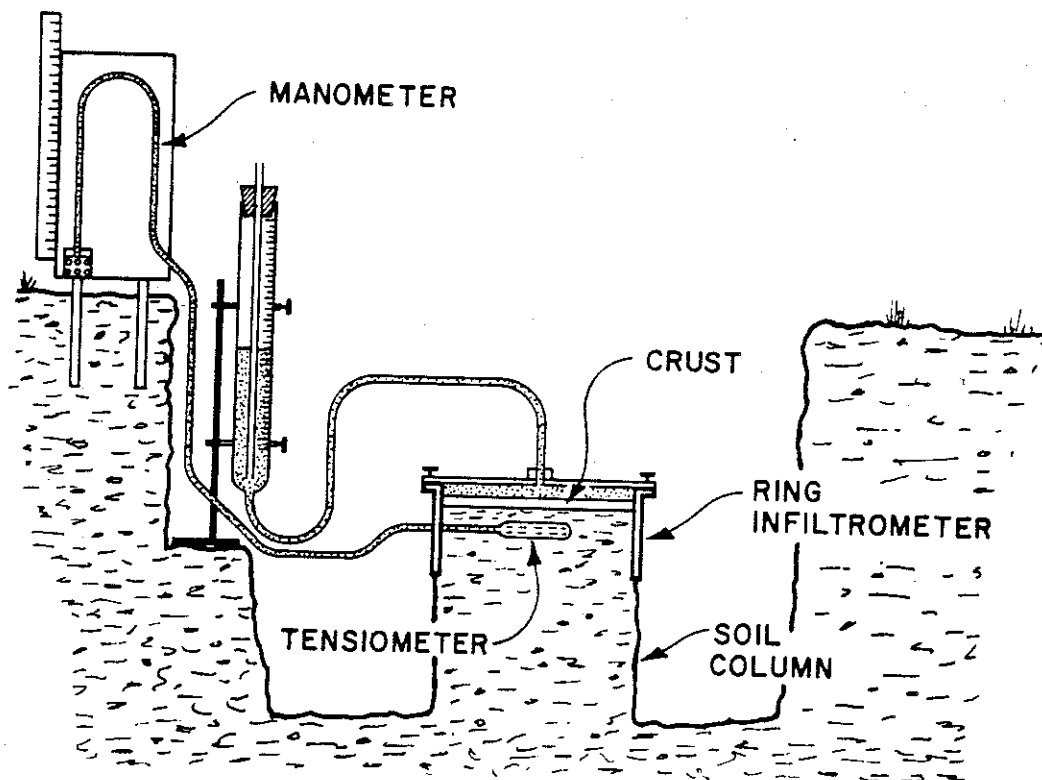


Figure 13. Schematic diagram of the crust test procedure.

Table VII. Recommended maximum loading rates of septic tank effluent for different soil types

Estimated percolation rate (min/inch)	Soil texture	Loading rate + cm/day (gpd/ft ²)		Operating conditions (28)
		Bouma(28)	Machmeier(54)	
0-10	sand	5 (1.23)	(1.20)	4 doses/day uniform distribution trenches or beds
10-30	sandy loams, 3 loams	(0.72)	(0.60)	1 dose/day uniform distribution trenches preferred
30-45	*some porous silt loams, and silty clay loams	3 (0.72)	(0.50)	1 dose/day uniform distribution desirable shallow trenches only
45-90	*clays, some compact silt loams and silty clay loams	1 (0.24)	(0.45)	dosing and uniform distribution desirable shallow trenches only

+Bottom area only.

*Should not be applied to soils with expandable clays.

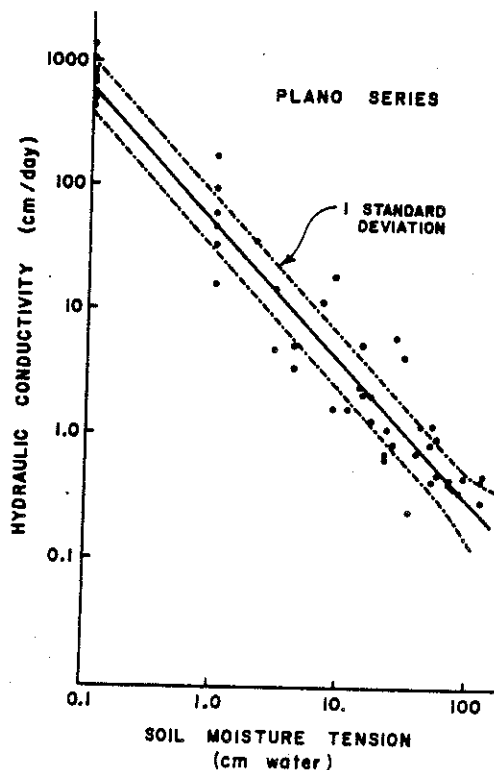


Figure 14. Hydraulic conductivity data for Plano series. Regression line is solid line, and dashed lines indicate one standard deviation about regression line (after Baker (53))

Estimation of High Groundwater

To insure adequate purification of the wastewater before it reaches groundwater, three feet of unsaturated soil is necessary below the infiltrative surface. If saturated soils ever occur within the three feet minimum, transmission of harmful pollutants to the groundwater may result (31-36). To determine if saturated conditions do occur within the minimum is often difficult, however, because water table levels fluctuate in response to changing weather conditions. Typically, the water table is low during the summer, while in the spring and fall, it rises. Ideally, the highest groundwater level should be observed when it occurs, but this is not very practical. Moreover, observations

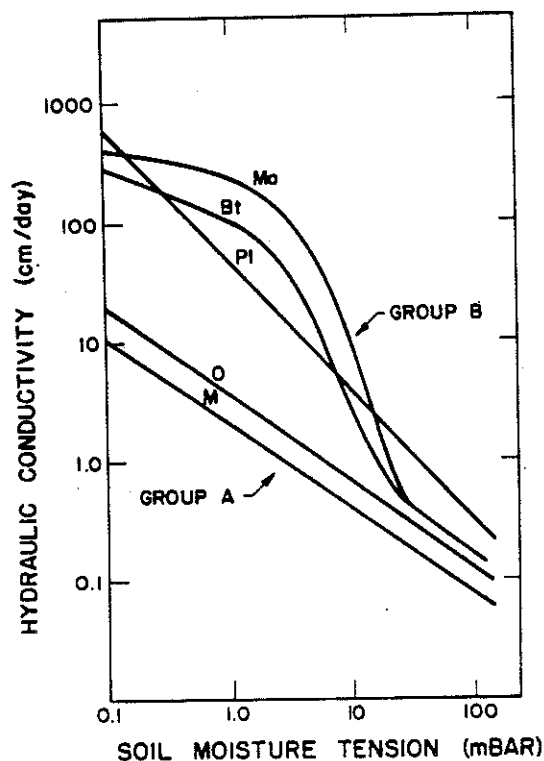


Figure 15. Hydraulic conductivity groups;
 Group A: Ontonogon and Magnor series
 Group B: Plano, Batavia and Morley series
 (After Baker (53))

made in relatively dry years do not represent those that occur in normal years. Thus, other methods must be used to determine the high water elevation.

Soil mottling is sometimes an indicator of the presence of seasonally high water levels. Mottles are spots of contrasting colors found in soils subject to periodic saturation. The spots are usually bright yellow-orange-red surrounded by gray-brown matrix and described according to their color, frequency, size and prominence (55). Well-drained soils are usually brown in color due to the presence of finely divided insoluble iron and manganese oxide particles distributed throughout the horizon. However, under reducing conditions often produced by saturation over

prolonged periods, the iron and manganese is mobilized until reoxidized when the soil drains. Repetitive wetting and drying cycles quickly produce local concentrations of these oxides on pore surfaces forming red mottles (56). Soil from which much of the iron and manganese has been completely reduced loses its brown color becoming grey by a process referred to as gleying. Therefore, the upper limit of the mottled soil is often a good estimate of the high groundwater level though it may also be due to a periodic perched water table. The latter instance may be confirmed by the lack of mottles or gleying in lower horizons (57).

MAINTAINING THE INFILTRATIVE CAPACITY OF THE SOIL

Loading rates recommended in Table VII are based on observations from properly functioning septic tank-soil absorption systems. If a system is to operate satisfactorily for a reasonable length of time at these loading rates, then the infiltrative capacity of the soil must be maintained. This requires that proper design, construction and maintenance procedures be followed.

Sizing of the Soil Absorption System

Estimation of Flow Waste flows from single homes, restaurants, motels, etc., are intermittent and subject to wide fluctuations. Variation in the number of persons contributing to the flow and their activities have profound effects on the daily volume of waste discharged. Accurate estimates of waste flow volumes are therefore difficult.

A study of eleven rural homes showed the average per capita flow from a single household to be 43 gpd (58-60). The greatest flow contributions come from the laundry and bathing events (see Figure 17). Social events, such as family gatherings and overnight guests, will accentuate the peak flows shown. In addition, the number of people occupying the home may increase through additions to the family or sale of the house. Thus, the use of the average per capita flow for design purposes is unwise. Rather, it is necessary to design for the expected peak flows to ensure the system will not fail when large flows occur.

Peak flows can be empirically estimated for households by assuming two people occupy each bedroom. This is realistic since the number of occupants in a home is a function of the number of bedrooms. Figure 16 shows that a peak flow of about 3 gal/hr/capita can be expected which corresponds to approximately 75 gallons per person for the entire day. Thus, 150 gpd/bedroom gives a reasonable estimate of the peak flow. This is the design basis recommended by the Public Health Service (45) which has proved satisfactory in practice.

Estimation of flow from public buildings, commercial establishments and recreational facilities is more difficult. The Small Scale Waste Management Project is presently determining various daily and peak flows from bowling alleys, camps, churches, schools, country clubs, laundromats, marinas, motels, restaurants, service stations, shopping centers, theaters and stadia.

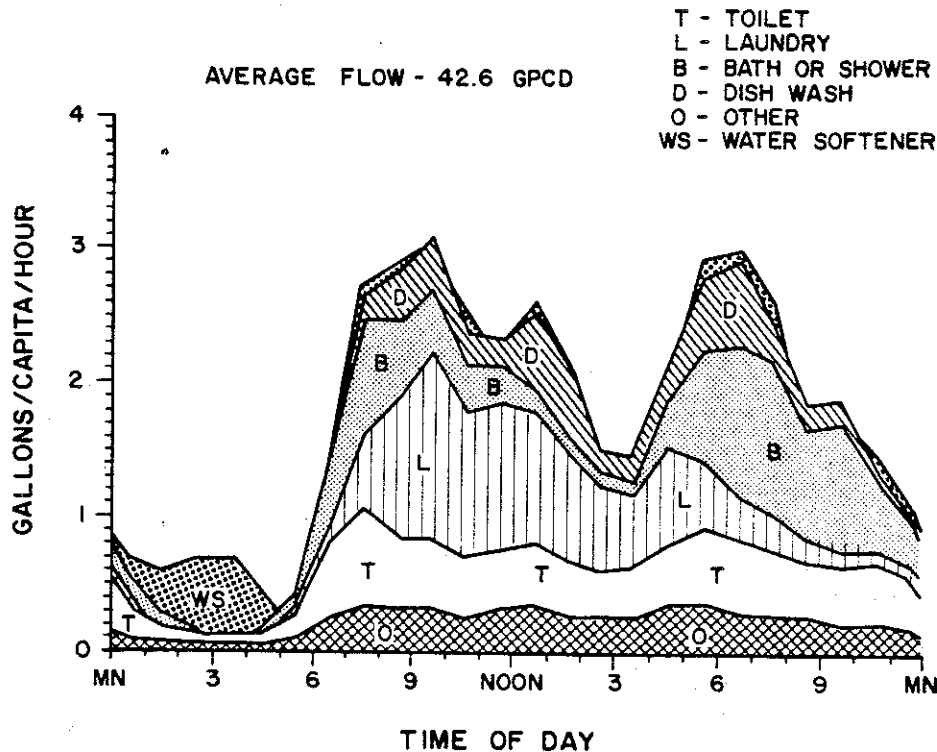


Figure 16. Average daily flow pattern from eleven rural households (58)

Sidewall Versus Bottom Area Absorption A soil adsorption system has two infiltrative surfaces; the horizontal bottom of the trench or bed and the vertical sidewall. When the bottom area begins to clog, the waste effluent ponds in the system and the sidewall begins to absorb liquid (28). In some soils the sidewall may become the more significant infiltrative surface as clogging continues (16, 46).

The rate of water movement through soil is proportional to the total water potential gradient primarily due to the gravitational and matric potentials. In an unclogged absorption system, the potential gradient is lower for the sidewall area than the bottom area because the gravitational potential is zero. As the clogged zone develops, the matric potential of the bottom area may be reduced to where the sum of the gravitational and matric potential is less than the matric potential of the sidewall area.

The sidewall then becomes the dominating infiltrative surface.

Absorption systems should be designed to maximize the most significant infiltrative surface. For the Midwest, the bottom areas are more important because of changes in moisture tensions occurring in the soil during wet seasons. The horizontal gradients can be reduced to levels lower than the vertical gradients because of relatively low natural drainage rates. This is particularly true in early spring and late fall when evapotranspiration rates are low. The matric potential is lowered because of soil wetness. Maximizing the infiltrative area through consideration of sidewalls as a reserve capacity is recommended but in the Midwest, the bottom area should be sized to absorb the entire estimated daily flow.

Distribution of Liquid Over the Infiltrative Surface

Localized overloading of septic tank effluent on the soil often occurs because of poor distribution. This may result in poor purification of the effluent in highly permeable soils and accelerate clogging in slowly permeable soils. Uniform application of the wastewater over the infiltrative surface is usually beneficial.

Absorption systems with uniform distribution and dosing are not necessary in all types of soil to eliminate poor purification and soil clogging. Sands and weakly structured sandy loams and loams would benefit most (61). After a new system is put into service in natural sands, local overloading may cause unnoticed groundwater contamination until clogging develops. Development of a clogged zone may take several years. Conversely, excessive clogging due to poor distribution tends to occur in weakly structured soils. Uniform distribution aids in reducing the clogging by applying the liquid simultaneously to the entire infiltrative surface at rates no greater than the soil is able to accept (62).

Gravity Distribution Liquid flow by gravity is the most common method of distributing waste effluent over the infiltrative surface of the soil absorption field. Perforated 4-inch diameter pipe is laid level or at a uniform slope to 2 to 4 inches per 100 feet with the holes downward. Such a system does not provide uniform distribution. The liquid trickles out the holes nearest the point of inlet and at points of lowest elevation (see Figure 16).

Clogging usually seems to start near the inlet of the absorption system and progresses down the length of the bed (11).

The large holes permit too much liquid to be discharged close to the inlet. Thus, the soil below these points receives a nearly continuous trickle of water and is soon constantly ponded. Clogging develops forcing the liquid to infiltrate further down the trench where the infiltrative surface is still fresh. This sequence continues until the entire bottom is clogged (Figure 17). Altering the orientation of the holes or changing the slope of the pipe does not improve distribution significantly (63).

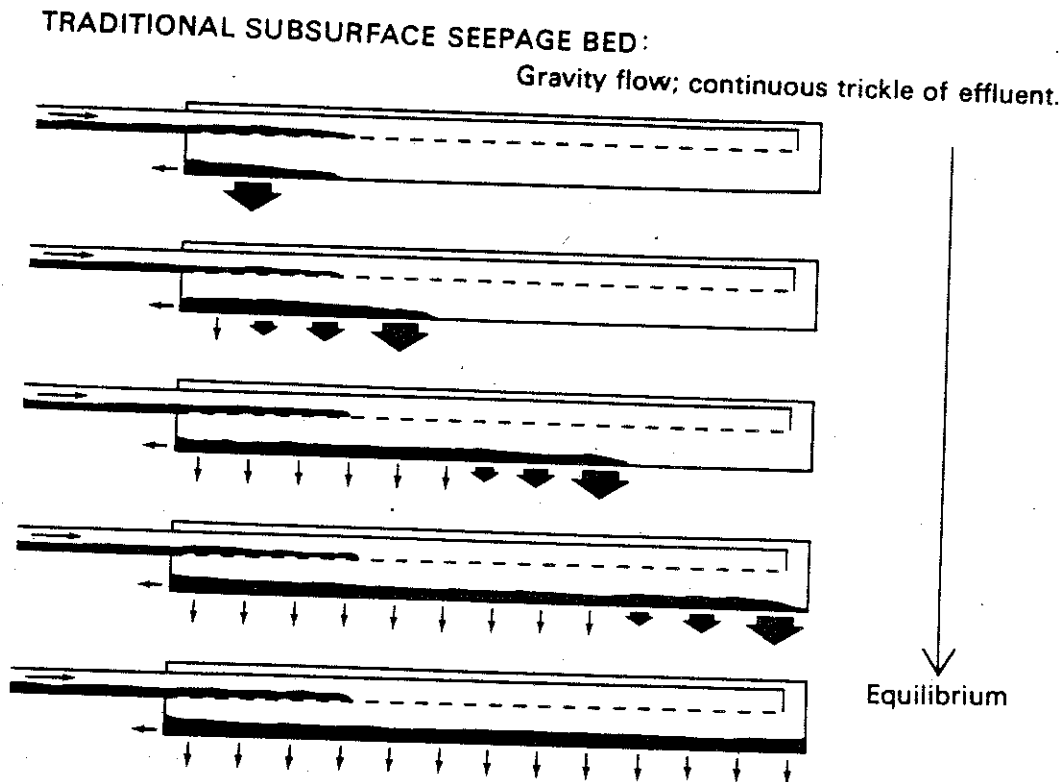


Figure 17. Progressive clogging of the infiltrative surfaces of subsurface absorption systems (11)

Dosing Periodic dosing of large volumes of effluent onto the field improve distribution and provides an opportunity for the soil to drain between applications. Drainage exposes the infiltrative surface to air, reducing clogging (16, 46, 62). However, even with dosing, the effluent is not distributed over the entire infiltrative surface if the 4-inch pipe is used (63).

Pressure Distribution Pressure systems help provide uniform

distribution. Networks of small diameter pipe with small holes are designed such that the entire pipe network fills before much liquid passes out the holes, thereby achieving uniform distribution (61-63). These systems combine uniform distribution with dosing enhancing purification and reducing clogging.

Proper loading of permeable soils to prevent saturated flow is vital to insure purification of the waste effluent. Pressure distribution systems provide this loading control. Conventional gravity distribution is ineffective (61).

Pressure distribution systems also retard clogging. Since the network is designed to apply no more liquid than an area of the absorption bed can absorb each day, the soil remains well aerated. Absorption fields in sand with pressure distribution have shown no evidence of clogging after four years of operation (61) while fields in sand with conventional distribution begin to clog after six months (11). The aerobic environment maintained by pressure systems promotes the growth of microorganisms which destroy clogging materials and appears to attract larger fauna, such as worms, to consume nutrients accumulating at the infiltrative surface. The worm's burrows help break up the clogging zone. Worm activity perhaps explains why an absorption field in a silt loam underlain with glacial fill dosed with pressure distribution at three times the USPHS (45) recommended rate has not clogged after three years of operation (62).

Construction Practices

Probably the most frequent cause of early failure of soil absorption systems is poor construction techniques. Rapid absorption of waste effluent by soil requires maintaining open pores at the infiltrative surface, but often the pores are sealed during construction by compaction, smearing or puddling of the soil by excavating equipment.

Compaction, smearing and puddling occur primarily in soils containing clay. The flat clay particles adhere to each other in dry soil making it hard and very stable to high compressive forces. However, when wet, the clay plates separate when forces are applied to the soil. The water acts as a lubricant as the clay plates move relative to one another to close channels and vugs reducing the permeability of the soil to very low levels.

Not all soils are equally susceptible to this structural destruction. Tendency toward compaction and puddling depends upon the soil type; the moisture content and the applied force. Soils with high clay contents are easily puddled while sands are not affected. However, soils with clay will not puddle if they are only slightly moist. Instead, under pressure, dry clay breaks into small fragments along pedal boundaries rather

than smearing, thereby keeping the large pores open.

Careful construction techniques, following the recommendations below, will minimize this cause of soil clogging (64,65).

1. Work should be done in clayey soils only when the moisture content is low. If the soil forms a "wire" instead of breaking apart when attempting to roll it between the hands, then it is too wet.
2. Excavating equipment should not be driven on the bottom of the system. Trenches rather than bed construction is preferable in clayey soils because equipment can straddle the trench, thus reducing compaction and smearing.
3. Shallow systems should be constructed to place the infiltrative surface in more permeable horizons and to enhance evapotranspiration. This is particularly beneficial in clayey soils because they are generally wetter for longer periods of time, especially at greater depths.
4. Any smeared or compacted surfaces should be removed. Compaction may extend as deep as 8 inches in clays. This requires hand spading to expose a fresh infiltrative surface.
5. Work should be scheduled only when the infiltrative surface can be covered in one day because wind blown silt or raindrop impact can clog the soil.

Modifying the Treated Wastewater Characteristics

While the search for improved methods of on-site disposal has centered largely around the soil absorption system, recently more emphasis has been put on altering the characteristics of the effluent discharged to the soil. Improving the quality of effluents has been purported to enhance soil infiltration, reduce the dependence on soils for final treatment or eliminate the need for soil altogether.

Modifying the Wastewater Source One of the simplest ways to improve the effluent discharged to the soil is to provide changes at the source, either by reducing the total volume of waste discharged or by preventing entry of pollutants into the waste stream.

Flow reduction to produce lower wastewater volumes can be accomplished through water conservation and recycle. Reductions can be achieved through improved water use habits or by simple modifications in water-use appliances or plumbing fixtures. With less wastewater to

Event	No/cap/day	Average size of event	Normal use	With 3 gal/flush	With sudsaver @ 27.86	With 15 gal bath shower	With all three used	Recycle bath/laun. to toilet
Toilet	2.29	3.99	9.16	6.87	9.16	9.16	6.87	0
Laundry	.31	33.49	10.51	10.51	8.64	10.51	8.64	8.64
Bath	.47	21.35	10.00	10.00	10.00	7.05	7.05	7.05
Dishes	.39	12.50	4.86	4.86	4.86	4.86	4.86	4.86
W. Soft.	.03	81.07	2.64	2.64	2.64	2.64	2.64	2.64
Other	--	--	5.43	5.43	5.43	5.43	5.43	5.43
Total (% savings)	--	--	42.60	40.41 (5%)	40.73 (4%)	39.65 (7%)	35.49 (17%)	28.62 (33%)

Table VIII. Average calculated wastewater reductions in eleven rural homes
(All volumes in GPCD) (58)

Event Parameter	Fecal toilet flush	Nonfecal toilet flush	Garbage disposal	Kitchen sink usage	Automatic dish washer	Clothes washer - wash	Clothes washer - rinse	Bath/shower
BOD ₅ U	4340	6380	10900	8340	12600	10800	4010	3090
BOD ₅ F	2340	3980	2570	4580	7840	6970	2840	1870
TOC U	3530	4250	7320	5000	7280	7700	2610	1750
TOC F	1580	3170	3910	4110	4690	5380	1910	1130
TS	10700	17800	25800	13800	18200	37500	10900	4590
TVS	7760	12000	24000	9730	10500	14700	4800	3600
TSS	6240	6280	15800	4110	5270	7930	3040	2260
TVSS	5090	5120	13500	3840	4460	4700	1810	1580
TOT-N	1500	2640	630	420	490	580	150	310
NH ₃ -N	590	520	9.6	32.3	54	19.4	11.4	40
NO ₃ -N	6.3	21.1	.2	1.8	4.1	17	10.3	7.4
TOT-P	270	280	130	420	820	1600	550	36
ORTHO-P	120	190	90	180	380	410	110	20
Temp. °F	66	66	71	80	101	90	83	85
Flow (gal)	4.3	4.3	3.8	4.8	12.0	15.7	14.4	13.0
No./samples	32-40	24-37	4-7	7-11	13-15	24-27	24-28	18-24

Table IX. Mean wastewater contributions from household events, mg/capita/day (59)

treat and dispose of, the life of the on-site disposal would increase.

Nearly 70% of the total wastewater generated in the homes is derived from the toilet, laundry and bath (58). The most substantial water savings can therefore be made in these areas. Low flow toilets, "sudsaver" washing machines, restricted flow shower heads, and recycling of bath and laundry wastes for toilet flushing are four commonly mentioned water saving devices. By reducing the toilet flushing volume to 3 gallons, clothes washing to 28 gallons by using a sudsaver, and baths and showers to 15 gallons, average water use could be reduced to 17% in rural Wisconsin homes (58). Recycling bath and laundry wastes to flush toilets could increase the savings to 33% (see Table VIII). These savings compare well with values from other studies (66, 70).

Waste segregation to eliminate pollutants from the waste stream improves the quality of the wastewater. Analysis of wastewaters generated from various water-use events in rural households suggest which events should be modified for the most beneficial results (59, 60) (see Table IX).

Recently, the concept of segregating toilet wastes (black water) from the other household wastewaters (grey waters) for separate treatment and disposal has drawn attention. Serious questions have been raised by those planning development in water-short areas regarding the use of valuable drinking water to transport body wastes and the practice of co-mingling the black and grey waters prior to on-site treatment and disposal. Segregation of black water from other household wastewater by use of a non-water carriage toilet could conserve water resources and reduce the volume and pollutant load discharged to on-site disposal systems. Daily savings would vary considerably at a given home and between homes. Ranges of recent possible reductions are shown in Table X (58-60, 66).

If the toilet wastes can be segregated and adequately disposed of, attention must then be directed toward the disposal of the grey water. Grey water has been considered to be relatively uncontaminated, compared to black water. However, grey water contains substantial quantities of physical and chemical pollutants as well as pathogenic indicator organisms (see Tables XI and XII).

Pollutant concentrations of grey water are similar to those of black water or combined wastes in rural homes (60). Black water contains high concentrations of suspended solids, nitrogen and bacteria but grey water also contains sufficient quantities of pollutants and pathogenic indicators to cause concern.

To date, little research has been done on treatment and disposal of household grey water. One method is the septic tank-soil absorption system. However, simple alternatives might be more desirable in certain applications. The Small Scale Waste Management Project is evaluating alternative methods for grey water treatment and disposal.

The reductions in flow or waste strength may increase the life of a soil absorption field, but if so, by what factor it is not known. For new installations, it has been suggested that the smaller absorption fields could perhaps be allowed if water saving devices are used. However, unless assurances are made that the flow reduction of waste segregation facilities could not be removed or fail, reduced field sizing is not recommended.

Table X. Effect of toilet waste segregation on household wastewater

Parameter*	% Reduction
Flow	22-31%
BOD ₅	22-49
Suspended solids	36-67
Total Phosphorus, P	14-42
Total Kjeldahl Nitrogen, N	68-99

*Although not shown, there would also be substantial reductions in the quantities of pathogenic organisms.

Modifying Treatment Another method to improve effluent quality is to provide better treatment than the septic tank offers. Of the numerous alternatives available, small extended aeration units and intermittent sand filters seem to be the most promising.

Aerobic treatment is often suggested because the process can produce a higher quality effluent than the septic tank. Many different process designs exist, but extended aeration is most commonly used for small waste flows. Two compartment tanks are typically used, one for aeration and the other settling. Some designs provide a small septic tank ahead of the aeration tank.

The performance of several aerobic units of different designs has been compared to septic tanks operating under both laboratory and field conditions (71-73). Two years of evaluation showed the aerobic units can

Table XI. Pollutant contributions by black and grey wastewater streams¹(105)

Pollutant	Grey				Black			
	Mean %	Range %	Mean g/c/d	Mean mg/l	Mean %	Range %	Mean g/c/d	Mean mg/l
BOD ₅	63	51-80	28.5	255	37	20-49	16.7	280
Suspended solids	39	23-64	17.2	155	61	36-77	27.0	450
Nitrogen, N	18	1-33	1.9	17	82	67-99	8.7	145
Phosphorus, P	70	58-86	2.8	25	30	14-42	1.2	20
Flow	65	53-81	29.4	gal/c/d	35	19-47	15.9	gal/c/d

¹The values shown are based on the results of several studies (60, 66, 70). The results are average values for households with typical conventional appliances, excluding the garbage disposal.

Table XII. Selected bacteriological characteristics of bath and laundry wastewaters (105)

Event	Bacteria	Data Points	Mean ¹ #/100 ml	95% C.I. #/100 ml
Bath/shower	Total coliforms	32	1810	530-6160
	Fecal coliforms	32	1210	330-4410
	Fecal Strept.	32	326	70-1510
Laundry	Total coliforms	41	215	45-1020
	Fecal coliforms	41	107	28-405
	Fecal Strept.	41	77	19-305

¹
Log normalized data.

achieve a higher degree of treatment than septic tanks. Removals of biodegradable organic material from the waste by aerobic units were significantly higher than those achieved by septic tanks, but suspended solids concentrations in all effluents were nearly identical (see Figures 19 and 20 and Table XV). However, the septic tanks were more stable. Periodic upsets resulted in substantial variability in aerobic unit effluent quality.

Effluent Quality and Soil Clogging Improving the effluent quality before discharge to the soil may inhibit clogging. In studies with packed columns of sands, sandy loams and loams, clogging was found to be a function of the sum of the suspended solids and BOD₅ concentrations in the effluent (74). The relationship found was:

$$\text{Adjusted area required} = \text{Area required for standard septic tank system} \times \sqrt[3]{\frac{\text{BOD}_5 + \text{TSS}}{250}}$$

Assuming an average effluent quality of 40 mg/l BOD₅ and 40 mg/l of suspended solids, the calculated adjusted absorption area is approximately two-thirds of the "standard" area.

However, in several other studies (24, 53, 75), investigating effects of effluent quality on soil infiltration, only slight differences in clogging rates were found over a range of qualities tested. Differences in rates of clogging were small in hand-packed lysimeters of sand and sandy loam loaded with septic tank effluent (74 mg/l BOD₅ and 51 mg/l suspended solids) and aerobic unit effluent (81 mg/l BOD₅ and 75 mg/l suspended solids) (75). The aerobic unit effluent produced earlier but less intense clogging in the sand, but the reverse was true in the sandy loam. Upon resting, the soils receiving aerobic unit effluent recovered more quickly. In general, however, there was little difference between the soil clogging characteristics of the two effluents.

In studies with undisturbed cores of Almena silt loam (percolation rates of 70 min/in in the topsoil and 100 min/in in the subsoil) columns were continuously ponded with septic tank effluent, aerobic unit effluent and distilled water (24). The aerobic effluent had a significantly lower biodegradable organic concentration than the septic tank effluent (chemical oxygen demand concentrations of 150 mg/l and 60 mg/l respectively), but the suspended solids concentrations were similar (40 mg/l and 33 mg/l respectively). More severe clogging occurred with the aerobic effluent. No clogging occurred in the soil loaded with distilled water (see Figure 18). It was hypothesized that finely divided suspended solids in the aerobically

treated wastewater were to enter the small pores in the soil which clogged the soil with depth creating a more effective barrier to flow.

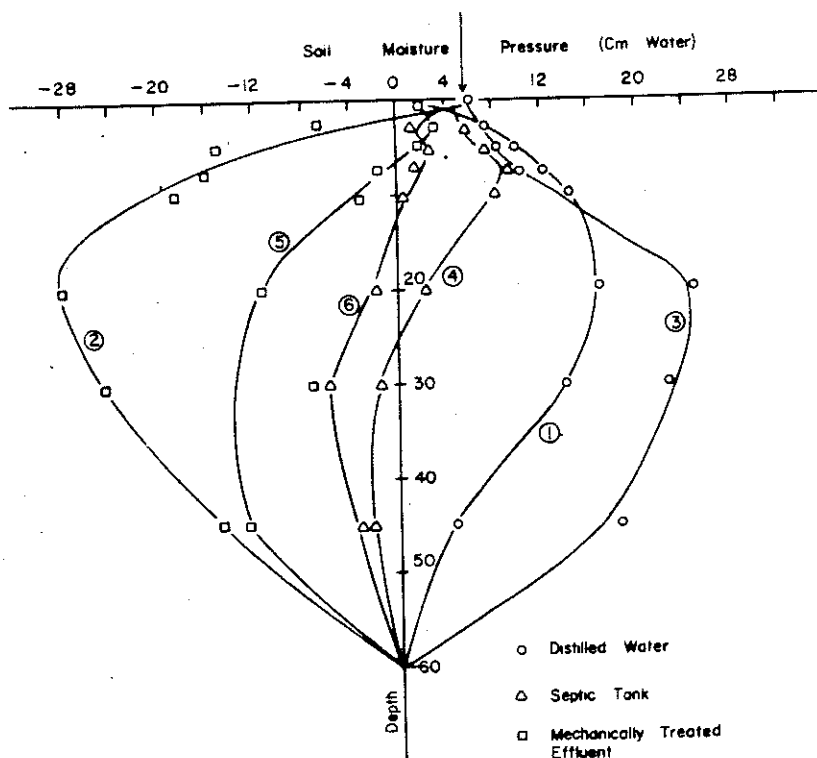


Figure 18. Measured moisture pressures in six columns of Almendra silt loam, ponded with distilled water (nos. 1 and 3), septic tank effluent (nos. 4 and 6) and aerobic unit effluent (nos. 2 and 5) (24)

Subsequent studies designed to test the solids clogging hypothesis indicated that the initial saturated hydraulic conductivity is more significant than effluent quality (104). Undisturbed cores of Almendra silt loam were paired according to their initial saturated hydraulic conductivity, one pair representing a high and low initial K_{sat} . Three sets of four replicates each were dosed with 1 cm/day of septic tank effluent (48 mg/l BOD₅, 27 mg/l TSS), aerobic unit effluent (27 mg/l BOD₅, 61 mg/l TSS), and tap water. The length of time to when each column remained ponded between daily doses was recorded. The aerobic columns showed mean ponding times of 21.3 weeks, the septic tank set 20.6 weeks, and the tap

water 18.3 weeks. When initial K_{sat} values were compared between all three sets the ponding times for the high K_{sat} columns was 28 weeks while the ponding times for the low K_{sat} columns was 14.8 weeks.

These studies to date seem to indicate that, in unstructured soils, such as sands and sandy loams, applied effluent quality may affect the degree of clogging. A similar effect has not been found in finer textured soils.

Restoring the Infiltrative Capacity of a Clogged Absorption Field

Soil absorption systems often fail after several years of satisfactory service because the clogging zone eventually develops to a point where insufficient amounts of effluent pass through it. Methods are being sought to rejuvenate old fields so that failed systems need not be replaced.

Resting One effective method is resting the system (11,14,15,46). Resting allows the absorption field to gradually drain exposing the clogged infiltrative surface to air. After several months, the clogging materials are broken up to physical and biochemical processes, restoring the infiltrative capacity of the system. A second bed must be available to allow continued use of the disposal system while the failed bed is resting. Two beds can be constructed when the disposal system is first installed at the outset, each with 50 to 75 percent of the total absorption area required. The two beds can then be used alternately by diverting the wastewater from one to another every six months. If a system with only one bed has failed and a new one is constructed, provisions should be made such that the old one is not abandoned, but can easily be alternated with the new bed.

Oxidizing Agents The infiltrative surface also can be rejuvenated by the addition of oxidizing agents to the absorption field. The oxidizing agents perform the same function as resting but the clogging zone is destroyed within a day or two rather than several months. Such a method does not necessitate taking the clogged bed out of service which eliminates the need for a second bed.

Laboratory and field tests indicate that chemical oxidation can restore the infiltrative surface to near its original permeability (27). The oxidant preferred is hydrogen peroxide (H_2O_2) because it is effective at the natural pH of absorption fields, produces no noxious byproducts and is inexpensive.

Hydrogen peroxide treatment would best be used in a preventive maintenance program because smaller quantities are cheaper and safer to handle. Routine septic tank pumping could be coupled with absorption field maintenance with peroxide. Five gallons of 50% H_2O_2 solution may

be sufficient to reduce the clogging developing in the bed. Since the field is still permeable, the oxidant can reach the clogging zone easier than in a sealed system. Tank pumping and peroxide treatment would best be performed while the system is not in use, for example, during a vacation, to give the reagent time to work without being diluted with peak effluent, and to allow aerobic conditions to become well established in the bed. Evaluation of preventive maintenance with hydrogen peroxide is in progress.

ALTERNATIVE SYSTEMS FOR PROBLEMS SOILS

There are many areas where the conventional septic tank-soil absorption field is not a suitable system of wastewater disposal. Sites with very slowly permeable soils, excessively permeable soils, or soils over shallow bedrock or high groundwater, for example, are simply not suited for the conventional system. However, alternate systems can be used which still utilize the capabilities of soil to absorb and purify wastewater (76).

Slowly Permeable Soils

Slowly permeable soils constitute a major group of problem soils. Soils with percolation rates faster than 120 min/in often have seasonal perched water tables within 2 feet of the ground surface, especially during the spring and fall. Infiltrating surface water during these wet periods is unable to percolate through the subsoil fast enough and flooding occurs from lateral movement of water through the topsoil from higher elevations. Such conditions are not suitable for conventional soil absorption systems.

To overcome these conditions, one alternative is to raise the absorption field above the natural soil by building the seepage system in a mound of medium sand (77). This raises the seepage system above the wet slowly permeable subsoil and places it in a dry permeable sand (see Figure 19). There are several advantages to this. First, the percolating liquid enters the more permeable natural topsoil over a large area and can safely move out laterally until absorbed by the less permeable subsoil. Second, the clogging zone that eventually develops at the bottom of the gravel trench within the mound will not clog the sandy fill to the degree it would in the natural soil. Finally, smearing and compaction of the wet subsoil is avoided, since excavation in the natural soil is not necessary.

The design of the mound is based upon the expected daily wastewater volume it will receive and the natural soil characteristics. It must be sized such that it can accept the daily wastewater flow without surface seepage when perched water exists in the natural soil in the spring and fall, as well as when the water table is lower during the summer and winter. Size and spacing of the seepage trenches is important to avoid liquid from rising into the fill below the trenches when the water table is high. In addition, the total effective basal area of the mound must be sufficiently large to conduct the effluent into the underlying soil.

A clean, medium sand is used as the fill material in construction of

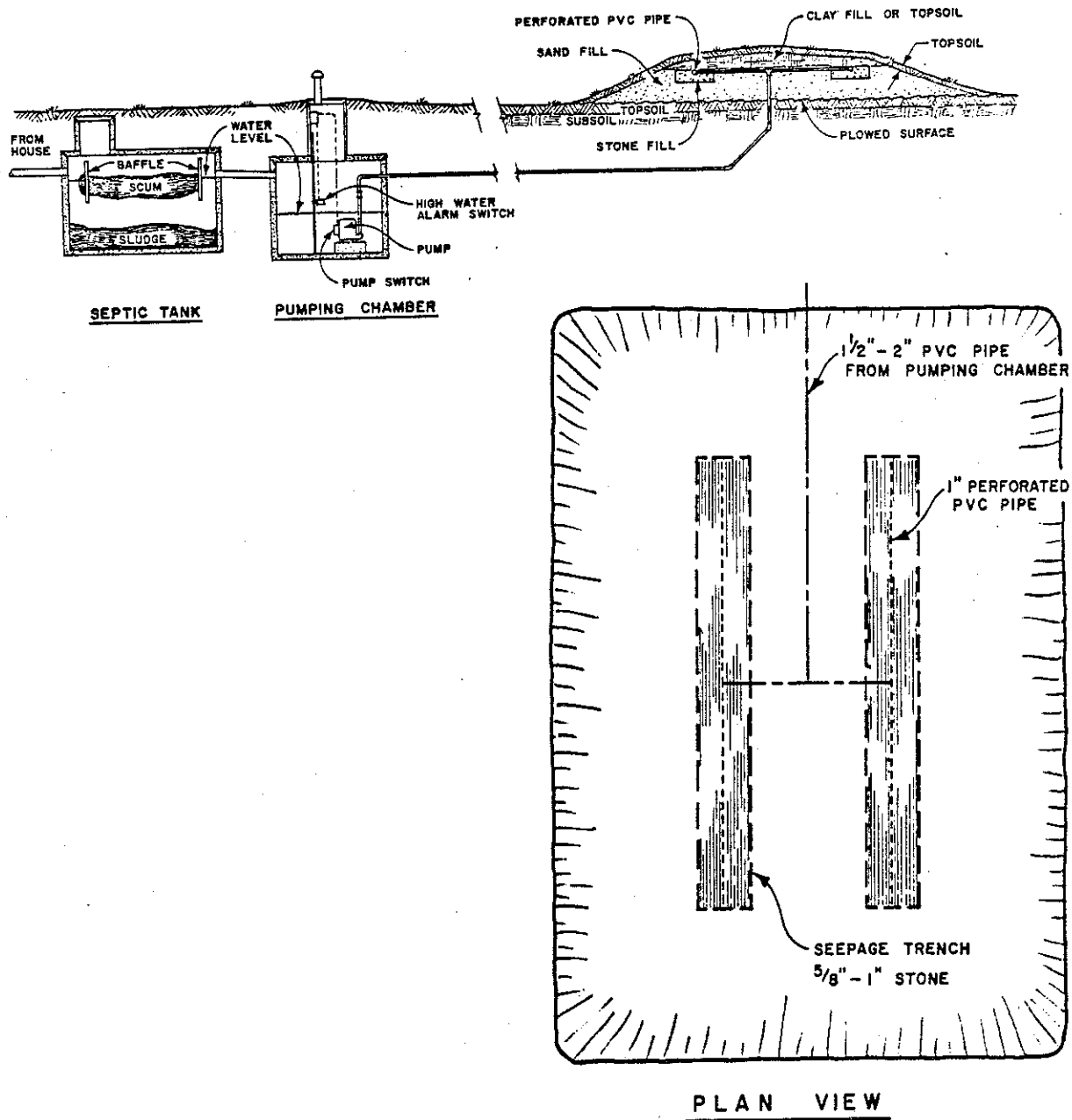


Figure 19. A plan view and cross-section of a mound system for problem soils

the mound and the gravel trenches constructed within consist of 1 - 1-1/2 inch stone. As in any seepage trench, a clogging mat will develop at its bottom. The ultimate infiltration rate through this zone has been shown to be 5 cm/day (28). Therefore, one consideration must be to insure that sufficient trench bottom area is available for the design flow.

If more than one trench is included, another consideration is the spacing between trenches. The area between trenches must be long enough for the underlying natural soil to absorb all the liquid contributed by the upslope trench. Infiltration rates into the natural soil is based on the hydraulic conductivity characteristics of the least permeable soil horizon below the proposed site. The basal area required for the mound is based on this as well.

To distribute the wastewater to each of the trenches a pressure distribution network is used. This provides uniform application which is necessary to prevent local overloading and eventual surface seepage.

Mound systems installed in these soils have been monitored since 1972 and are performing satisfactorily (78). However, application of proper siting criteria design and construction techniques which have been described in detail (77) are critical for satisfactory performance. Not all sites are acceptable for the mound design.

Shallow, Permeable Soils Over Creviced or Porous Bedrock

Shallow, permeable soils over creviced or porous bedrock constitute a major group of problem soils because inadequate soil is available to purify the percolating waste before it reaches the porous bedrock which leads directly to the groundwater. To overcome these limitations, the absorption field can be raised above the natural soil by using the mound system (see Figure 19). This increases the amount of soil available for percolation and with uniform application of effluent, purification will be adequate by the time the percolating effluent reaches groundwater (31, 35, 79, 80). However, nitrates will not be removed.

The design of the mound follows the same procedures as described for the mound in slowly permeable soils. However, the seepage system within the fill may have nearly any shape desired, since the permeability of the natural soil is not a limiting factor. A bed is usually more suitable than trenches. Detailed site criteria design and construction procedures are described elsewhere (81) and should be followed to assume proper operation of the mound.

Permeable Soils With Seasonally High Groundwater

Mound Systems Homes should not be built in areas with a permanently high groundwater table. However, in some areas, homes are built where the water table is high only occasionally during the year. During high water table periods, a conventional septic tank-soil absorption system cannot function properly due to flooding of the system and improper purification. A properly designed and constructed mound system provides sufficient unsaturated distance for purification before the effluent reaches the groundwater (Figure 19). The design of the mound follows the same procedures as described for the mound in slowly permeable soils but the seepage system within the mound is usually a rectangular bed. Normally the permeability of the natural soil is not a limiting factor but the mound must be designed to prevent the intrusion of the perched water table to the base of the mound. Detailed site criteria, design, and construction procedures are described elsewhere (82) and should be followed to assume proper performance of the system.

Curtain or Underdrain Systems Conventional subsurface trenches can be constructed where periodic high water tables are a problem if the natural soil is drained. Agricultural drain tile is used to lower the water table and to discharge the water to the ground surface. Careful placement of the drain is necessary to insure a sufficient depth of unsaturated soil is maintained for purifying the wastewater to avoid short circuiting (83,84). Systems of this type presently are being evaluated.

There are disadvantages to these alternate systems. Construction of mound systems depends upon a source of suitable fill material and relatively large lots. Mounds cost \$2500 to \$3500 or more to construct depending on hauling costs. Underdrain systems may be cheaper but systems not dependent on soil for disposal sometimes may be more desirable.

SYSTEMS NOT DEPENDENT ON SOIL AND SITE CONDITIONS

At some sites, the soils may be totally inadequate as a treatment and disposal medium as the lot may be too small to accommodate a proper absorption system. In such instances, on-site wastewater disposal systems not dependent upon soil disposal but which discharge the treated wastewater to surface waters or the atmosphere, are necessary.

Systems Discharging to Surface Waters

There are numerous alternative treatment processes currently available to treat small flow wastewaters. To make a proper decision on these alternatives, the homeowner must examine both the in-house wastewater modification processes as discussed earlier, as well as treatment options which might best meet local water quality objectives. A general outline of these alternatives is presented in Table XIII.

Table XIII

In-house

- Water conservation-flow control, reuse
- Waste segregation - non-water carriage toilets,
low flow toilets

Biological processes

- Aerobic-suspended growth, fixed media, emergent
vegetation
- Anaerobic-septic tanks, fixed media systems

Physical-Chemical Processes

- Filtration
- Ion exchange
- Adsorption
- Chemical flocculation/coagulation
- Disinfection-halogens, ultraviolet, ozone

Land application

- Soil absorption
- Irrigation
- Lagoons (absorption)
- Evapotranspiration

Systems designed to discharge treated wastewaters to surface waters must be capable of producing a high quality effluent. The U.S. Environmental Protection Agency has currently set concentration maximums of 30 mg/l BOD₅ and 30 mg/l suspended solids for municipal treatment plants which discharge to water courses. Lower maximums might be set for scattered individual systems, discharging to small intermittent streams. It might be expected that bacteria counts in effluents from individual systems could not exceed the maximums of total and fecal coliforms of 1000/100 mls and 200/100 mls respectively, recommended for recreational waters (85). In addition, limitations on the nutrients, nitrogen and phosphorus, will likely be required for discharges to lakes or impoundments.

Aerobic Processes Although a number of the options listed in Table XIII have been field evaluated, many others are untried in the context of small flows application. Currently, aerobic treatment processes have received the greatest attention as an alternative to the septic tank. Over 75 years of experience with this biological process in larger scale applications makes it a logical candidate for small flow on-site treatment.

Some of the first controlled aerobic processes for household wastewater treatment were small trickling filters. Nichols (86) described aerated pebble filters following septic tanks and Frank and Rhymus (87), in one of the first large research projects on household wastewater disposal, detailed the construction and operation of lath trickling filters. Relatively little experience has been currently reported with fixed media biological filters, but the potential of synthetic media filters in either the stationary or rotary mode is most promising. The most recent operational experience by SSWMP with a rotating biological contactor has not been good owing primarily to shaft breakage in both laboratory and field units (88). On the other hand, an anaerobic-submerged media system, initially designed for ships, has proven to be very stable and relatively maintenance free (88).

The first notable research on the adaptation of the activated sludge process (extended aeration) to household use was conducted at Purdue University in the early 1950's (89). A very simple prototype receiving toilet wastes only produced an effluent with average BOD₅ and suspended solids concentrations of 28 mg/l and 42 mg/l respectively. Similar studies were conducted at Ohio State University employing a proprietary extended aeration package plant. Average BOD₅ and suspended solids concentrations of 24 mg/l and 43 mg/l respectively were reported during a 23-month period with relatively trouble-free service (90).

In 1970, the National Sanitation Foundation (NSF) issued its "Standard 40", pertaining to individual aerobic treatment units (91). This standard outlined criteria for the evaluation of the units and presented a procedure

by which they would be tested and certified. Several states now require NSF certification as a prerequisite for approval of aerobic units.

In addition to controlled studies of aerobic units, there are several reports as to how they function under actual conditions (72, 92, 93, 94). Although some information on effluent quality exists, probably the most valuable information is that on operation and maintenance.

Bennett and Linstedt reported on results from several homeowner operated field units in Colorado (92). Their results indicated a mean value for BOD₅ and suspended solids of 150 mg/l. They cited lack of proper maintenance and an adverse effect of surge flows for this poor performance. Voell and Vance (93) provided data on a large number of field operated aerobic units in a New York county. Average values for BOD₅ and suspended solids were about 90 mg/l. The lack of proper maintenance was again cited for this performance. Six different aerobic treatment units were installed and monitored over an eight-month period by Glasser (94). Average values for BOD₅ and suspended solids were reported to be 48 and 85 mg/l respectively. Glasser recommended maintenance and supervision no less than four times per year.

The performance of several aerobic units of different designs have been compared to septic tanks operating under both laboratory and field conditions (72, 95). Two years of evaluation showed the aerobic units can achieve a higher degree of treatment than septic tanks. Removals of biodegradable organic material from the waste by aerobic units were significantly higher than those achieved by septic tanks, but suspended solids concentrations in all effluents were nearly identical (see Figures 20 and 21 and Table XIV). However, the septic tanks were more stable. Periodic upsets resulted in substantial variability in aerobic unit effluent quality.

Periodic carryover of solids were the major reasons for effluent quality deterioration from aerobic units. Bulking sludge (sludge that will not settle), toxic chemical additions from the home and instability due to excessive buildups of sludge, seemed to be most common causes of carry-over. Several design modifications have been suggested to help prevent some of the operational problems but regular servicing is necessary to insure proper functioning (72). Inspections should be made at least once every two months and excess solids pumped every eight to twelve months (72, 95).

Intermittent Granular Filtration Field experience to date with on-site aerobic treatment processes has indicated that additional polishing of effluents will be necessary prior to surface discharge in order to meet current EPA effluent guidelines.

Treatment ment (Sys- tems)	BOD ₅ (mg/l)			COD (Unfiltered) (mg/l)			COD (Filtered) (mg/l)			TSS ¹ (mg/l)			Fecal Coliforms ¹ (no./ml)			Fecal Strep ¹ (no./ml)		
	Mean	95% Conf. Int.	Coef. of Var.	Mean	95% Conf. Int.	Coef. of Var.	Mean	95% Conf. Int.	Coef. of Var.	Mean	95% Conf. Int.	Coef. of Var.	Mean	95% Conf. Int.	Coef. of Var.	Mean	95% Conf. Int.	Coef. of Var.
Aerobic Units (C, G, H)	47	38-57	136 (69)	136	121-150	75 (60)	65-84	53 (69)	45-64	107 (67)	74-153	32.9 (70)	22.7- 47.6					
	0.79	0-208	0.45	26-349	0.52	7-210	0.19	4-252	0.32	1.0- 2200		0.44	1.0- 1930					
Septic Tanks (A, B, C, D, F)	158	142-174	360 (97)	285	264-306	54 (93)	47-62	4210 (94)	20.1- 72.4			38.2 (97)	0.87	0.0- 11,200				
	0.50	20-480	0.36	66-780	0.36	47-531	0.17	11-695	0.22	5-180,000								

Treat-
ment
(Sys-
tems)

Total Nitrogen (mg-N/l)	Ammonium-N (mg-N/l)			Nitrite-Nitrate-N (mg-N/l)			Total Phosphorus (mg-P/l)			Orthophosphate (mg-P/l)		
	Mean	95% Conf. Int.	Coef. of Var.	Mean	95% Conf. Int.	Coef. of Var.	Mean	95% Conf. Int.	Coef. of Var.	Mean	95% Conf. Int.	Coef. of Var.
Aerobic Units (C, G, H)	37.6 (38)	33.2- 42.0	0.02 (44)	30.1 (46)	25.5- 34.8	35.2 (36)	27.0- 43.5	28.9 (32)	24.5- 33.3			
	0.36	15.8-77.6	1.10 0.68	0.52 71.6	0.3- 0.79	0.69 140	6.8- 140	0.42 51.2				
Septic Tanks (A, B, C, D, F)	55.3 (53)	48.9- 61.6	38.7 (63)	0.56 43.0	0.79- 0.82	14.6 (54)	11.4- 17.7	11.5 (43)	10.2- 12.8			
	0.42	9.7- 124.9	0.45 90.7	2.63 74.5	0.0- 0.80	3.8- 90	2.8- 20.4					

¹ No. of samples

² Log-normal distribution

Table XIV. Comparison of septic tank and aerobic unit effluent characteristics (71, 72)

Filtration appears to be one of the most promising alternatives currently available to provide this polishing step. Whether the filtration is provided by granular beds or by mechanical filter systems employed as a part of the biological process or as a separate process, depends upon economics, effectiveness and maintenance requirements.

Granular filtration appears to be particularly well suited to on-site system design. At least two basic flow configurations have been successfully tested in the field; the recirculating sand filter and the intermittent sand filter.

The recirculating sand filter system consists of a septic tank, a recirculation tank, and an open sand filter (96). Wastewater is dosed on to the filter by a submersible pump located in the recirculation tank. The sump pump is actuated by a time clock and is sized to pump between 5 to 10 gallons per minute for single households. A recirculation ratio of 4:1 (recycle to forward flow) is recommended. The recirculation tank, normally the same size as the septic tank, receives flow from the septic tank and the recirculated portion of the filter effluent. Baffles provide proper mixing of the septic tank and filter effluents prior to recycle. Filter effluent recycle flow is controlled by a rubber float valve located in the filter effluent return line. When the recirculation tank is filled, filter effluent is discharged from the system.

The filter bed consists of 3 feet of coarse filter sand with a desired effective size of 0.6 to 1.5 mm and a uniformity coefficient of less than 2.5. Approximately 12 inches of graded gravel support the sand and surround the underdrain system. The filter is designed to operate at a flow rate of 3 gallons per day per square foot based on raw septic tank flow. It is estimated that approximately 1 inch of sand should be removed once per year in order to avoid serious ponding conditions. After 12 inches of sand have been removed, new sand would be added.

Results for a household system indicate that effluent BOD₅ values average less than 5 mg/l and TSS values less than 6 mg/l (96).

In the intermittent granular filter, pretreated wastewater is applied over a 2 to 3 foot deep bed of sand and the filtrate collected by underdrains. The sand remains aerobic and serves as a biological filter, removing not only suspended solids, but also dissolved organics. A summary of filter performance based upon a review of the literature is given in Figure 22 (97).

Filters receiving septic tank or aerobic unit effluent have been tested under field and laboratory conditions. A typical filter system is

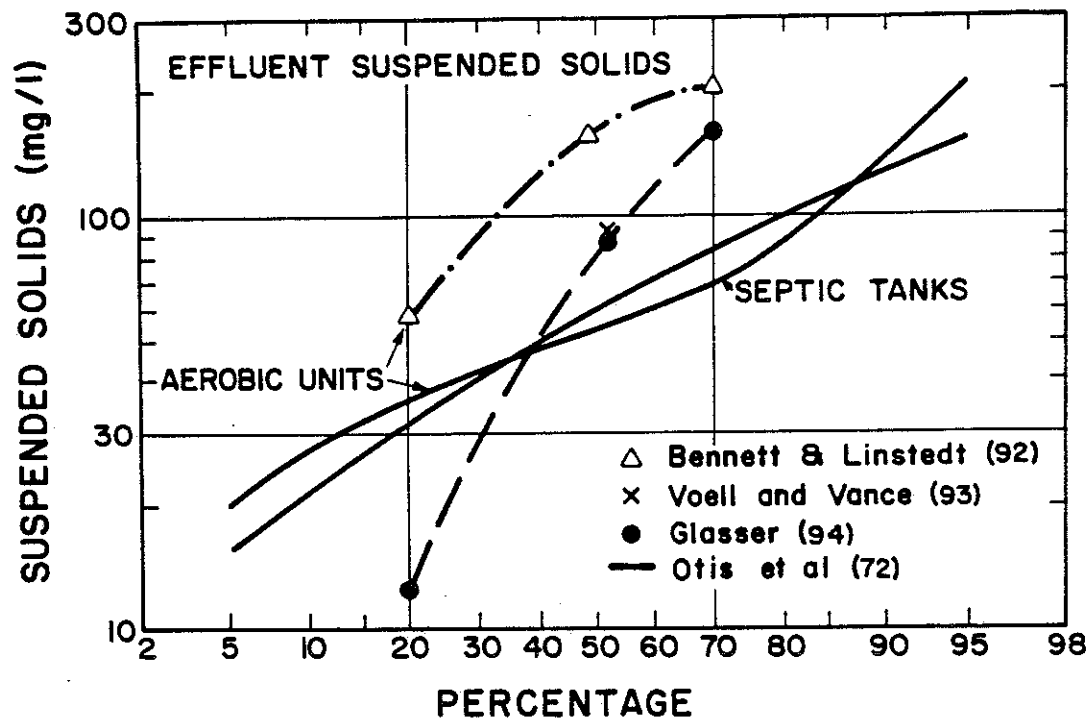


Figure 20. Comparison of septic tank and aerobic unit effluent concentrations of suspended solids

depicted in Figure 23. Of major concern in sizing intermittent granular filters are the trade-offs between effluent quality and maintenance requirements as depicted in Figure 22.

Effluent quality of sand filtered septic tank and aerobic unit effluents appear in Tables XV and XVI for field systems operated for over two years (98,99). It may be noted that relatively little difference is shown between aerobic unit-sand filter effluent and septic tank-sand filter effluent for comparable loading conditions although the aerobic unit system employed a finer sand (0.19 mm as compared with 0.45 mm). It is apparent that effluents from this filter could meet current EPA standards for BOD and TSS, but would require further pretreatment for coliforms or phosphorus. Excellent ammonia conversion is also produced by both systems.

Table XV. Septic tank-sand filter effluent quality data

	Septic tank effluent	Sand filter effluent	Chlorinated effluent
BOD (mg/l)	123	9	3
TSS (mg/l)	48	6	6
Total Nitrogen-N (mg/l)	23.9	24.5	19.9
Ammonia -N (mg/l)	19.2	1.0	1.6
Nitrate-N (mg/l)	0.3	20.0	18.9
Total Phosphorus-P (mg/l)	10.2	9.0	8.4
Orthophosphate -P (mg/l)	8.7	7.0	7.9
Fecal coliforms (#/100 ml)	5.9×10^5	6.5×10^3	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).
Effective size--0.45 mm; uniformity coefficient--3.0.

Filter runs are dependent upon grain size, hydraulic loading, influent organic strength, and maintenance techniques. There is apparently a substantial difference in clogging mechanisms in septic tank effluent loaded filters and aerobic unit loaded filters (88, 98, 99). Recommended filter operation schedules for a septic tank-sand filter system are presented in Table XVII. It is recommended that two filters be employed in an alternating mode, each designed for a hydraulic loading rate of 5 gal/day/ft². When one filter becomes ponded, it is taken out of service, raked to a depth of 2 to 4 inches, and rested prior to reapplication of wastewater. After a second loading period, the top 4 inches of sand from that filter should be replaced with clean sand.

Aerobic unit-sand filter systems do not normally require a second filter (99). An application rate of 5 gal/day/ft² is suggested with a six-month maintenance interval. Removal of the solids mat, along with 1 inch of sand, and replacement with 1 inch of clean sand is the only required

Table XVI. 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
Total Nitrogen -N (mg/l)	39.1	37.5	38.3
Ammonia -N (mg/l)	0.4	0.3	0.4
Nitrate -N (mg/l)	33.8	36.8	37.6
Total Phosphorus -P (mg/l)	31.8	23.1	24.0
Orthophosphate -P (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).
 Effective size--0.19 mm.
 Uniformity coefficient--3.31.

Table XVII. Septic tank-sand filter operation schedule

Sand		Loading and resting period (months)
Effective size (mm)	Uniformity coefficient	
0.2	3-4	1
0.4	3	3
0.6	1.4	5

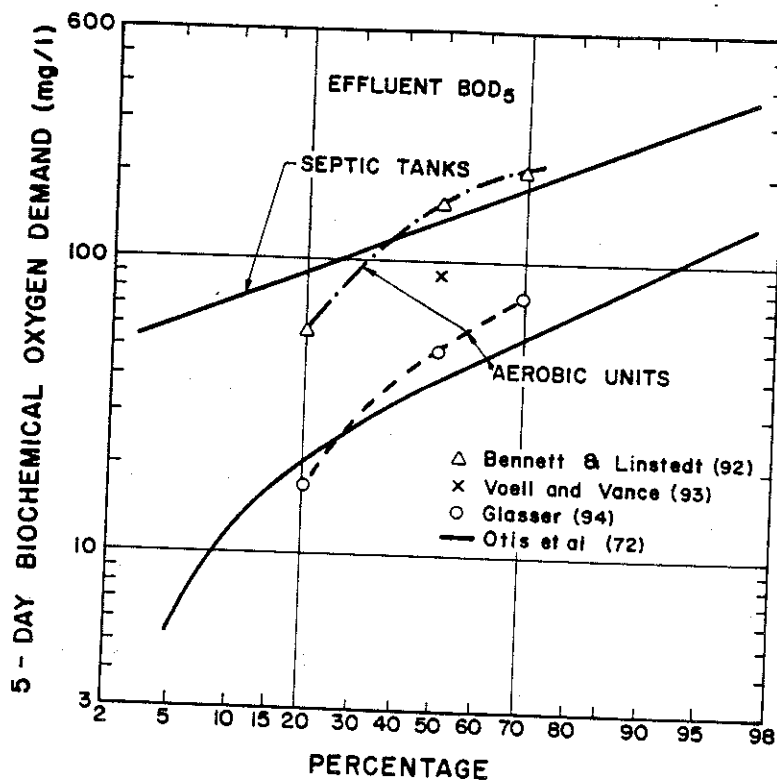


Figure 21. Comparison of septic tank and aerobic effluent concentrations of biodegradable organic material

maintenance step. Reapplication of wastewater is possible immediately after maintenance is performed. Experience has shown that periodic biological and hydraulic upsets of the biological process can be assimilated by the sand filters, however, extended periods of upset will lead to shorter filter runs.

Disinfection Alternatives Where disinfection of final effluent is required, several alternative systems have proven to be effective. The use of dry feed chlorinators will normally produce effluents which will meet current EPA standards (Tables XV and XVI). Unfortunately, a major problem associated with the use of dry feed chlorinators is the lack of control of dose to the wastewater. Periodic high chlorine concentrations were determined over an extended field testing program (97). Methods to

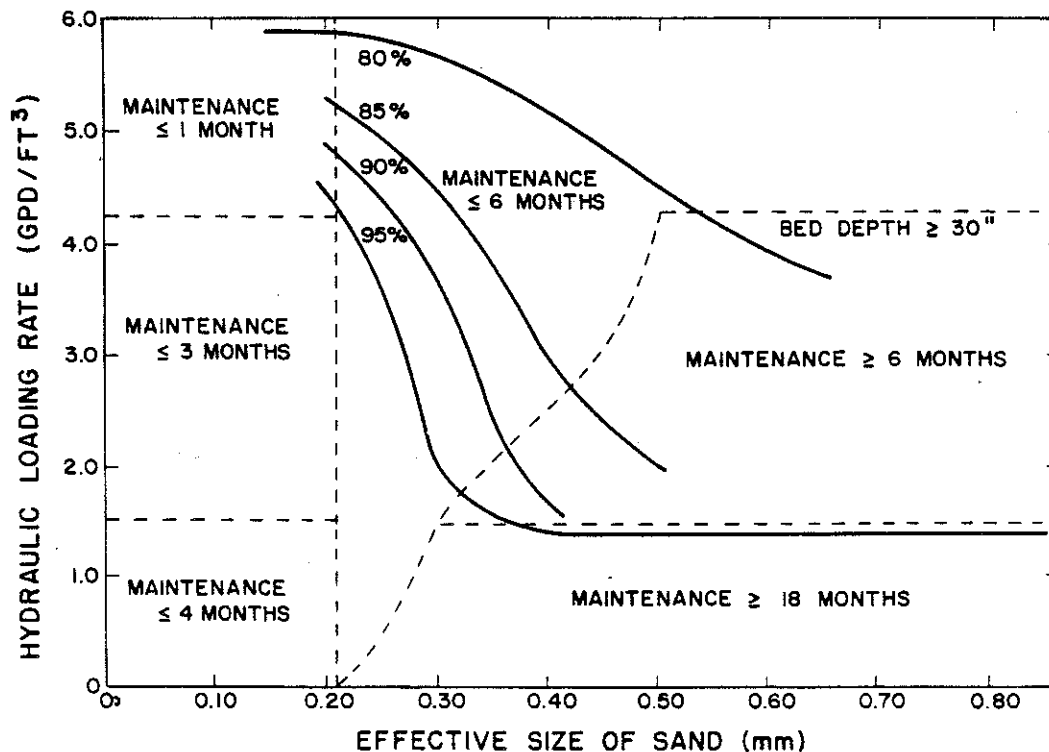


Figure 22. Trends of percent BOD reduction and required maintenance of literature sand filters treating septic tank wastewater (97) (BOD (ave.) of septic tank wastewater -- 94 mg/l)

more effectively control hypochlorite feed are available, however (88). In light of the toxicity of chlorine, consideration must also be given to dechlorination of effluent prior to final surface water discharge.

Initial studies with ultraviolet irradiation of sand filtered household effluents have proven to be most promising. Four months of operating data with a commercially available UV unit are presented in Table XVIII (88). Long-term tests are continuing with these units in several field installations. One major drawback to UV irradiation is the high initial capital investment. As greater demand for this type of system increases, costs will likely decrease, however.

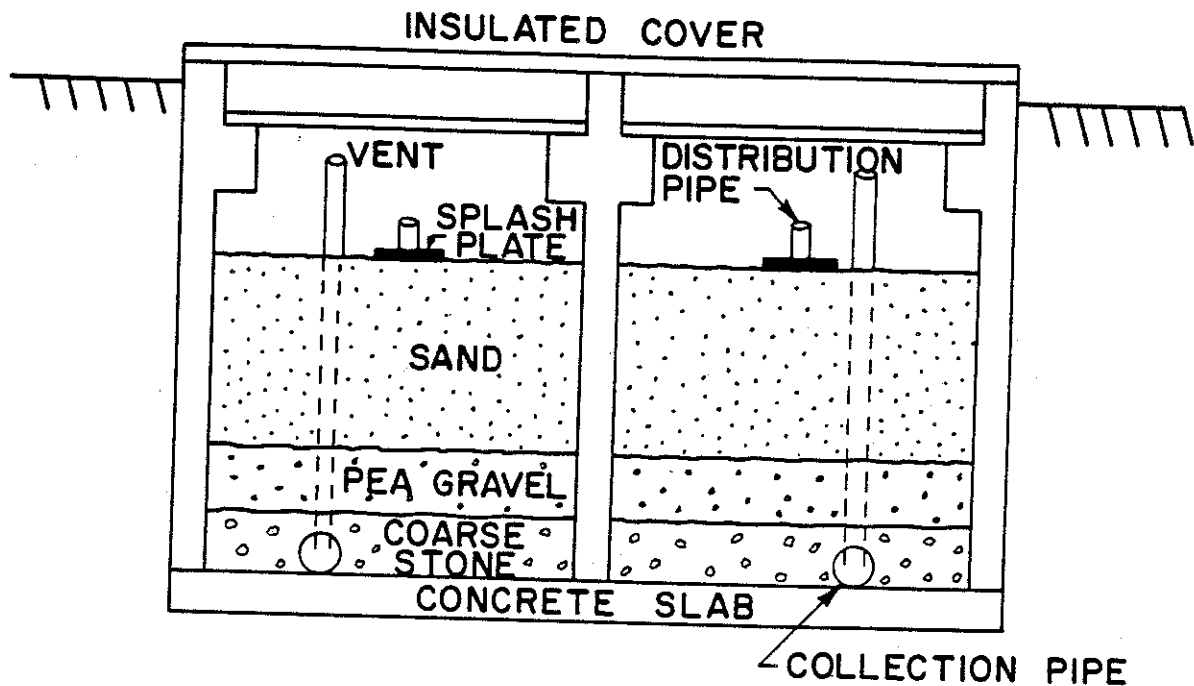


Figure 23. Profile of intermittent sand filter

Other alternative methods of disinfection include iodine, bromine salts, formaldehyde and ozone. Experience with iodine disinfection using an iodine saturator has been excellent, but iodine costs are high (88). Feeding of bromine salts appears to be too complex for small flows application and ozone treatment also involves relatively complex equipment. Little experience has yet been gained with formaldehyde feed equipment.

Other Treatment Processes A number of other unit processes are available for small flow application, but field experience is very limited. Chemical feed equipment is available but maintenance is relatively high (95). The use of ion exchange and carbon adsorption techniques is within the realm of practicality but maintenance and operation requirements are high as are costs. Currently, treatment "packages" employing a number of these unit operations are being field tested at the University of Wisconsin on both combined and grey water effluents from households.

Table XVIII. Coliform analysis of sand filter and UV water purifier unit effluent

	Aerobic unit - sand filter effluent	UV effluent	Septic tank sand filter effluent	UV effluent
Fecal coliform (#/100 ml)	11 - 13	< 1	$(2.6-4.4) \times 10^3$	< 1
Total coliform (#/100 ml)	64 - 75	< 1	$(3.6-5.1) \times 10^3$	< 1

Costs of Surface Water Discharge Systems Costs of the two systems described above are largely dependent upon the volume of wastewater to be treated, the availability of quality filter sand and the amount of maintenance required by the system. A cost analysis involving the application of septic tank effluent and aerobic unit effluent onto sand filters has been performed (Table XIX) (99). Assumptions in the analysis include a three bedroom home, a family size of five, wastewater production of 50 gal/cap/day ($0.19 \text{ m}^3/\text{cap}/\text{day}$) and the availability of a sand with effective size $\approx 0.4 \text{ mm}$ 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 XIX suggest that the two alternatives examined have similar, albeit high costs, when compared with septic tank-soil absorption fields. These costs would likely be reduced somewhat if water conservation was practiced. 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 average.

Evapotranspiration Systems

Evapotranspiration (ET) may provide a means of wastewater disposal in some localities where site conditions preclude soil absorption. Evaporation of moisture either from the soil surface or by transpiration by plants is the mechanism of ultimate disposal. Thus, in areas where the annual evaporation rate equals or exceeds the rate of annual moisture addition

Table XIX. 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 @ 4¢/kwhr	60/yr
Wet well pumping chamber	
Equipment and installation cost	250-350
Operation cost, $\frac{1}{4}$ kwhr/day @ 4¢/kwhr	4/yr
Sand filter	
Equipment and installation cost	10 - 15 /sq ft
Maintenance cost	1 /sq ft/yr
Chlorination and settling chamber	
Equipment and installation cost	700-1000
Operation cost* (chemical)	40/yr
Ultraviolet irradiation unit	
Equipment and installation cost	1100-1500
Operation cost, $1\frac{1}{2}$ kwhr/day @ 4¢/kwhr	20/yr
Maintenance cost, cleaning and lamp replacement	*

*Does not include pump replacement.

*Undetermined.

from rainfall or wastewater application. ET systems can provide a simple means of liquid disposal without danger of surface or ground-water contamination. ET systems can also be designed to supplement soil absorption in slowly permeable soils.

If evaporation is to occur continuously, three conditions must be

met (100). First, there must be a continuous supply of heat to meet the latent heat requirement (approximately 590 cal/gm of water evaporated at 15°C). Second, the vapor pressure in the atmosphere over the evaporative surface must remain lower than the vapor pressure at that surface to create a vapor pressure gradient between the surface and the atmosphere. This gradient is necessary to remove the vapor either by diffusion, convection or both. These two conditions of energy supply and vapor removal are influenced by meteorological factors such as air temperature, humidity, wind velocity radiation and vegetative cover. Energy can also be added from heat in the water itself or from biological activity.

The third condition is that there be a continuous supply of water to the evaporative surface. This depends on the matric potential of the soil and its hydraulic conductivity. The soil material must be fine textured enough to draw the water up from the saturated zone to the surface by capillary action but not so fine as to restrict the rate of flow to the evaporative surface.

When evaporation exceeds water input, therefore, evapotranspiration can be used to dispose of wastewater. Evapotranspiration systems have been designed to evaporate the entire wastewater volume or to supplement absorption in slowly permeable soils.

A typical ET bed system consists of a 1-1/2 to 3 foot depth of selected sand over an impermeable plastic liner (101). A perforated plastic piping system with rock cover is often used to distribute septic tank effluent in the bed. The bed layout may be in a square pattern on relatively flat land or in a series of trenches for sloping topography. A sketch of the cross-section of a typical bed is shown in Figure 24. The surface area of the bed must be large enough such that sufficient evapotranspiration occurs to prevent the water level in the bed from rising to the surface. This requires that the annual evaporation rate must be significantly higher than the annual rainfall.

In order to provide for a maximum evaporation rate, the water in the bed must be raised to the soil surface as rapidly as it is evaporated. This is accomplished by using a uniform sand in the approximate size range of $D_{50} = 0.10$ mm. A sand of this size is capable of raising water a distance of about three feet by capillary action. In this way the surface of the bed is kept moist even though the standing water level within the bed may fluctuate.

Evapotranspiration also is influenced by the vegetation on the disposal field. Trees and bushes with a large silhouette catch more

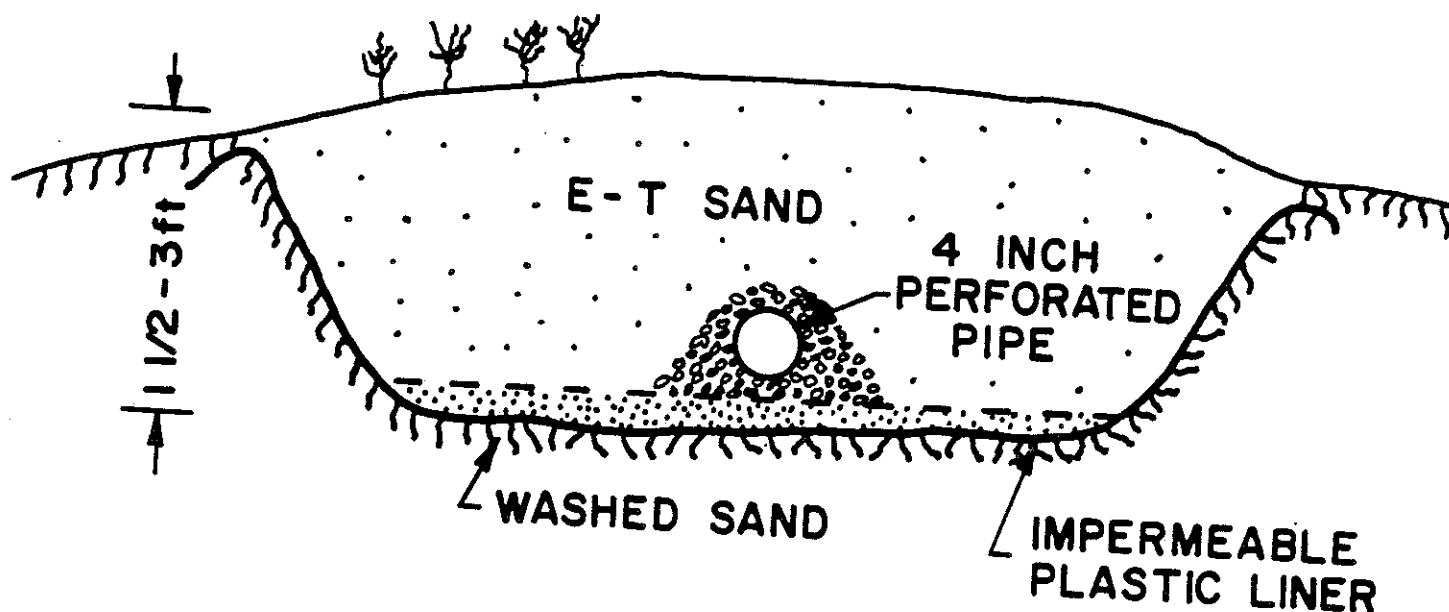


Figure 24. Typical evapotranspiration bed (101)

advected heat, similar to a clothesline. On the other hand, when vegetation is dormant, ET is much reduced. Snow cover reflects solar radiation which reduces ET. In addition, temperatures below freezing require more heat to change frozen water to vapor.

Thus, care must be used in selecting a site suitable for evapotranspiration. A procedure has been outlined to estimate the maximum ET that can be expected from disposal fields for areas in the United States east of the Mississippi River (102).

A study aimed at evaluating the design parameters for non-discharging ET systems is being conducted by the Department of Civil, Environmental and Architectural Engineering at the University of Colorado at Boulder (101). The study involves the use of twenty-eight outdoor lysimeter units, two feet in diameter, and twenty-eight inches deep. Several full-scale ET systems in use at private homes are also being monitored for purposes of data correlation.

The design of an ET bed is based on the annual weather cycle for the location involved. Evaporation rates are highest during the summer months, but the study shows that winter evaporation rates are extremely

important to the application of the system. Summer evaporation has been found to be approximately forty percent of the pan value, while winter values are about seventy percent. The average design evaporation value can be established from the annual pattern as shown in Figure 25. This rate can be matched with the total expected inflow based on household wastewater generation rate and rainfall.

A mass diagram approach is used to establish the storage requirements of the bed (101). Vegetative cover can increase the ET rate during the summer growing season but if this increased rate is to be utilized, additional storage within the bed must be provided for the winter season. Lawn grass has been found to increase evaporation rates slightly during June, July and August, but winter evaporation rates are reduced with respect to bare soil.

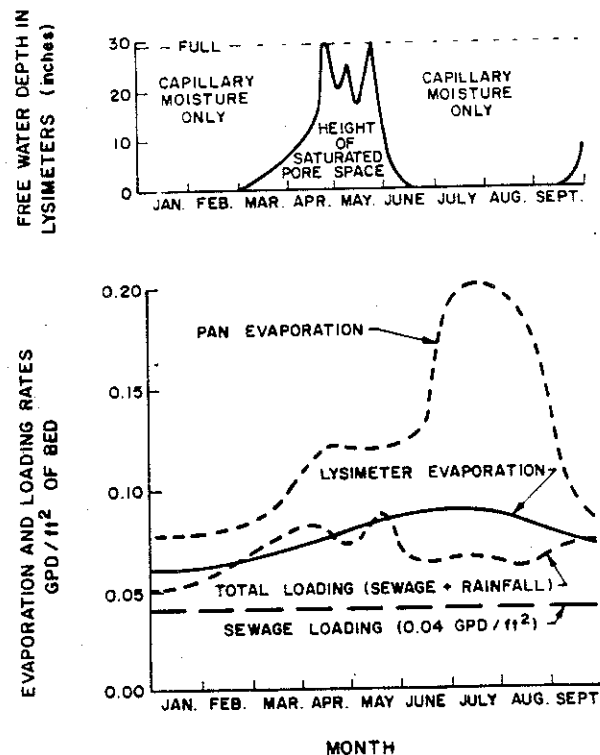


Figure 25. Evapotranspiration bed water balance characteristics

Alfalfa can produce an evaporation rate of 0.6 gpd/ft^2 at the peak of the growing season. Design year round sewage ET rates have been

found to be in the range of 0.04 gallons per day per square foot of bed in the Boulder, Colorado, area (101). This results in a bed area of approximately 5000 square feet for an individual home.

Evapotranspiration can theoretically remove significant volumes of effluent from subsurface disposal systems in late spring, summer, and early fall, particularly if high-silhouette, good transpiring bushes and trees are present. However, the practical application of non-discharging ET bed systems is limited to areas of the country where pan evaporation exceeds rainfall by at least twenty-four inches per year and where winter monthly evaporation is in excess of monthly precipitation by a value of two inches for each and every month. Also, extreme freezing conditions on deep snow cover should not exist where the systems are used. The decrease of ET in winter at middle- and high-latitudes greatly limits ET for winter disposal; under freezing conditions ET would be totally inadequate. Thus, in high latitude, cool-winter locations evapotranspiration cannot be relied upon (102).

Locations for possible application of evapotranspiration disposal systems exist in semi-arid regions of the U.S., including parts of the Southwestern states of Texas, Oklahoma, Colorado, New Mexico, Utah, Arizona, California and Nevada. Even in these areas, household water conservation should always be considered as part of the system (101).

The cost of an ET bed system is relatively high. In-place costs including excavation, suitable sand fill, liner and piping are in the range of 70 to 90¢ per square foot of bed surface (101). When the costs of the septic and piping are included, the total system costs range from \$3000 to \$5000 per house (or about \$1.00/sq ft). Studies are presently underway at the University of Colorado to develop a mechanical wastewater evaporation system that will have a greater range of application throughout the country (101). The increased range of use results from the concept of minimizing the precipitation catchment surface as it relates to the evaporation surface area. If the ratio of evaporation surface area to precipitation catchment area (E/PC) is high enough, precipitation becomes a minor factor in evaluating the utility of these systems. Thus, in locations where evaporation potential is high, but rainfall precludes the use of soil based ET systems, mechanical devices might be applied. The ultimate value of this approach will be dependent on the cost of commercially manufactured units.

The type of unit being evaluated in the prototype studies at the University of Colorado is a multiple, concentric disk system rotating on a common shaft. Preliminary studies are underway to determine optimum rotation speed, disk size and submergence, and disk material and surface characteristics. This work is due to be completed in the fall of 1977 (101).

CAUSES OF ON-SITE DISPOSAL SYSTEM FAILURE

Failure of on-site disposal systems do occur, while the symptoms are usually easy to recognize, the causes are not. A septic tank system passes through several stages of development before it is put into service. The site has been evaluated, a design made, approval granted by the regulatory agency and the system constructed. Following construction, the system becomes the responsibility of the owner who must operate and maintain it. At each stage, errors can be inadvertently made which shorten the life of the system. These can be avoided.

In an investigation of eight systems in silty soils, six major problems were identified which may occur from the time of the initial site evaluation to construction (103). Where one or several of these problems occurred, the result was failure within three years. The six problems were as follows.

1. Poor site evaluation by the installer.
2. Failure of the regulatory agency to reject applications with poor siting or design.
3. Design specifications not followed during construction.
4. Poor construction procedures followed by the installer.
5. Mistakes overlooked during the site inspection by the regulatory agency.
6. System overloading due to increased wastewater volume following installation.

In those systems investigated which were designed and installed through close cooperation with the regulatory agency, soil scientist and installer, early failure was avoided.

In addition to being properly designed and constructed, regular maintenance must be performed if the on-site system is to function satisfactorily. Settled solids accumulate in the septic tank and must be periodically removed or they are washed out into the soil resulting in clogging of the absorption field and subsequent failure. It is recommended that the septic tank be pumped at least every three years. If

a garbage grinder is used, which is not recommended, the tank should be pumped more often.

Reliable methods of site evaluation, system design, installation and management, together with an effective educational and regulatory program, best prevent premature failure of on-site systems.

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