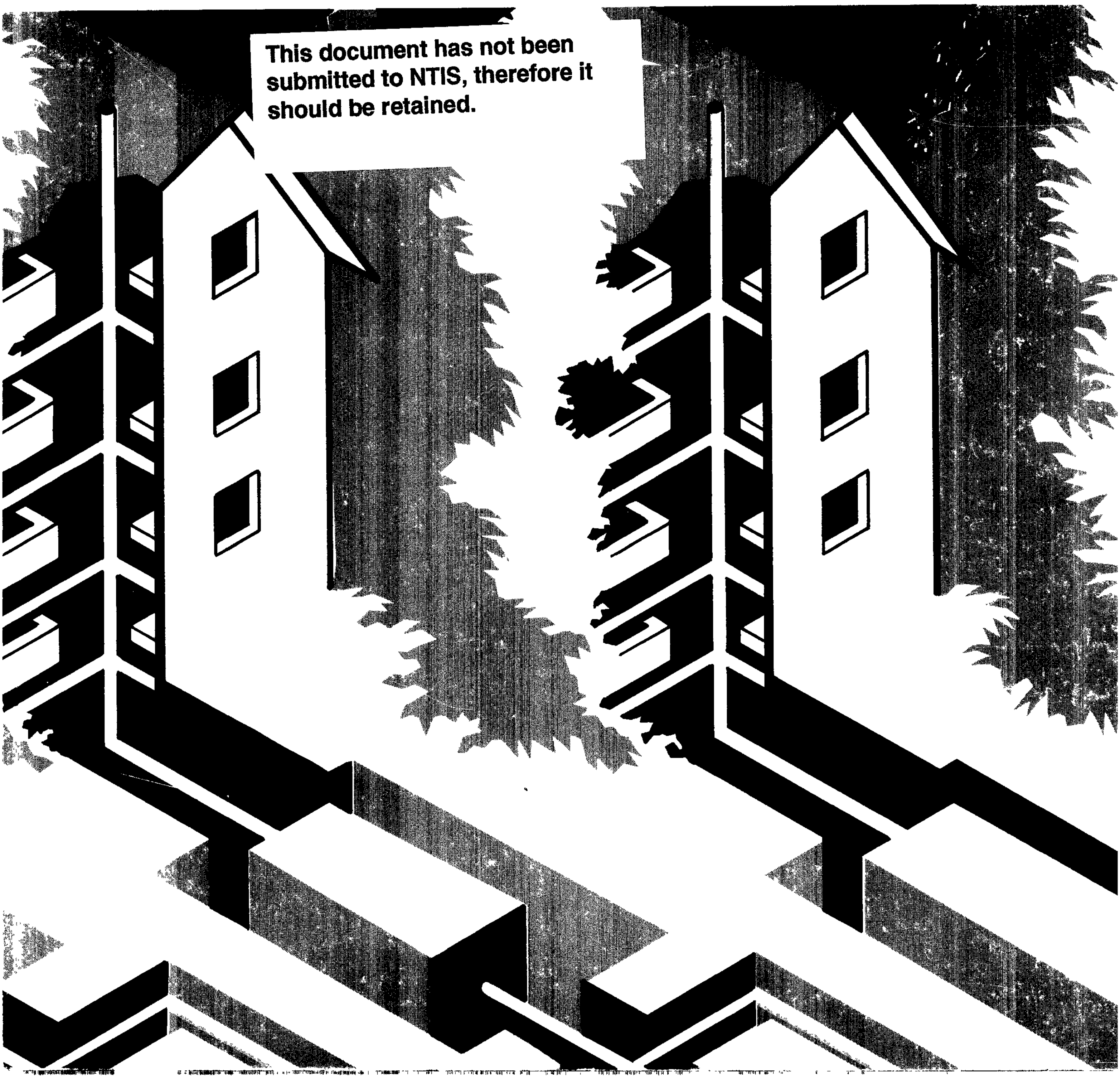
Alternatives for Small Wastewater Treatment Systems

On-Site Disposal/
Septage Treatment and Disposal

EPA Technology Transfer Seminar Publication



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ALTERNATIVES FOR SMALL WASTEWATER TREATMENT SYSTEMS

On-Site Disposal/Septage Treatment and Disposal

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CONTENTS

Part I

ON-SITE DISPOSAL OF SMALL WASTEWATER FLOWS

INTRODUCTION

In 1970, approximately 19.5 million households, or nearly 30 percent of all housing units in the United States, disposed of their wastewaters through some form of private sewerage facilities.[1] This number is growing at an increasing rate, owing to population movement to rural areas where community sewage treatment facilities are not usually available. Retired persons are moving back to rural areas, as are young families who follow industries to the fringes of metropolitan centers.[2] Most of these rural households use septic tank systems. Because of poor design, construction, or maintenance, however, a large number of these systems fail to provide adequate sewage treatment and disposal.

Many households, while located in rural areas, are situated in small communities or subdivisions ranging in size from a few households to 100 or more. In such instances, failing septic tank systems allow raw or poorly treated sewage to reach the ground surface, the surface body of water, or even the ground water, creating a severe public health hazard and nuisance because of the proximity of homes. Public wastewater facilities are often the only solution to this problem.

Assessing wastewater facility needs of small rural communities is difficult because of a lack of information. The last known published status report is a survey conducted by the U.S. Department of Agriculture in 1962.[3] At that time, 92 percent of the communities with populations of less than 1,000 had no public facilities, compared with 19 percent of communities with 1,000 or more. The results of the survey are presented in table I-1.

Since 1962, several government programs have been initiated that attempt to abate water pollution by providing grants-in-aid of construction for community sewerage facilities—namely, programs under the Federal Water Pollution Control Act (Public Law 92-500) and various State

Table I-1.—*Number of communities with and without public sewerage facilities in 1962*[3]

Population	Public sewerage facilities	
	Communities with	Communities without
26-999	3,803	42,837
1,000-2,499	3,079	1,391
2,500-5,500	2,027	349
More than 5,500	2,926	142
Total	11,835	44,719

programs. Consequently, the data presented in table I-1 need to be updated, but they do indicate that few small communities have public wastewater facilities.

The need for improved facilities exists in many of these communities, which 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 away from the house into a ditch or stream. More conscientious homeowners installed septic tank systems; but without good design 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 that conveys all the wastewaters to a community treatment plant. This central system has been preferred by government authorities, engineers, and the public for several reasons. First, the system has been tried and proven. Much technical expertise has gone into the theory, design, and operation of central sewerage, which has led to great confidence in it. Second, central sewerage is usually more cost-effective than other systems because of economies of scale. It is less costly to serve many people with one system than to serve each person individually. Third, central sewerage allows ready application of a central (usually public) management that is responsible for the proper functioning of the system.

For smaller communities and subdivisions, however, such a conventional collection and treatment facility is impractical because of the financial burden it places on residents or the developer. This is largely owing 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 operation and maintenance of the collection system. A more recent study of Sloggett and Badgers[5] of 16 small communities in Oklahoma showed a similar distribution. It is clear from these breakdowns (see table I-2) that the collection system is the most expensive component of any facility.

In small communities, homes are typically scattered, which causes sewer costs to be dramatically higher than those in larger communities. Sloggett and Badgers[5] demonstrated that the costs per customer rise as the number and density of customers decline. Construction costs per customer were compared with the density and number of customers served. (See tables I-3 and I-4.) Both

Table I-2.—*Distribution of total annual costs for municipal wastewater collection and treatment facilities*

[Percent]

	Amortization		Operation and maintenance		Overhead	Total
	Collection	Treatment	Collection	Treatment		
Smith and Eilers[4]	60.3	15.3	4.7	8.4	11.3	100.0
	Collection and treatment					
Sloggett and Badger[5]	72.6		14.2	3.2[a]	10.0	100.0

[a]Lagoons.

Table I-3.—*Cost of construction per customer relative to density of customers for 16 community wastewater facilities in Oklahoma*[5]

Item	Customers per mile of sewer			
	Fewer than 30	30 to 39	40 to 49	More than 50
Number of systems	5	5	1	5
Average cost per customer, 1972 dollars	1,100	847	696	575
Average number of customers	96	119	310	256

Table I-4.—*Cost of construction per customer relative to number of customers for 16 community wastewater facilities in Oklahoma, 1972*[5]

Item	Number of customers served			
	Fewer than 100	100 to 199	200 to 299	300 to 400
Number of systems	6	4	3	3
Average cost per customer, dollars	1,000	798	594	434
Customers per mile of sewer	28.3	37.8	49.4	55.2

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 Badgers[5] made similar comparisons using total annual costs. They found number and density of customers to be significant. (See table I-5.) In 1972, average annual costs per customer ranged from $76.90 to $43.36 for communities with populations of less than 100 and of 300 to 400, respectively. In 1968,[4] the national average for municipalities, large and small, was $19.80.

Because of the prohibitive costs of extending sewers, outlying sections of a community may not be served. In 30 percent of the communities with public facilities surveyed in 1962,[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.

In smaller communities, costs of conventional facilities can become prohibitive, exceeding $10,000 per household for the capital portion alone and costing even more if more than secondary treatment is required to meet water quality standards. It is not unusual for the cost of a system to approach the total equalized value of the community.[6]

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 give grants-in-aid of construction for 75 percent of the grant-eligible portions of the wastewater facility. Such grants would offset the high per capita costs in small communities, but such communities have difficulty obtaining them in many cases.

Noncentral Wastewater Facilities for Small Communities

A noncentral 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 nonurban setting. As table I-2 indicates, approximately two-thirds of the total annual cost of a conventional facility is for 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 to widely spaced homes would not be necessary. Instead, individual or jointly used septic tank systems or other treatment and disposal systems, located where the wastes are generated, could be used. Such systems could result in substantial savings because of the following advantages.[7]

- Functioning septic tank-soil absorption systems (ST-SAS) can be used rather than providing new service. Often, homeowners who are not having disposal problems or who have recently installed septic tank systems do not wish to support community action on a new wastewater facility that will cost them more money unnecessarily. Incorporating existing systems into the public system minimizes such opposition, as well as reduces the total cost of the public facility.

- Isolated homes and clusters of homes can be served individually instead of by costly sewer line extensions. This could be equally advantageous to existing communities and newly platted subdivisions. Where growth was not expected to be great enough to warrant sewer extensions, individual septic tank systems could be used. In cases where substantial growth was expected, such as in newly platted subdivisions, the first few homes built could be served by holding tanks pumped and maintained by the management entity. When the number of homes warranted a common disposal system, it could be built on land reserved for that purpose. This would delay construction until enough contributors were available to pay for it.

- Less costly treatment facilities usually can be constructed, and subsurface disposal often can be used that 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 the use of other simple treatment methods. In addition, by limiting the area served, the maximum future capacity can be more accurately predicted and a more optimal design can be provided.

- A more cost-effective facility may encourage smaller communities to proceed with construction rather than to wait 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.

- More rational planning of community growth is possible. Linear development, which is encouraged by the construction of interceptor sewers used to collect wastes from out-

Table I-5.—*Total average annual cost per customer for 16 community wastewater facilities in Oklahoma, 1972*[5]

[Dollars]

Number of customers	Total average annual cost		
	0 percent construction grant	75 percent construction grant	100 percent construction grant
Fewer than 100	76.90	33.06	18.44
100 to 199	57.55	25.39	14.63
200 to 299	52.10	24.09	14.75
300 to 400	43.36	20.72	13.17

lying clusters of homes, could be avoided. Growth could be encouraged in the more desirable areas by providing public service in those areas only.

- Noncentral facilities are more ecologically sound because they dispose of wastes over wider areas. Thus, the environment is able to assimilate the waste discharge more readily, and the need for mechanical treatment and associated energy consumption is reduced.

Management of Noncentral Disposal Systems

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 to properly manage a noncentral facility are similar to those needed with a conventional community system.

Powers Needed by a Management Entity.[7] To effectively administer on-site wastewater disposal systems, an entity must be able:

- To own, operate, manage, and maintain all wastewater systems within its jurisdiction. It 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 individual septic tanks 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 jurisdictional authority to operate, maintain, and perhaps own such systems outside its boundaries, by State statute, case law (essentially interpretations of State laws made by the courts), or terms of a contract.

- To enter into contracts, undertake debt obligations by borrowing and issuing bonds, and sue and be sued. These powers are more than legal niceties; without them, the entity could not acquire the property, equipment, supplies, and services necessary to construct and operate on-site systems.

- To 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; therefore, nongovernmental management entities must have the authority implied or directly granted to set and charge user fees to cover administrative costs.

- To plan and control how and when wastewater facilities will be extended to those within its jurisdiction.

Although not necessary to provide adequate management of a noncentral facility, two additional powers are desirable:

- To enact rules and regulations on 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 and replacement of malfunctioning systems according to its plans. This power, however, may be inferred from the statutory authorization to operate a system.

- To meet eligibility requirements for both loans and grants-in-aid of construction from Federal and State governments. Although a management entity can function without such loans and grants, viability of the noncentral system is strengthened when grant money is used to offset some or most of the costs to customers. Low-income families especially can ill-afford to finance the entire cost of their sewerage system. Experience has shown that low-income rural families cannot pay wastewater bills in excess of $7.00 per month or a combined water-sewage bill in excess of $14.00 per month.[8] Yet charging lower rates is difficult without public subsidy. The inequity should be especially obvious to most nonrural residents, who typically pay considerably less.

Acceptable Management Entities. The kinds of entities that could manage a noncentral facility vary from State to State. State constitutions, statutes, and administrative agency regulations must be examined to determine which entities are authorized to manage on-site systems. In addition, case laws must be checked to determine if the courts have construed the constitution, statutes, or regulations as having given to or removed from an entity the authority to manage such a system. Those entities that may have the necessary powers include municipalities, counties, townships, special districts, private nonprofit corporations, rural electric cooperatives, and private profit-making businesses.

Although they have disadvantages, the potential of noncentral facilities seems to warrant their further investigation. Many of the possible shortcomings of this alternate facility may vanish as some are constructed and experience is gained.

Collection and Treatment Alternatives for Noncentral Facilities. Proper facilities planning involves a systematic comparison of all feasible alternatives for wastewater treatment and disposal so that the most cost-effective solution, which will minimize total costs to the community and the environment over time, can be found.

The trend toward gravity sewers with a common central treatment plant has eliminated many worthy alternatives from consideration. If this bias can be changed and the noncentral concept used, environmental and monetary costs of wastewater facilities in many communities could be significantly reduced by reducing the size of, or eliminating, the collection system and by simplifying the treatment facility.

The most extreme noncentral system would be one in which each home and establishment was served by an individual septic tank system. The most cost-effective community system probably lies between the 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 individual and joint systems.

Alternatives for dealing with wastewater treatment and disposal are numerous. To evaluate which method is most cost-effective for a particular community would seem a monumental task. It can be greatly simplified, however, by selecting the proper point from which to begin the design.

A wastewater facility must produce an effluent that 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 is too great for the environment to assimilate, pollutants will accumulate. The physical characteristics of the local environment 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 large volumes of water can be easily discharged into rivers and streams. This practice requires, however, rather high degrees of treatment before discharge to prevent degradation of the stream. If soil is the disposal medium, lower levels of pretreatment are required before disposal because of soil's greater assimilative capacity. The trade-off is that large areas of land are required for absorption. When operation and maintenance costs of high levels of treatment for surface-water disposal are compared with costs for land disposal, however, land disposal may be a more cost-effective alternative. A similar situation may exist for atmospheric disposal, as in evapotranspiration (ET).

Thus, the first step in designing community wastewater facilities is to characterize the local environment. Once it is determined what disposal media are available, systems can be designed to fit for cost-effective comparisons. This requires a knowledge of the receiving environment's waste assimilation capabilities. Federal and State regulatory agencies have already set effluent standards for surface waters. The assimilative capacities of soil and ET systems are poorly understood, however, and need to be reviewed.

USE OF SOIL FOR TREATMENT AND DISPOSAL OF WASTEWATER

Liquid Movement Into and Through Soil and Soil Materials

Proper performance of on-site wastewater disposal systems depends on the ability of the soil or a soil material to absorb and purify the wastewater. Failure occurs if either of these functions is 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 between the grains. The size and shape of these pores is determined by 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 cement together, forming aggregates (peds). Planar voids form between the peds. Tubular channels made by plants and animals living in the soil and irregularly shaped discontinuous pores called vughs also are found. (See fig. I-1.)[9]

Soil permeability, or capability to conduct water, is not determined by soil porosity but by 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. The soil structure that helps to maintain the pores is very dynamic and may change greatly in response to changes in natural conditions, biological activity, and soil-management practices. Repeated wetting, drying, and freezing help to form peds; plants with extensive root systems and soil fauna activity promote soil aggregation and channeling. Mechanical compacting and adding soluble salts can break down the peds, reducing the capacity of the soil to conduct water.

Characterization of Water in Soils. Under natural drained conditions, some pores are filled with water. The distribution of the water depends on the characteristics of the pores; the water's movement is determined by its relative energy status. 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 and is equal to the weight of the water. To raise water against gravity, work must be done; 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.

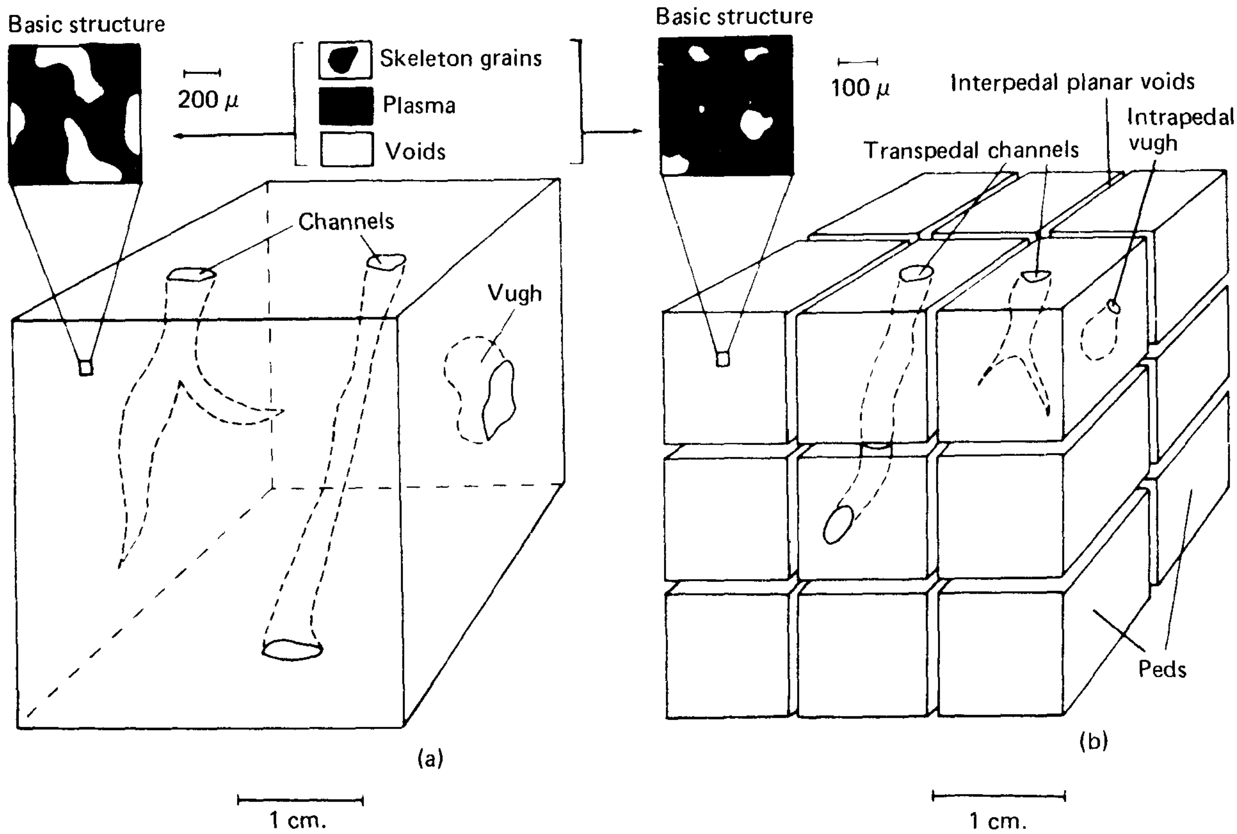


Figure I-1. Schematic of (a) apedal (single-grained) and (b) pedal (aggregated) soil fragment.[9]

The matric potential is produced by the affinity of water for soil particle surfaces. The pores and surfaces of soil particles hold water because of forces produced by adsorption and surface tension. Molecules within the liquid are attracted to one another, 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 and 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 to other water molecules; 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 to a relatively low capillary rise of 28 cm water (pressure below meniscus equals -28 cm water), whereas a pore radius of 30 microns results in a relatively high rise of 103 cm (pressure equals -103 cm water) as illustrated in figure I-2.[10] This indicates that it takes more energy to pull water from a small pore than from a large one. The water within the tube is at less than atmospheric pressure, as noted, because it is pulled downward by gravity as it is pulled upward by capillary action. The water is under tension, as the tube sucks the water into it. This negative pressure is called soil tension or soil suction and is measured in millibars (mbars).

Adsorption forces also contribute to matric potential. Molecular forces between the surface of the soil particles and the water cause the water to form envelopes over the particle surfaces and to be retained in the soil. (See fig. I-2.)

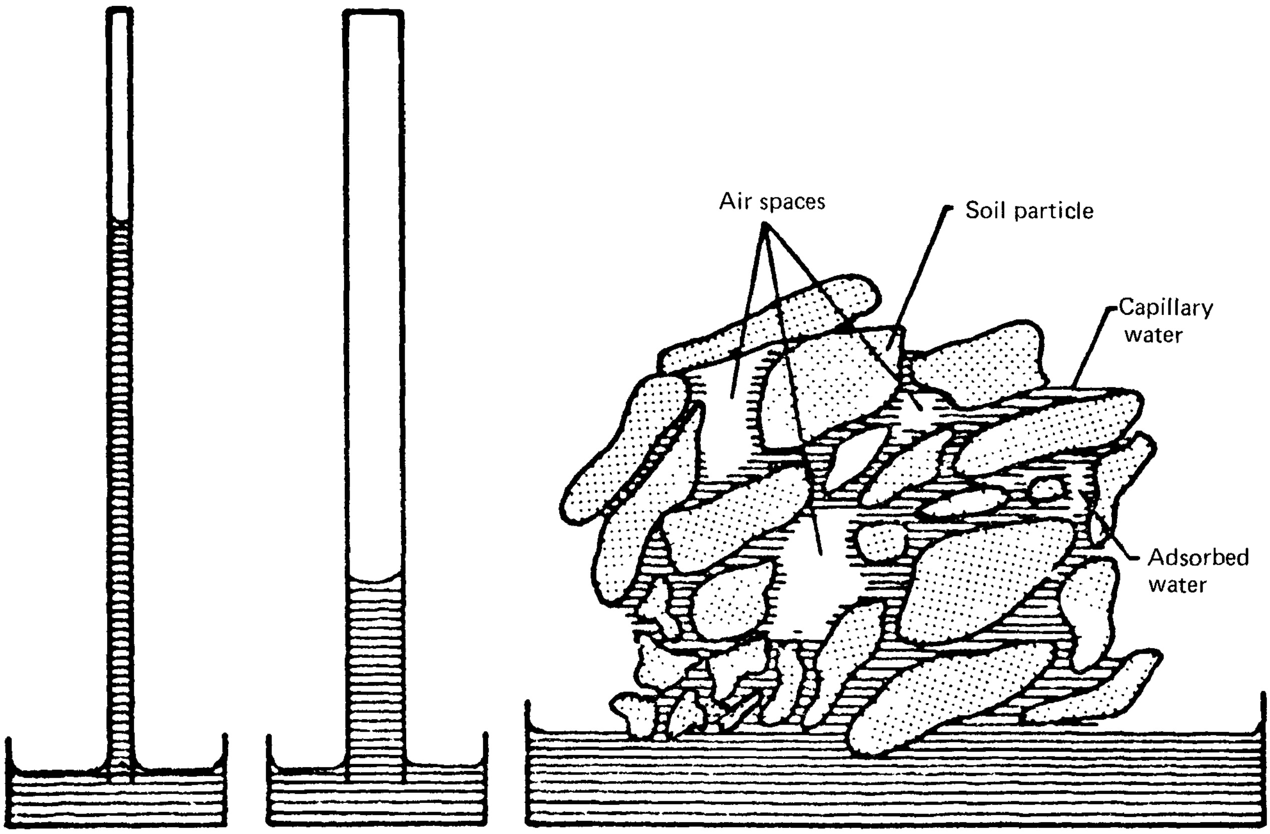


Figure I-2. Upward movement by capillarity in glass tubes as compared with soils (after Brady[10]).

When soil is saturated, its pores 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 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 soil moisture as tension increases is a function of pore-size distribution and is characteristic for each soil type. Figure I-3 shows the soil moisture retention curves for sand, silt loam, sandy loam, and clay. 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 held in very fine pores. The silt loam has a greater number of coarse pores than does the clay, so its curve lies somewhat below that of the clay. The sandy loam has a greater number of fine pores than the sand does, so its curve lies above that of the sand.

Liquid Flow in Soils. Gravitational potential pulls water downward; 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 hydraulic conductivity, K, defined by Darcy's Law:

$$Q = KA \, dH/dZ$$

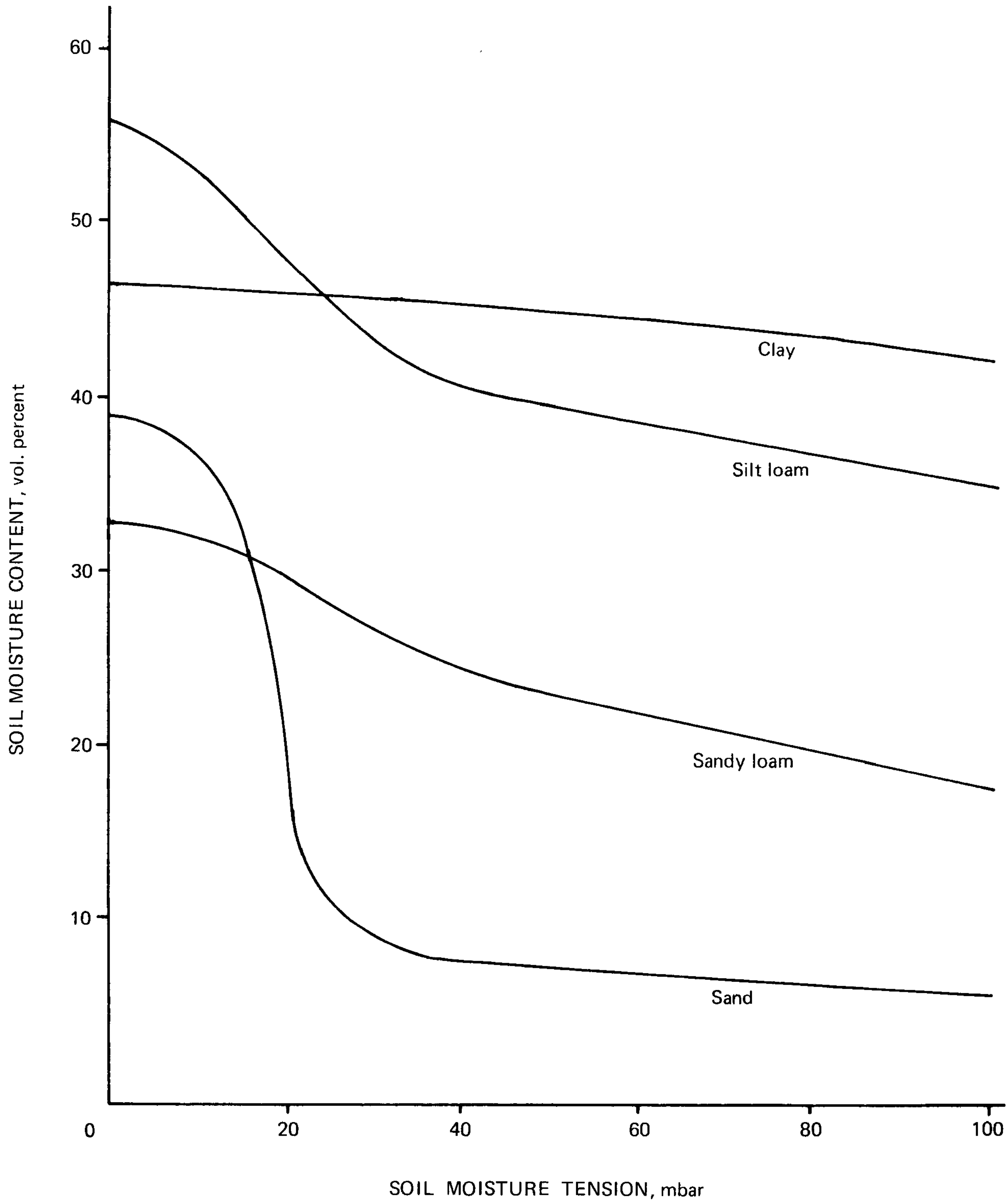


Figure I-3. Soil moisture retention for four soil materials.[9]

where

Q = flow rate		Q	1 = flow rate
K = hydraulic conductivity		K	1 = hydraulic conductivity
A = cross-sectional area of flow		A	1 = cross-sectional area of flow
dH/dZ = hydraulic gradient		dH/dZ	1 = hydraulic gradient

This parameter (K) accounts for all factors affecting flow in soil, including tortuosity and size of the pores. Thus, the measured K values for soils vary widely because of differences in pore-size distributions and pore continuity.

The hydraulic conductivity often changes dramatically with changes in soil moisture tension. At a tension equal to or less than zero, the soil is saturated and all pores 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 that cause flow become capillary. As the water content decreases or tension increases, the path of the water flow becomes more tortuous because the water travels along surfaces and through sufficiently small pores to retain water at the prevailing water potential. The unsaturated K, therefore, usually is much lower.

To illustrate this, three soil materials with differing pore-size distributions are represented in figure I-4. One soil is coarse and porous (like a sand); one is fine and porous (like a clay); and one has both large and fine pores (like a sandy loam). When there is an open infiltrative surface and a

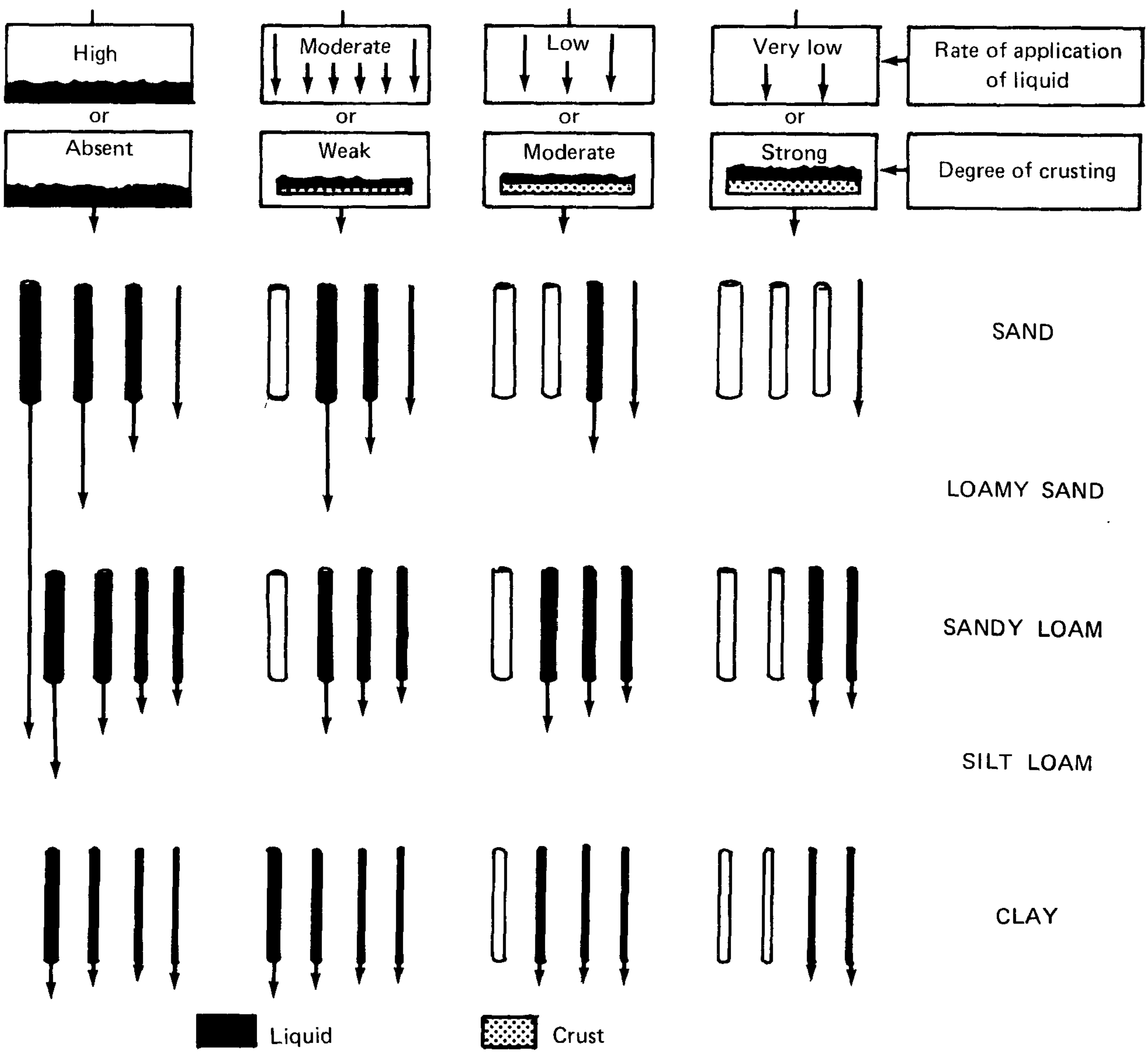


Figure I-4. Effect of increasing crust resistance and decreasing rate of application of liquid on the rate of percolation in soil.[9]

sufficient water supply, all soil pores are filled, and each conducts water downward because of 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 and restricts 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; therefore, progressively smaller pores empty at increasing crust resistance. This crusting leads to a dramatic reduction in K. (See fig. I-5.)[11]

If no crust is present, similar phenomena occur when the rate of water applied to the capillary system is reduced. When there is an 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 because the smaller pores have a greater capillary attraction for water. Thus, larger pores can fill with water only if smaller pores are unable to conduct away all the applied water.

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 K decreases more slowly with increasing tension. The K curves determined in situ show such patterns. (See fig. I-5.)

The K curves for the pedal silt loam and clay horizons demonstrate the physical effect 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/d 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/d for the silt loam).

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 by hydrostatic pressure and capillary pull.

Phenomena contributing to the development of a clogging zone at the infiltrative surface of soil absorption systems include: compaction, puddling, and smearing of the soil during construction; puddling caused by the constant soaking of the soil during operation; blockage of soil pores by solids filtered from the waste effluent; accumulation of biomass from growth of microorganisms; deterioration of soil structure caused by exchange of ions on clay particles; precipitation of insoluble metal sulfides under anaerobic conditions; and excretion of slimy polysaccharide gums by some soil bacteria.

Many systems fail, usually within a year or two, because of poor construction techniques. Absorption of water by soils depends on preservation of a suitable soil structure, but soil structure can be partially or completely destroyed during construction. Extensive damage does not occur in single-grained soils (sands) but can occur in aggregated soils with high clay content. 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 and soil particles to 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 heavy machinery frequently passing over the field; smearing from excavating equipment; and puddling from exposure of the infiltrative surface for a day or more to rainfall or wind-blown silt that seals off the soil pores. As a result, the absorption field may be clogged before it is put into service.

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 resistant to high compressive forces. When wet, however, the clay plates separate when forces are applied to the soil. The water

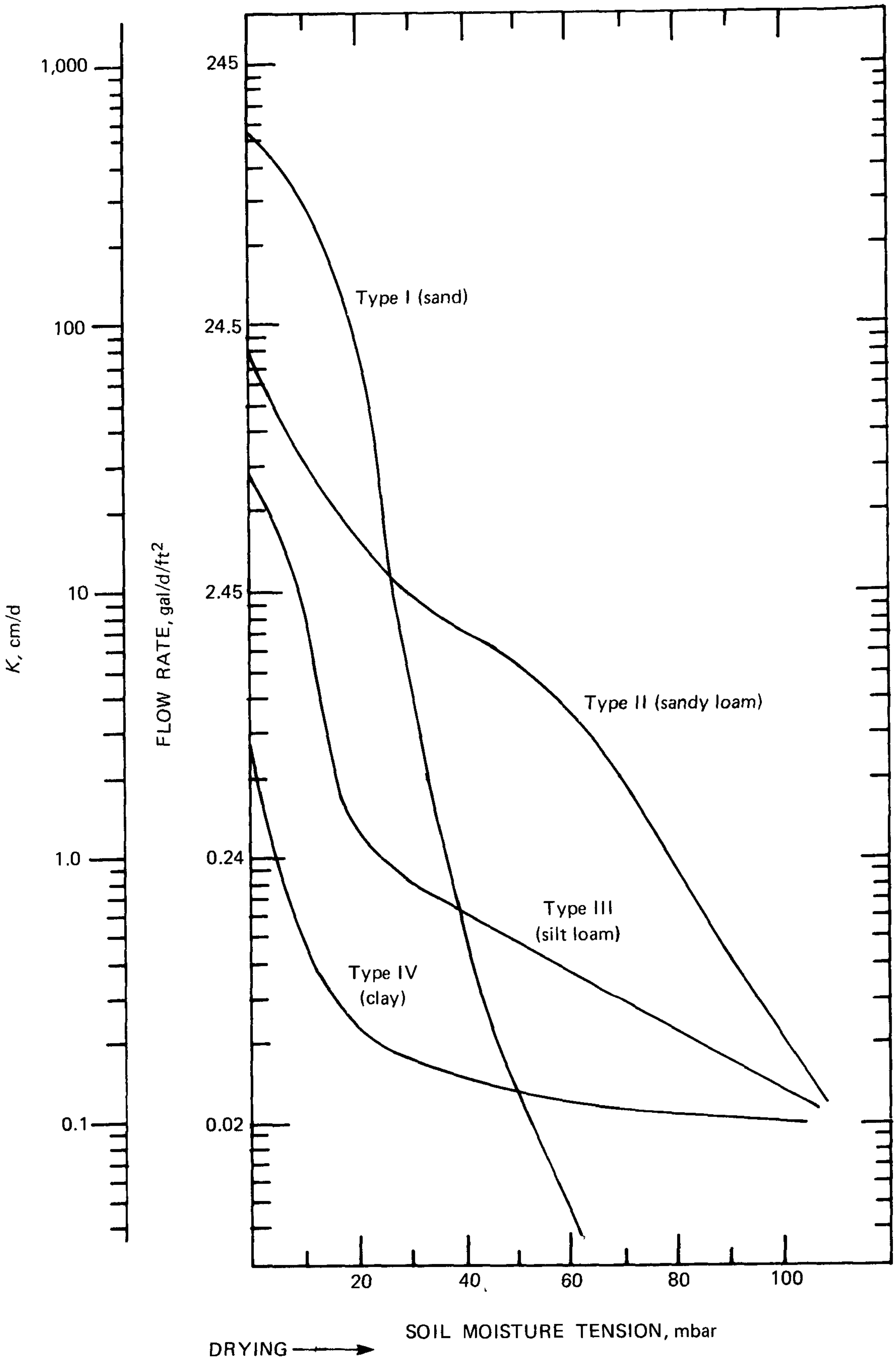


Figure I-5. Hydraulic conductivity (*K*) as a function of soil moisture tension measured in situ using the crust test.[11]

acts as a lubricant as the clay plates move relative to one another to close channels and vughs 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 on soil type, moisture content, and applied force. Soils with high clay content are easily puddled; sands are not affected. Clayey soils will not puddle, however, if they are only slightly moist. Under pressure, dry clay breaks into small fragments along pedal boundaries rather than smears, thereby keeping the large pores open.

Several studies indicate that physical and biological mechanisms are the primary causes of soil clogging in an absorption field not smeared and compacted during construction.[9,12-26] In these instances, clogging seems to develop in three stages: slow initial clogging, rapid increase of resistance leading to permanent ponding, and leveling off toward equilibrium. Initial development of a clogging zone seems to be largely the result of accumulation of suspended solids (SS) from the wastewater, so that the liquid seeps away more and more slowly between loadings. Aerobic bacteria decompose many of the organic solids, thereby helping to keep soil pores open; but they can function only when the infiltrative surface drains between doses, allowing air to enter. As the clogging zone begins to form, decreasing the aerobic periods between ponding, the aerobic bacteria eventually are unable to handle the influx of solids. Permanent ponding results, leading to anaerobic conditions. 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 because anaerobic bacteria destroy soil-clogging organics less efficiently. 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 that grow in such an environment produce gelatinous materials (bacterial polysaccharide slimes or gums) that clog soil pores very effectively. At that point the clogging mat seems to reach an equilibrium in which the resistance to flow changes little. An absorption field will not fail, however, if the application rate does not exceed the equilibrium rate. The process can be reversed and much of the original infiltrative capacity restored if the ponded surface is allowed to drain and rest, permitting aerobic biological decomposition and the drying and cracking of the clogging materials.

Infiltration not only depends on the resistance of the clogging zone but also on the capillary properties of the underlying soil.[11] 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/d in a sandy loam, 7 cm/d in a sand, 4 cm/d in a silt loam, and 1.8 cm/d in a clay.[27] 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.

Significance of Unsaturated Flow. Liquids flow at a much slower rate in unsaturated soil than in saturated soil because flow only occurs in the finer pores. This slows infiltration but enhances purification. Wastewater effluent is purified by filtration, biochemical reactions, and adsorption-processes that are more effective in unsaturated soils because average distances between effluent particles and the soil particles decrease as the time of contact increases. This flow phenomenon is illustrated in figure I-6, which shows a thin section of the C horizon of a Saybrook silt loam, a stony sandy loam till with a K_{sat} of 80 cm/d. The flow velocity of water in the soil pores can be estimated from its moisture retention curve (fig. I-3). This velocity can be used to derive the time for water to travel 1 foot (30 cm), assuming a hydraulic gradient of 1 cm/cm owing only to gravity. Successively smaller pores empty at increasing tensions and K decreases correspondingly. (See figs. I-4 and I-5.) Calculated travel times increase from 3 hours at saturation to 30 hours at 30 mbar and 8 days at 80 mbar of soil moisture tension.

In structured soils it is possible for flow to be predominantly through the planar voids, bypassing the interiors of the peds. High liquid application rates may result in a great deal of dispersion

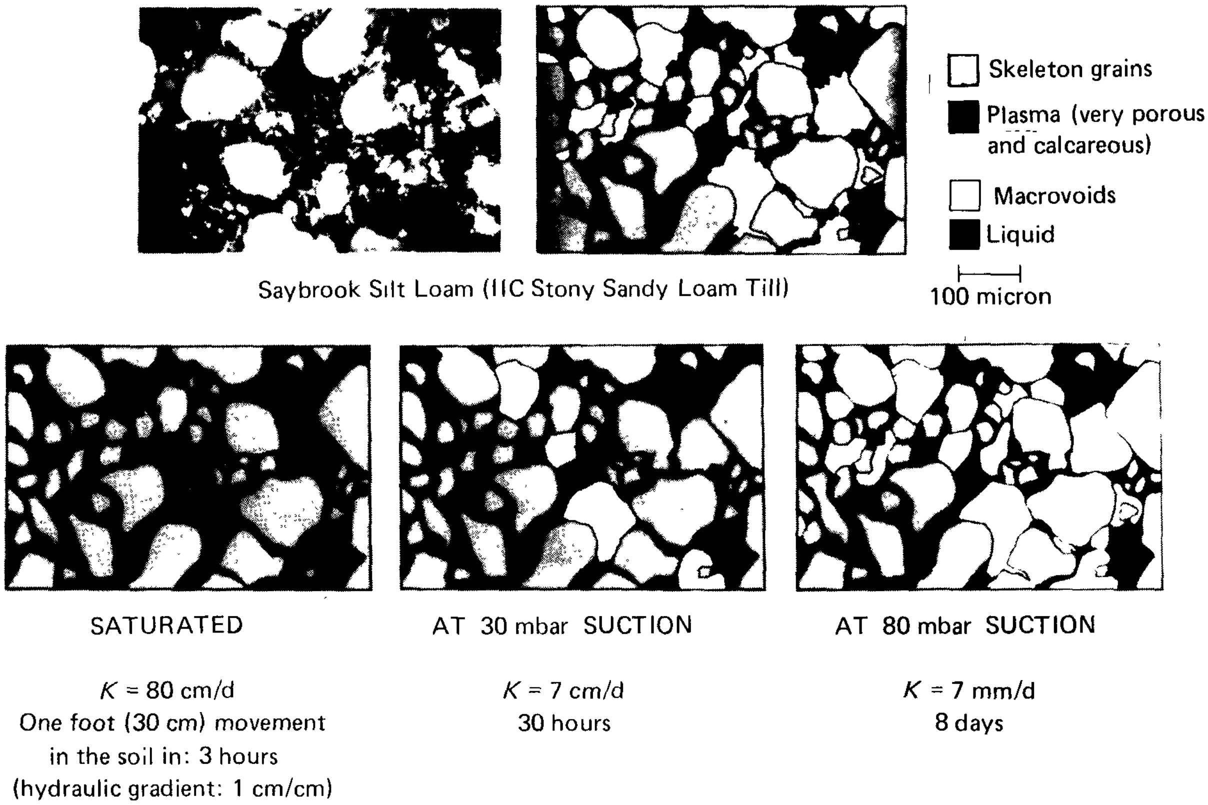


Figure I-6. Occurrence and movement of liquid in a saturated and unsaturated sandy loam till, C horizon, of Saybrook silt loam.[9]

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 application rates 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 have shown short-circuiting to be a particular problem on drained soils dosed at relatively high rates.[27]

Short-circuiting in a structured soil is illustrated in figure I-7. If the large planar voids are drained and air-filled, liquid applied at the surface at a high rate will quickly pass through the large pores before much of it can enter the fine pores of the peds. The retention time of most of the liquid, therefore, is low, and only a portion of the 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. Also, 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 in achieving 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. When an absorption system in a highly porous or dry structured soil is first put into service, however, and there is no clogged zone, there is danger of inadequate purification unless design precautions are taken to ensure that the soil remains unsaturated.

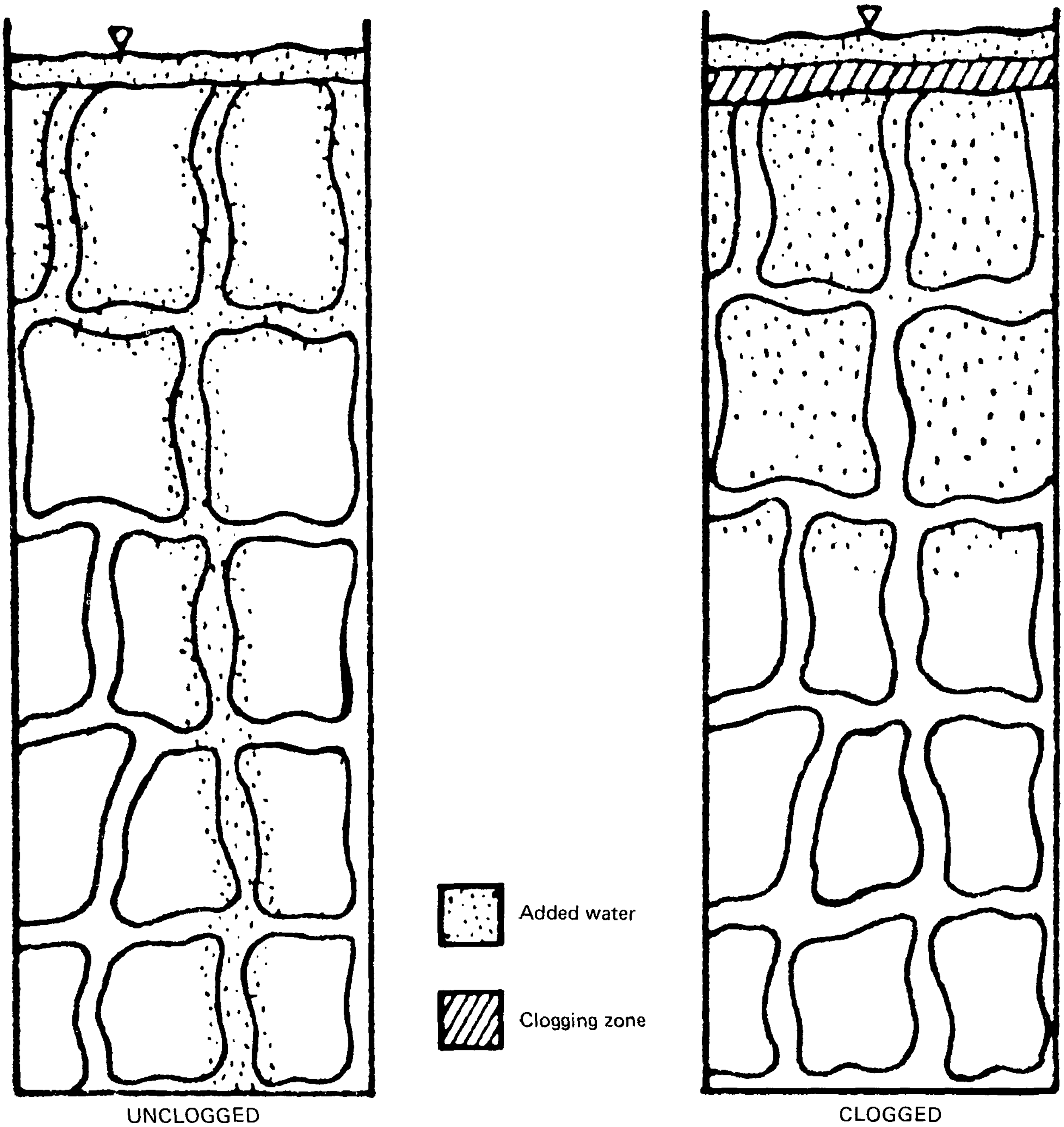


Figure I-7. Influence of clogging zone on short-circuiting in structured soils.

Treatment Capabilities of Soil

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, pathogenic organisms, and viruses that are not removed before they are applied to the soil must be removed or transformed by it. Numerous studies show that under proper conditions, soil is an extremely efficient purifying medium.

Bacteria and Virus Removal. 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 soil's efficiency in removing pathogens and the various parameters that affect its efficiency, such as soil type, temperature, pH, organism adsorption to soil and soil-clogging materials, soil moisture, nutrient content, and biological antagonisms.[28] 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 I-8 shows removal of fecal coliforms (FC) and fecal streptococci (FS) from septic tank effluent by two columns packed with 2 feet of Plainfield loamy sand (effective size 0.14 mm, uniformity coefficient 1.99).[29, 30] Both columns were loaded well below their K_{sat} rates of nearly 400 cm/d (96 gal/d/ft^2), but one was loaded at twice the rate of the other. The number of bacteria discharged from both columns reached a plateau during the first 100 days of application, then declined. Fewer bacteria passed through the column that had the lower loading rate. Column one, loaded at 10 cm/d (2.4 gal/d/ft^2), removed approximately 92 percent of FC applied per day; column two, loaded at 5 cm/d (1.2 gal/d/ft^2), removed 99.9 percent. FS and *Pseudomonas aeruginosa* were also found in the effluent from the more heavily loaded column one. These organisms were not detected in effluent from column two. During this period a clogging zone developed on the infiltrative surface of each column, and FC counts in the effluents from both columns eventually dropped to between 10 and 100 FC/100 ml.[30]

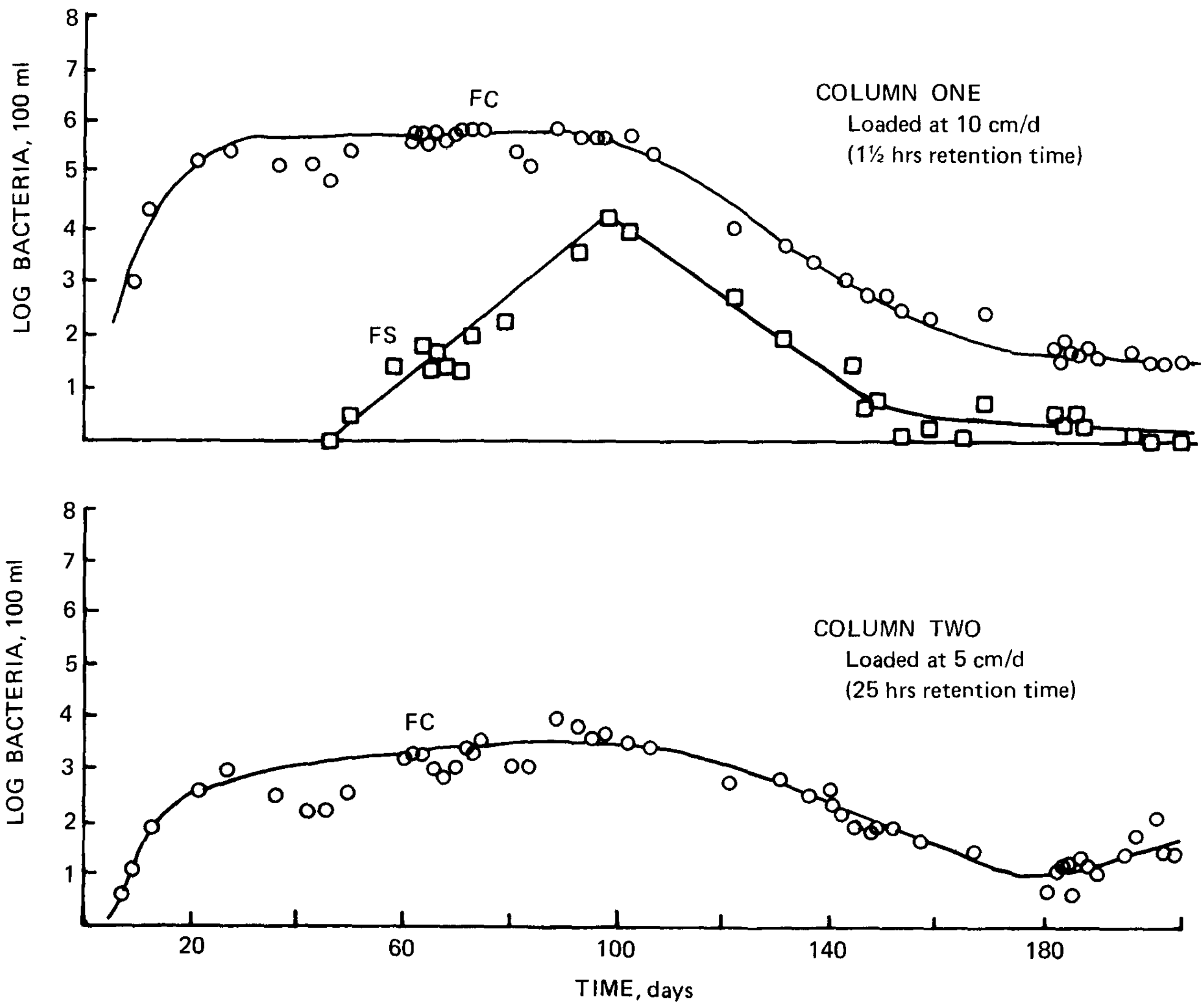


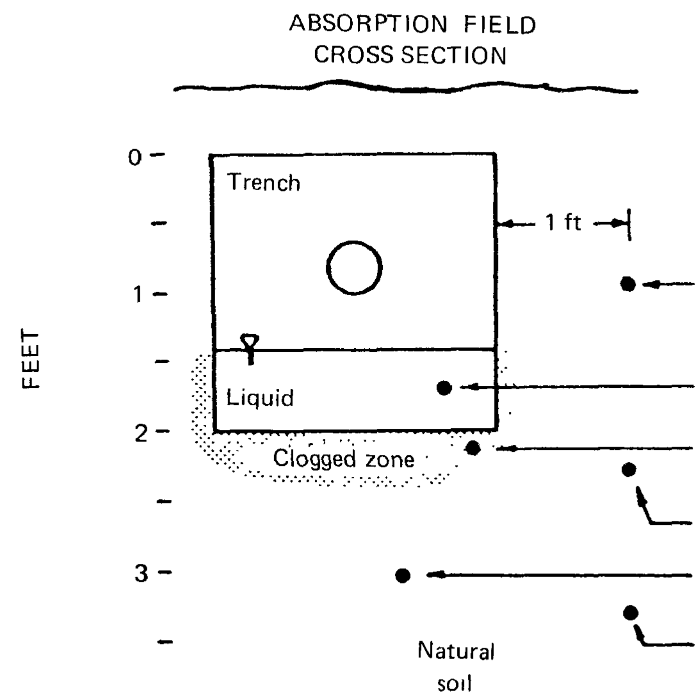
Figure I-8. Bacteria counts in effluents from sand columns loaded with septic tank effluent.[29]

Septic tank systems installed in sands also exhibit the effects of the clogging zone in removing indicator bacteria. Figure I-9 shows bacterial counts from 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 to the side of the trench were similar to those found in the natural soil.[9,29,30]

Concurrent studies of Almena silt loam were also conducted.[30] This soil has a lower capacity to conduct liquid than the unstructured sands and most of the flow is through the larger pores between peds. Undisturbed cores, 2-feet deep, of Almena silt loam were loaded with septic tank effluent at a rate of 1 cm/d (0.24 gal/d/ft^2). 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/d to 3 mm/d to promote slow flow through the peds rather than through the larger cracks around them (fig. I-7), bacterial counts decreased dramatically to below 2/100 ml of FC, FS, and *P. aeruginosa*. When the loading was restored to 1 cm/d, high counts were again observed. (See fig. I-10.)

Virus adsorption and inactivation in soils have been of considerable interest to scientists and engineers. When viruses enter a septic tank or other treatment process, they are likely to associate with cells in fecal material. These masses settle, releasing some viruses, depending on turbulence within the process. (In laboratory studies up to 89 percent of the polio virus added in fecal material was released by vigorous shaking.[31]) Secondary adsorption on wastewater solids may occur in treatment processes. The free and particle-adsorbed virus will then be discharged to treatment processes or the soil absorption field.

Removal of viruses in soils results from the combined effects of sorption, inactivation, and retention. On entering the soil, viruses 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.[32] In the adsorbed position, the viruses are inactivated in a spontaneous process that is temperature-dependent, being greater at higher temperatures.[31] Virus detention in the soil is



Bacteria, 100 ml or 100 g of soil FS	FC	Total coliforms	Total bacteria X 10^7
<200	<200	<600	0.6
160,000	1,900,000	5,700,000	3.0
54,000	4,000,000	23,000,000	4,400
<200	17,000	23,000	6.7
<200	<200	<600	3.7
<200	700	1,800	2.8

Figure I-9. Cross section of seepage trench in sand showing bacterial counts at various points near the trench.[17,19]

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 viruses to come in contact with surfaces to which they can adsorb.

In laboratory studies using packed sand columns, septic tank effluent was inoculated with more than 10^5 plaque-forming units (PFU) per litre of polio virus type I.[31,33] All viruses were removed in the 24-inch columns at a loading rate of 5 cm/d (1.24 gal/d/ft^2) over a period of more than one year. At a loading rate of 50 cm/d (12.4 gal/d/ft^2), virus breakthrough occurred (fig. I-11). Analysis of the sand residue after virus application indicated that adsorbed viruses in the column were inactivated at a rate of 18 percent per day at room temperature and at 1.1 percent at 6° C to 8° C.[33]

In contrast, viruses were detected approximately 60 inches within columns packed with calcareous loamy sand and fed secondary effluent containing 3 X 10^4 PFU polio virus type I at a rate of 15 cm/d (3.7 gal/d/ft^2).[32] Most of the viruses were adsorbed in the top 2 inches of the soil and removal was not appreciably affected at application rates of between 15 and 55 cm/d (3.7 and 13.6 gal/d/ft^2). Only deionized water desorbed the viruses, but drying for 5 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 with Almena silt loams to 7.9 logs per cm.[33] It should be emphasized,

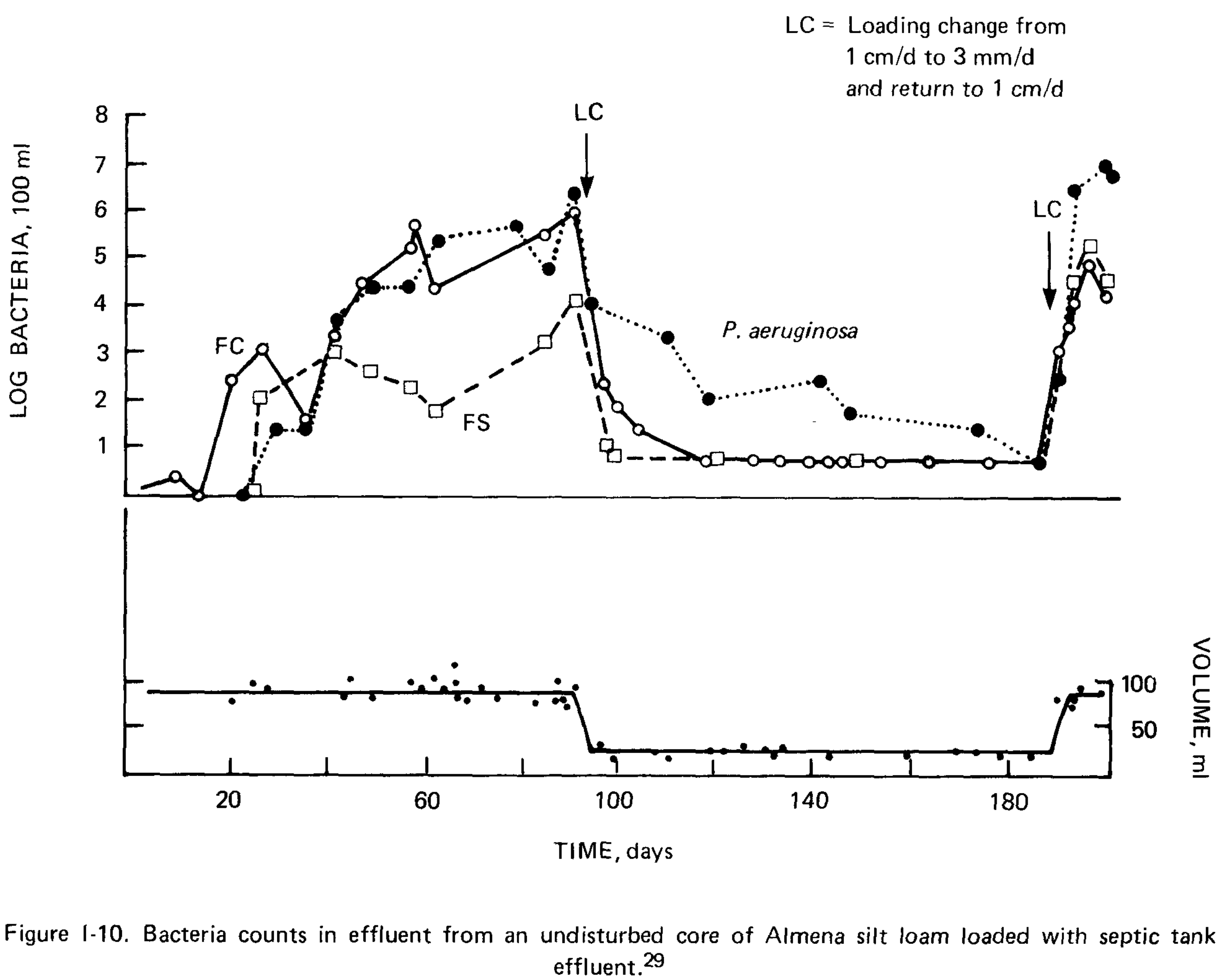


Figure I-10. Bacteria counts in effluent from an undisturbed core of Almena silt loam loaded with septic tank effluent.[29]

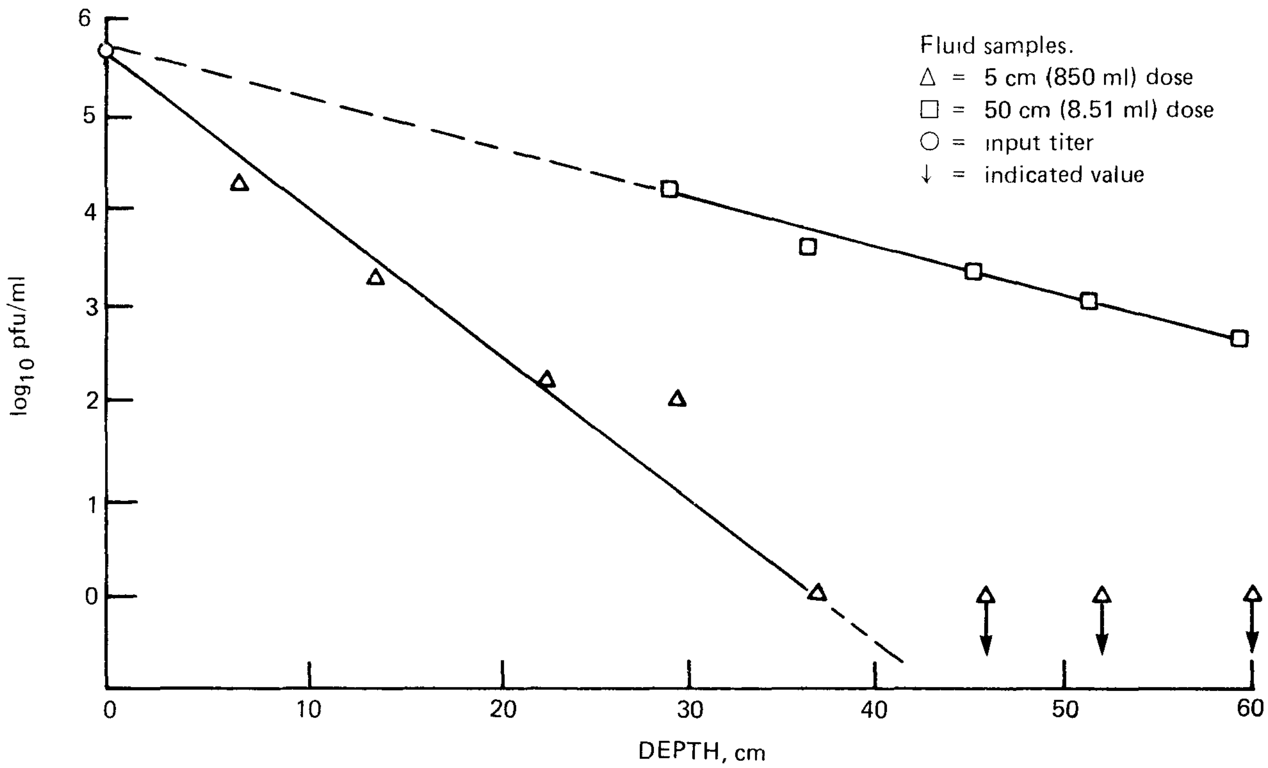


Figure I-11. Penetration of polio virus into a 60-cm conditioned sand column at room temperature.[33]

however, that finely ground soils do not exist in the field. Channels in natural soil reduce opportunity for virus adsorption, and viruses may travel long distances when loading rates are high.

Chemical Transformation and Removal. Domestic wastewaters may contain a few chemicals hazardous to public health or to the environment. Nitrogen and phosphorus compounds are discharged in household wastewater and 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.[34] A safety limit for nitrate of 10 mg/l as nitrogen is recommended by the U.S. Public Health Service (USPHS).[35] There are many reports of nitrate concentrations above the 10 mg/l nitrogen limit in wells near septic tank systems.[34,36-38] Accelerated eutrophication of surface waters is also attributed to nitrate from waste discharge.[37]

In solution, nitrate moves freely through the soil, although there can be some denitrification (reduction of nitrate to nitrogen gas) where organic material and an anaerobic environment occur together. Nitrogen in septic tank effluent is about 80 percent ammonium and 20 percent 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 the seepage field.[39] This is illustrated in figure I-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; the nitrogen then remains in the form of ammonium. Ammonium is readily adsorbed by soil materials of high clay content; hence it migrates much more slowly.[36,39]

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. Phosphorus enrichment of ground water seldom occurs below septic tank systems, however, because it is fixed in soil by sorption reactions or as phosphate precipitates of calcium, aluminum, or iron. Calcium is usually found in the wastewater, and aluminum and iron are abundant in most soils.[40] Phosphorus may

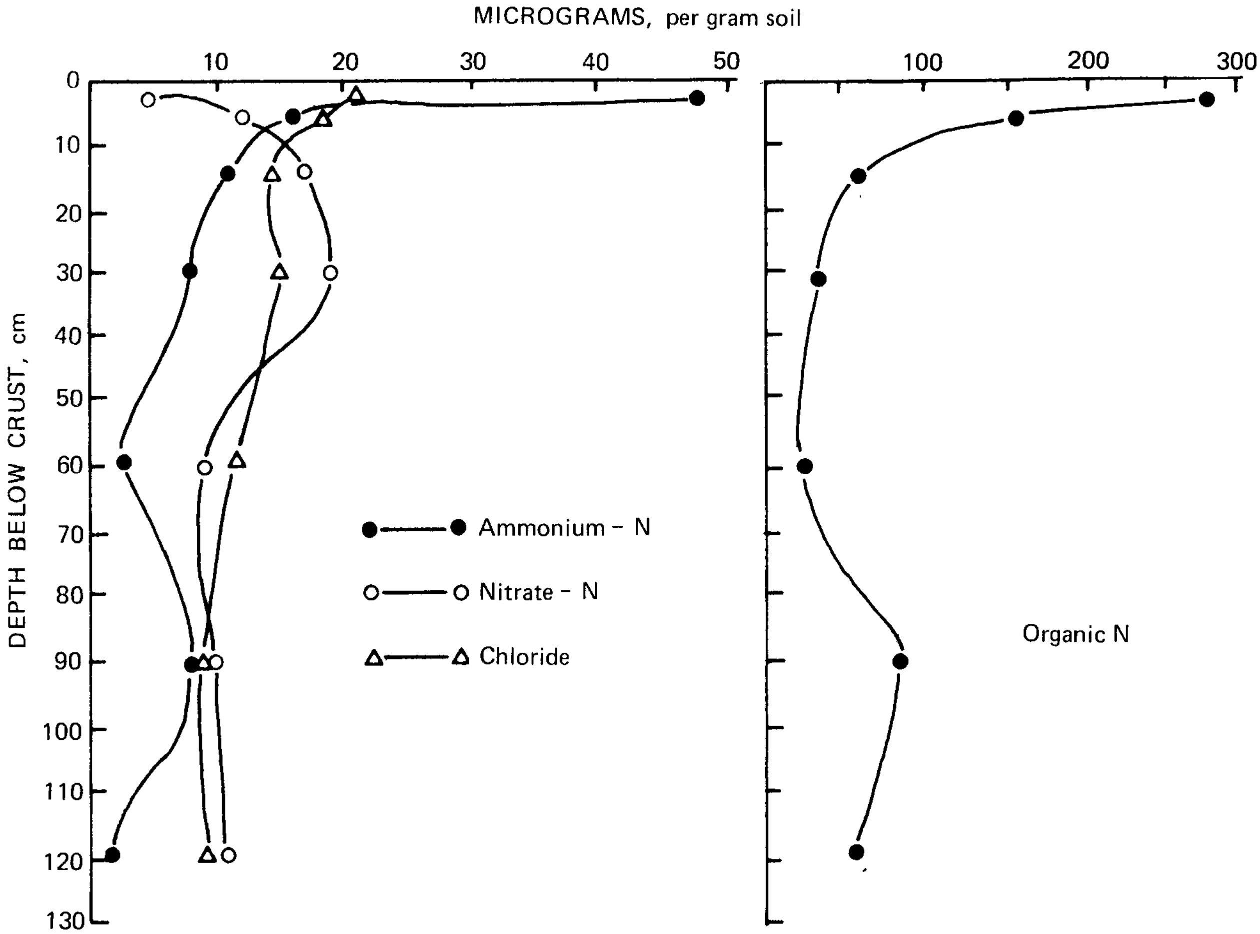


Figure I-12. Concentrations of NH_4-N, NO_3-N, organic N, and Cl in unsaturated soil below the clogged zone in sand.[29]

leak into the ground water, however, where high water tables or very coarse sand and gravel occur, or where the seepage bed has been loaded heavily for a long time. In such instances, concentrations of phosphate above 5 mg/l as phosphorus have been observed.[37] Phosphorus can move downward 50 to 100 cm/yr through clean silica sand,[41] but movement in loams, silt loams, and clays is much slower (5 to 10 cm/yr). Thus, except in coarse soils, it takes more than 10 years for phosphorus to move as much as 3 feet.[41]

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

ESTIMATING THE INFILTRATIVE AND PERCOLATIVE CAPACITIES OF SOIL

Site selection criteria for on-site systems vary from folk knowledge and experience to empirical tests, often codified into rules. The USPHS's general reference manual[42] has provided guidelines for many State, regional, and local manuals and codes of practices.

Several factors are usually considered in the selection of a site. The ability of the soil to absorb liquid, usually estimated by the percolation test, is a common requirement. Other factors considered are slope, depth to ground water, nature of and depth to bedrock, likelihood of seasonal flooding, and distance to well or surface water.[42] These traditional factors have several limitations and vary widely among codes.

Estimation of Soil Permeability

The Percolation Test. In 1926, Henry Ryon developed a test to obtain field data on failing seepage systems.[43] He dug a 1-ft^2 hole to the depth of the failed system, 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 1 inch (percolation rate). To calibrate the test, Ryon inspected several failing or near-failing systems and noted the loading of the system, soil characteristics, and percolation rate 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, although 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 may be predicted from its initial ability to absorb clear water.[43] From Ryon's data comparing absorption rates of existing septic tank systems with the percolation test, the measured rate is reduced by a factor ranging from 20 to 2,500 in order to size the absorption area.[9] The results of the test are highly variable, however, 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 percent;[44,45] thus, the procedure is unreliable.

The Crust Test. The soil below most operating absorption systems is unsaturated because of the clogging mat that develops at the infiltrative surface. To properly size an absorption system, therefore, the unsaturated *K* characteristics of the soil must be known. Because the standard percolation test does not provide these kinds of data, the crust test was developed.[9,44,46-49]

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 installed in the column just below the infiltrative surface measures the soil moisture tension as water is applied to determine the degree of saturation in the soil. (See fig. I-13.) To maintain unsaturated conditions in the soil column, a crust of gypsum and sand is placed over the soil surface. Water flowing through the infiltrometer 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 *K* curve. Additional tests run with crusts of various hydraulic resistances define the *K* curve as shown in figure I-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.

Although this procedure offers a direct measurement of *K*, it is time-consuming, requires a skilled operator, and cannot be run economically at each site. Because *K* depends on the pores in the system, however, the conductivity of a soil at various sites can be defined within statistical limits.[50] (See fig. I-14.)[50] Also, variability curves of soils in the same textural groups have similar *K* curves.[50] (See fig. I-15 and table I-6.)[50] Therefore, by defining families of *K* curves for groups of soils, the *K* characteristics of a particular soil or site can be predicted.

In Wisconsin, four major *K* types have been suggested based on the texture of the soil materials.[11] 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 similar groupings might be made,

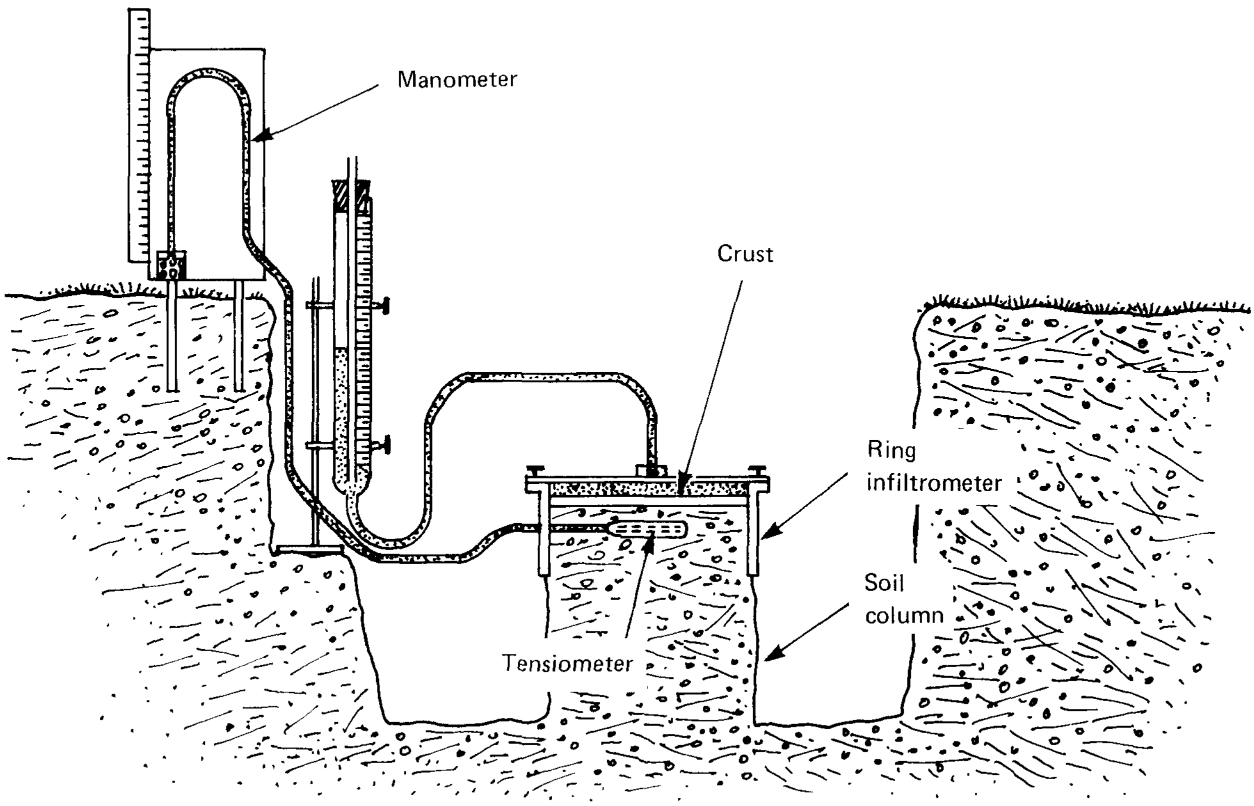


Figure I-13. Schematic of the crust test procedure.

but they must be based on field data because differences in soil mineralogy may affect groupings.) Typical K curves were developed from field measurements for each of these types. (See fig. I-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.[11] This information provided a design point on the curve for proper field sizing. The same procedure could be used to select design points for other types of soil systems. (See table I-7.)

The application rates for various soils presented in table I-7[11,51] represent the best estimates available. Because of the unstructured nature of the sands and sandy loams, the rates are reasonably accurate. Because of the nature of the flow that occurs in finer textured, structured soils, however, there is more variability in the tensions measured under operating fields.[11] In these soils the design rates must be used with care, particularly where expandable clays are present.

Estimation of High Ground Water

To ensure adequate purification of the wastewater before it reaches ground water, a minimum of 3 feet of unsaturated soil is necessary below the infiltrative surface. If saturated soils occur within 3 feet, transmission of harmful pollutants to the ground water may result.[29,33,52] Determining if saturated conditions occur within the minimum is often difficult, however, because water-table levels fluctuate with changing weather conditions. Typically, the water table is low during the summer and rises in the spring and fall. Ideally, the highest ground water level should be recorded, but this is not very practical. Moreover, observations made in relatively dry years are not representative. Thus, other methods must be used to determine the high water elevation.

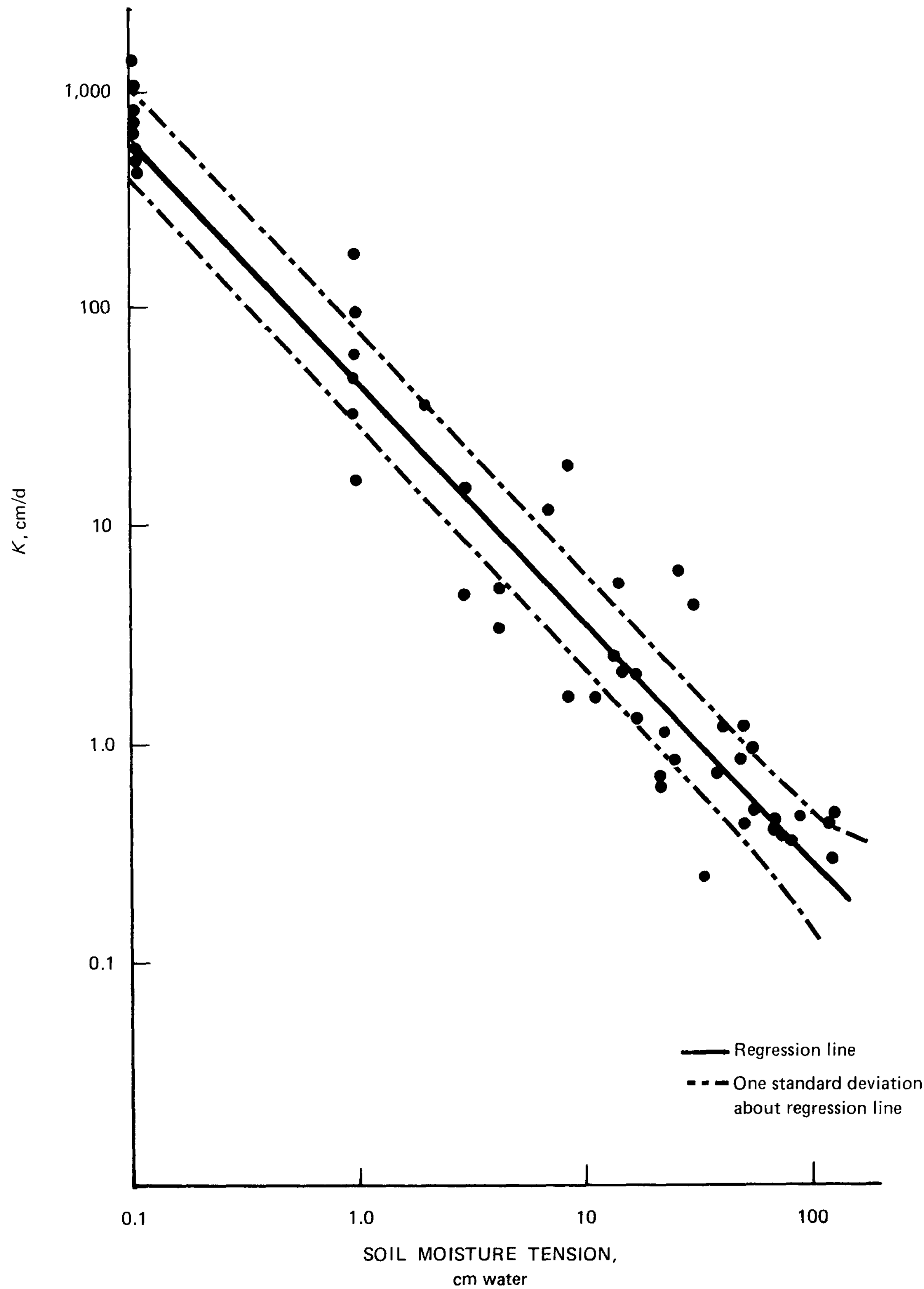


Figure I-14. Hydraulic conductivity (*K*) data for Plano series (after Baker[50]).

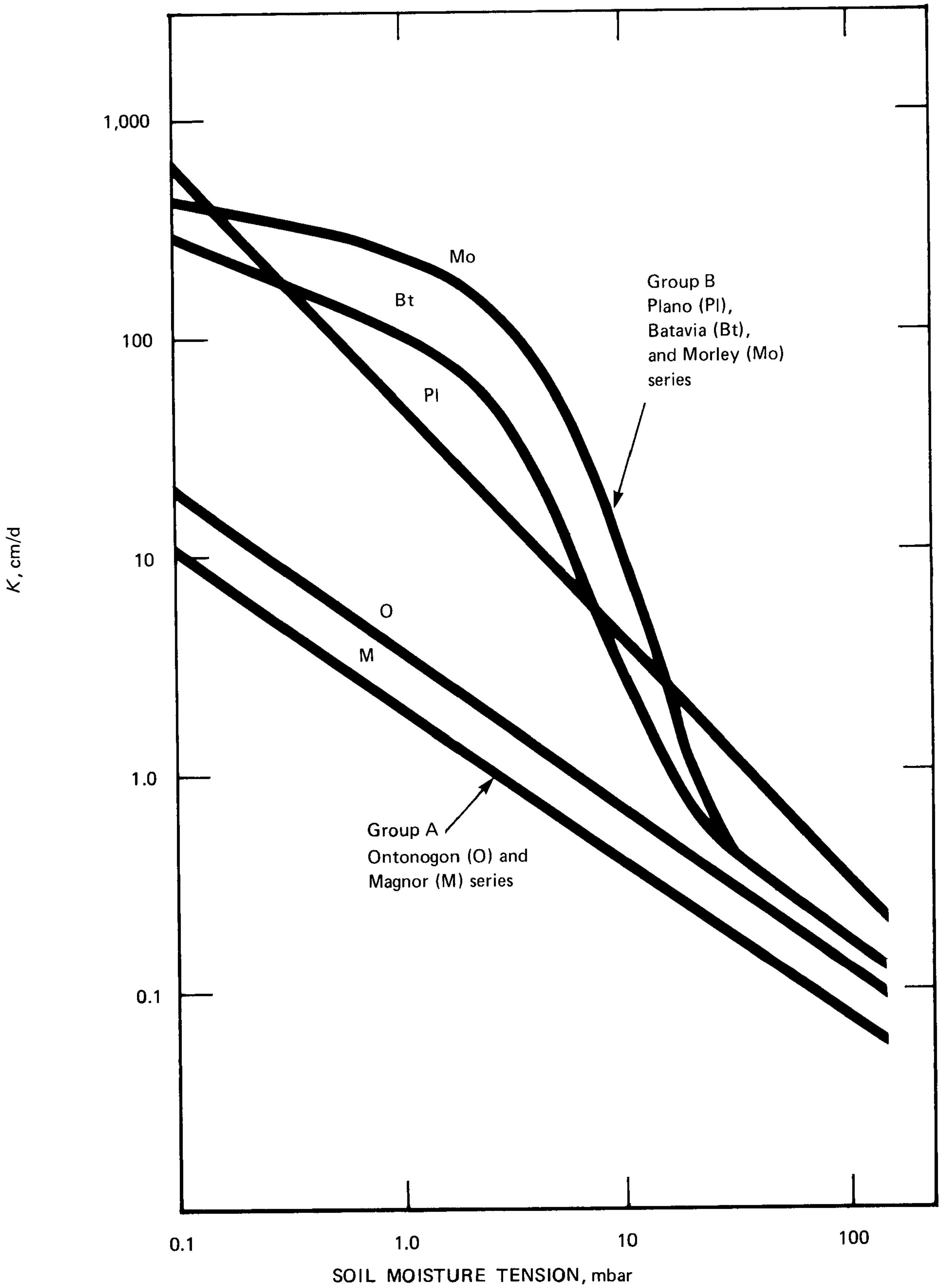


Figure I-15. Hydraulic conductivity (K) for Ontonogon and Magnor series and Plano, Batavia, and Morley series (after Baker[50]).

Table I-6.—*Summary of morphological characteristics of soil series from figure I-15*[50]

Soil identification	Horizon selected	Texture	Structure	Frequency of biopores
Group A:				
Ontonogon series:				
Very fine, illitic, frigid, Typic Eutroboralf	B2	Heavy loam	Moderate, medium angular blocky	Few coarse, common medium
Magnor series:				
Fine-loamy, mixed; Aquic Glossoboralf	IIB	Heavy loam	Moderate, medium subangular blocky	Few coarse, common medium
Group B:				
Plano series:				
Fine, silty, mixed, Mesic; Typic Arguidoll	B2	Silty clay loam	Moderate, medium subangular blocky	Few coarse, common medium and fine
Batavia series:				
Fine, silty, mixed Mesic; Mollic Hapludalf	B2	Silt clay loam	Moderate, medium subangular blocky	Few coarse, common medium and fine
Morley series:				
Fine, illitic, Mesic; Typic Hapludalf	IIB3	Heavy, silty clay loam	Moderate, medium prismatic, too coarse, blocky structure	Few coarse, common medium

Table I-7.—*Recommended maximum loading rates of septic-tank effluent for different soil types*

Estimated percolation rate, min/in	Soil texture	Loading rate[a]			Operating conditions[11]
		Bouma[11]		Machmeier[51]	
		cm/d	gal/d/ft^2	gal/d/ft^2	
0 to 10	Sand	5	1.23	1.20	4 doses per day, uniform distribution, trenches or beds
11 to 30	Sandy loams, loams	3	0.72	0.60	1 dose per day, uniform distribution, trenches preferred
31 to 45	Some porous silt loams, and silty clay loams[b]	3	0.72	0.50	1 dose per day, uniform distribution desirable, shallow trenches only
46 to 90	Clays, some compact silt loams and silty clay loams[b]	1	0.24	0.45	Dosing and uniform distribution desirable, shallow trenches only

[a]Bottom area only.
[b]Should not be applied to soils with expandable clays.

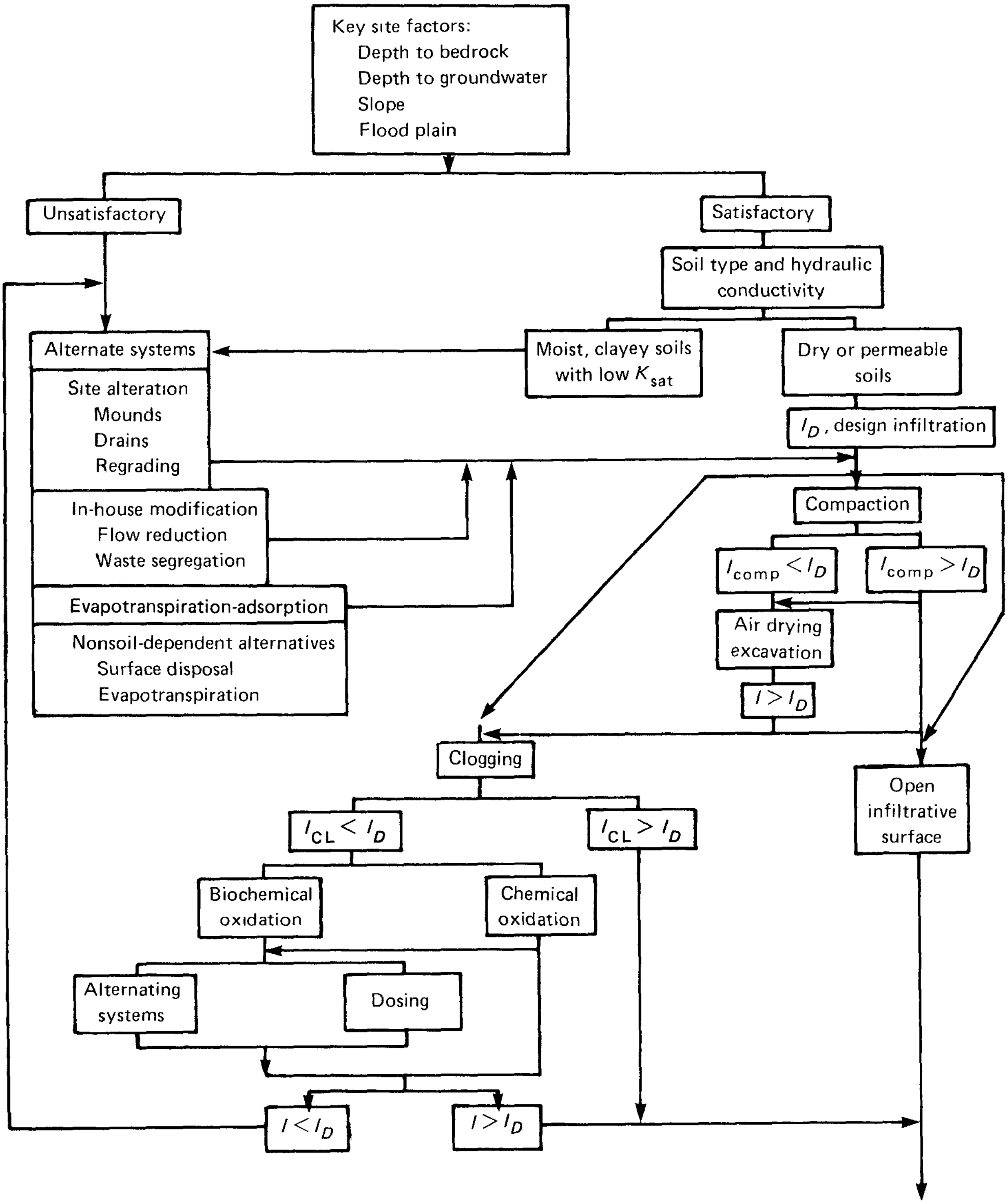
Soil mottling sometimes indicates seasonally high water levels. Mottles are spots of contrasting colors found in soils subject to periodic saturation. The spots are usually bright yellow-orange-red, have a gray-brown matrix, and are described according to color, frequency, size, and prominence.[53] Well-drained soils are usually brown in color because of finely divided insoluble iron and manganese oxide particles throughout the horizon. Under reducing conditions often produced by prolonged saturation, however, iron and manganese are mobilized until reoxidized when the soil drains. Repeated wetting and drying cycles quickly produce local concentrations of these oxides on pore surfaces, forming red mottles.[54] Soil from which much of the iron and manganese has been completely reduced, turns from brown to gray, a process referred to as gleying. The upper limit of the mottled soil, therefore, often is a good estimate of the high ground-water level, although it may also be the result of a periodic perched water table. The latter possibility can be confirmed by a lack of mottles in lower horizons.[55]

ON-SITE TREATMENT AND DISPOSAL SYSTEM ALTERNATIVES

There are numerous strategies, including soil-dependent and nonsoil-dependent systems, to consider for on-site treatment and disposal of wastewater. As has been discussed, however, soil can be an excellent medium if it is managed properly. Where suitable soil exists, it should usually be used in the system. Figure I-16[56] provides a useful chart for conceptualizing the boundary conditions of soils and techniques for maintaining infiltrative capacity.

Systems Dependent on Soil and Site Conditions

If a site satisfies all requirements for key factors (depth to bedrock, ground water, slope, flood plain), then soil type and *K* should be determined. If the soil is moist, slowly permeable, and clayey,



I = infiltration rate
I_{CL} = infiltration rate of clogged soil
I_D = infiltration rate (design)
I_{comp} = infiltration rate of compacted soil

Figure I-16. Strategies for on-site wastewater disposal (after Bouma[56]).

it is best not to construct absorption systems within the subsoil because irreparable damage may be done to the soil by compaction, smearing, and puddling. (An alternative strategy should be used such as constructing a mound over the natural soil that uses the soil for final disposal.)

Conventional ST-SAS. If the soil is dry or permeable, a system may be constructed to operate satisfactorily if the proper hydraulic loading rate is selected. The loading rates recommended in table I-7 are based on observations from properly functioning ST-SAS. To maintain this infiltrative rate for a reasonable span of 20 to 50 years, proper design, construction, and maintenance procedures must be followed as shown. If these procedures fail, then an alternative strategy must be investigated.

Estimation of Flow. Waste flows from homes, restaurants, motels, and so forth are intermittent and subject to wide fluctuations. Variations in the number of persons contributing to the flow and in their activities profoundly affect the daily volume of waste discharged, making accurate estimates of waste-flow volumes difficult.

A study of 11 rural homes showed the average per capita flow from a single household to be 43 gal/d.[57-59] The greatest flows result from laundering and bathing. (See fig. I-17.) Social

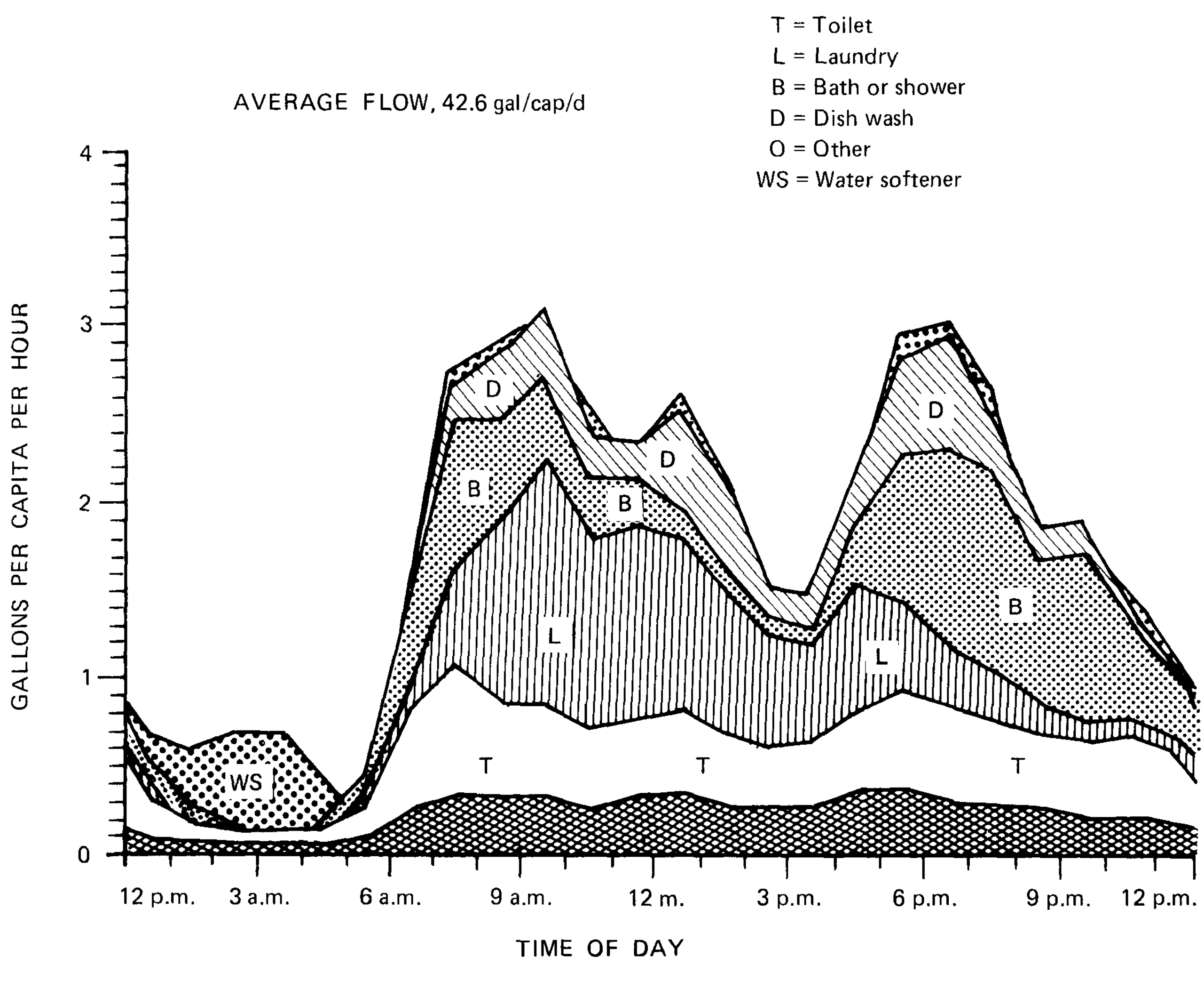


Figure I-17. Average daily flow pattern from 11 rural households.[57]

events, such as family gatherings and guests staying overnight, will accentuate the peak flows shown. In addition, the number of people living in a home may increase, through additions to the family or sale of the house. Thus, to ensure the system will not fail when large flows occur, it is necessary to design for the expected peak flows rather than for average per capita flow.

Household peak flows can be estimated by assuming two people occupy each bedroom. This is realistic because the number of occupants in a home is a function of the number of bedrooms. Figure I-17 shows that a peak flow of about 3 gal/hr per capita, or 75 gal/d per capita. Thus, 150 gal/d per bedroom gives a reasonable estimate of the peak flow. This is the design basis recommended by the USPHS,[42] and has proved satisfactory.

Estimation of flow from public buildings, commercial establishments, and recreational facilities is more difficult. The Small-Scale Waste Management Project (SSWMP) is determining daily and peak flows from bowling alleys, camps, churches, schools, country clubs, self-service laundries, marinas, motels, restaurants, service stations, shopping centers, theaters, and stadiums.

Sidewall Versus Bottom Absorption. A soil absorption system has two infiltrative surfaces: the bottom of the trench or bed and the sidewall. When the bottom begins to clog, the waste effluent ponds in the system and the sidewall begins to absorb liquid.[11] In some soils the sidewall may become the more significant infiltrative surface as clogging continues.[15,43]

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

Absorption systems should be designed to maximize the most significant infiltrative surface. In the Midwest, the bottoms 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 ET rates are low, and the matric potential is lowered because of soil wetness. Maximizing the infiltrative area by considering sidewalls as a reserve capacity is recommended, but in the Midwest, the bottoms should be sized to absorb the entire estimated daily flow.

Distribution of Liquid Over the Infiltrative Surface. Local overloading of septic tank effluent onto 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 soil types to eliminate poor purification and soil clogging. Sands and weakly structured sandy loams and loams benefit most.[60] After a system is put into service in natural sands, local overloading may cause ground-water contamination, which goes unnoticed until clogging develops. Development of a clogged zone may take several years. Excessive clogging caused by poor distribution tends to occur in weakly structured soils. Uniform distribution aids in reducing the clogging because the liquid is applied simultaneously to the entire infiltrative surface at rates no greater than the soil is able to accept.[61]

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 of 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 inlet and at points of lowest elevation. (See fig. I-18.)

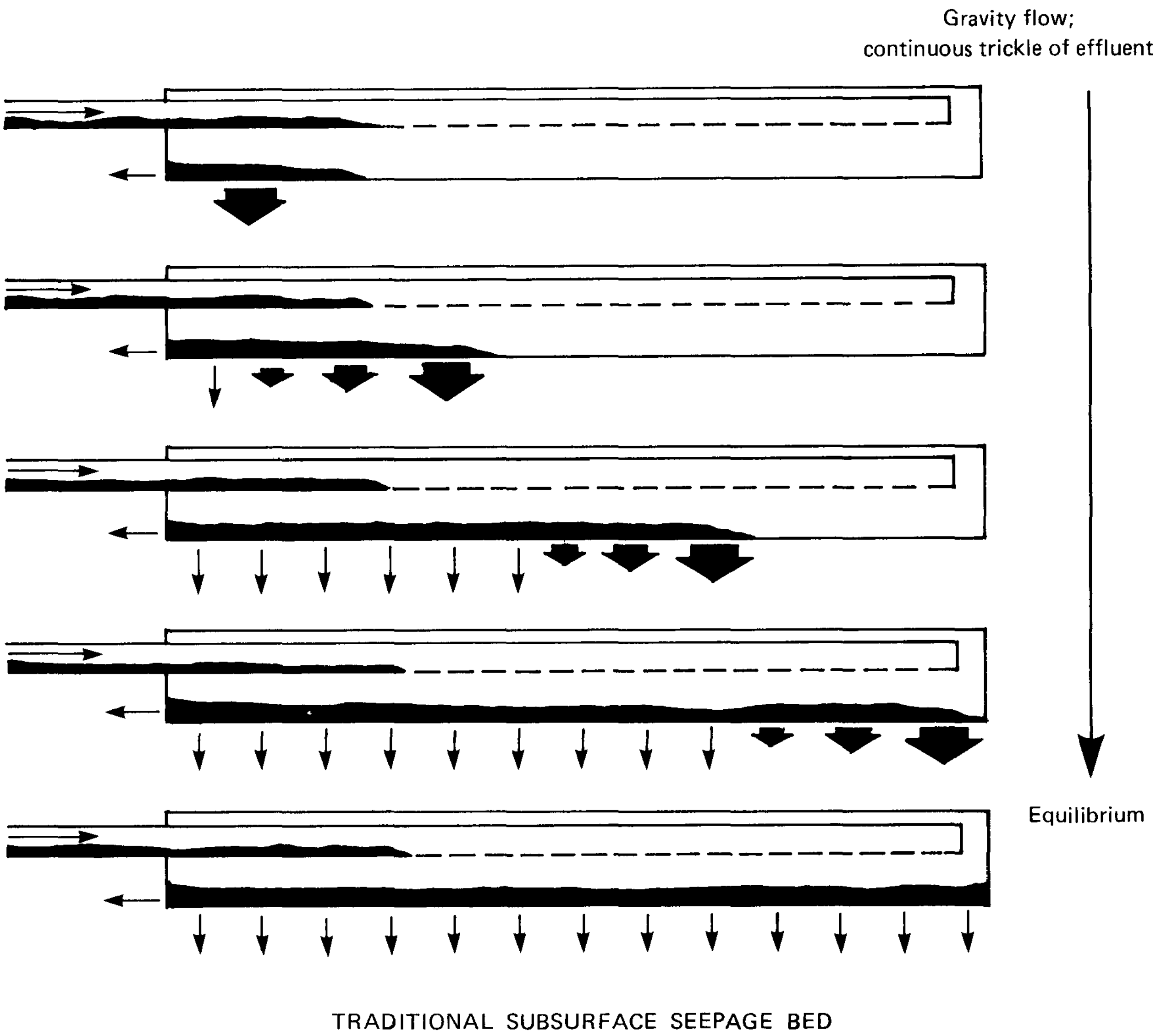


Figure I-18. Progressive clogging of the infiltrative surfaces of subsurface absorption systems.[9]

Clogging usually seems to start near the inlet of the absorption system and progresses down the length of the bed.[9] The large holes permit too much liquid to be discharged close to the inlet. Thus, the soil below receives a continual trickle of water and is soon constantly ponded. Clogging develops, forcing the liquid to infiltrate farther down the trench where the infiltrative surface is still fresh. This sequence continues until the entire bottom is clogged (fig. I-18). Altering the direction of the holes or the slope of the pipe does not improve distribution significantly.[62]

Periodic dosing of large volumes of effluent onto the field improves distribution and allows the soil to drain between applications. Drainage exposes the infiltrative surface to air, reducing clogging.[15,43,61] Even with dosing, however, the effluent is not distributed over the entire infiltrative surface if a 4-inch pipe is used.[62]

Pressure systems help provide uniform distribution. Networks of small-diameter pipes with small holes are designed so that the entire network fills before much liquid passes out of the holes, thereby achieving uniform distribution.[60-62] These systems combine uniform distribution with dosing, enhancing purification and reducing clogging.

Proper loading of permeable soils to prevent saturated flow is vital to ensure purification of the waste effluent. Pressure distribution systems provide this loading control. Conventional gravity distribution is ineffective.[60]

Pressure distribution systems also retard clogging. Because the network is designed to apply no more liquid than an area of the bed can absorb each day, the soil remains well aerated. Absorption fields in sand with pressure distribution have shown no evidence of clogging after 4 years of operation,[60] but fields in sand with conventional distribution begin to clog after 6 months.[9] The aerobic environment maintained by pressure systems promotes the growth of microorganisms that destroy clogging materials; it also appears to attract larger fauna, such as worms, which consume nutrients accumulating at the infiltrative surface. The worms' burrows help break up the clogging zone. Worm activity perhaps explains why an absorption field in a silt loam, underlaid with glacial fill and dosed with pressure distribution at three times the USPHS[42] recommended rate, has not clogged after 3 years' operation.[61] (See section on Pore Clogging.)

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

The following construction techniques are recommended to minimize soil clogging:[63,64]

- Work in clayey soils only when moisture content is low. If the soil forms a "wire" instead of breaking apart when rolled between the hands, it is too wet.
- Do not drive excavating equipment on the bottom of the system. Trenches rather than bed construction are preferable in clayey soils because equipment can straddle the trench, thus reducing compaction and smearing.
- Construct in shallow systems to place the infiltrative surface in more permeable horizons and to enhance ET. This is particularly beneficial in clayey soils because they are generally wetter for longer periods of time, especially at greater depths.
- Remove smeared or compacted surfaces. Compaction may extend as deep as 8 inches in clays; if so, hand spade to expose a fresh infiltrative surface.
- Schedule work only when the infiltrative surface can be covered in 1 day because windblown silt or raindrop impact can clog the soil.

Restoring the Infiltrative Capacity of a Clogged Absorption Field. Soil absorption systems often fail after several years of satisfactory service because the clogging zone 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.

One effective method is resting the system.[9,13,14,43] Resting allows the absorption field to gradually drain, exposing the clogged infiltrative surface to air. After several months, the clogging materials are broken down in 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 rests. Two beds, each with 50 to 75 percent of the total absorption area required, can be constructed when the disposal system is installed. The two beds can be used alternately by diverting the wastewater from one to the other every 6 months. If a system with one bed fails and a new bed is constructed, provisions should be made so that the old one is not abandoned but can easily be alternated with the new bed.

The infiltrative surface also can be rejuvenated by adding oxidizing agents to the absorption field. Oxidation serves the same function as resting, but the clogging zone is destroyed in a day or two rather than in several months. Such a method does not necessitate taking the clogged bed out of service; therefore, it eliminates the need for a second bed.

Laboratory and field tests indicate that chemical oxidation can restore the infiltrative surface to nearly its original permeability.[26] The preferred oxidant is hydrogen peroxide (H_2O_2) because it is effective at the natural pH of absorption fields, produces no noxious byproducts, and is inexpensive.

H_2O_2 treatment is best 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 using peroxide. Five gallons of 50 percent H_2O_2 solution may be sufficient to reduce the clogging developing in the bed. Because the field is still permeable, the oxidant can reach the clogging zone more easily than it could in a sealed system. It is best to perform tank pumping and peroxide treatment 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 H_2O_2 is in progress.

Mound Systems. The conventional septic tank-soil absorption field is not a suitable system of wastewater disposal in many areas, such as those with slowly permeable soils, excessively permeable soils, or soils over shallow bedrock or high ground water. Mound systems are alternatives, however, that can be used and that use the soils' ability to absorb and purify wastewater.[65]

Slowly Permeable Soils. Slowly permeable soils are 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 spring and fall. During these wet periods infiltrating surface water is unable to percolate fast enough through the subsoil, and flooding occurs from lateral movement of water from higher elevations through the topsoil.

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.[66] This raises the seepage system above the wet, slowly permeable subsoil and places it in a dry, permeable sand. (See fig. I-19.) This technique has several advantages. First, the percolating liquid enters the more permeable natural topsoil over a large area and can spread laterally until it is 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 compacting of the wet subsoil is avoided because excavation in the natural soil is not necessary.

The design of the mound is based on the expected daily wastewater volume it will receive and the natural soil's characteristics. The mound must be large enough to accept the daily wastewater flow without surface seepage in the spring and fall when perched water exists in the natural soil, as well as during the summer and winter when the water table is lower. Size and spacing of the seepage trenches is important to prevent 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 large enough to conduct the effluent into the underlying soil.

A clean, medium sand is used as fill for the mound; the gravel trenches consist of 1- to 1½-inch stones. As in any seepage trench, a clogging mat will develop at the bottom. The ultimate infiltration rate through this zone has been shown to be 5 cm/d.[11] One consideration, therefore, must be to ensure 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

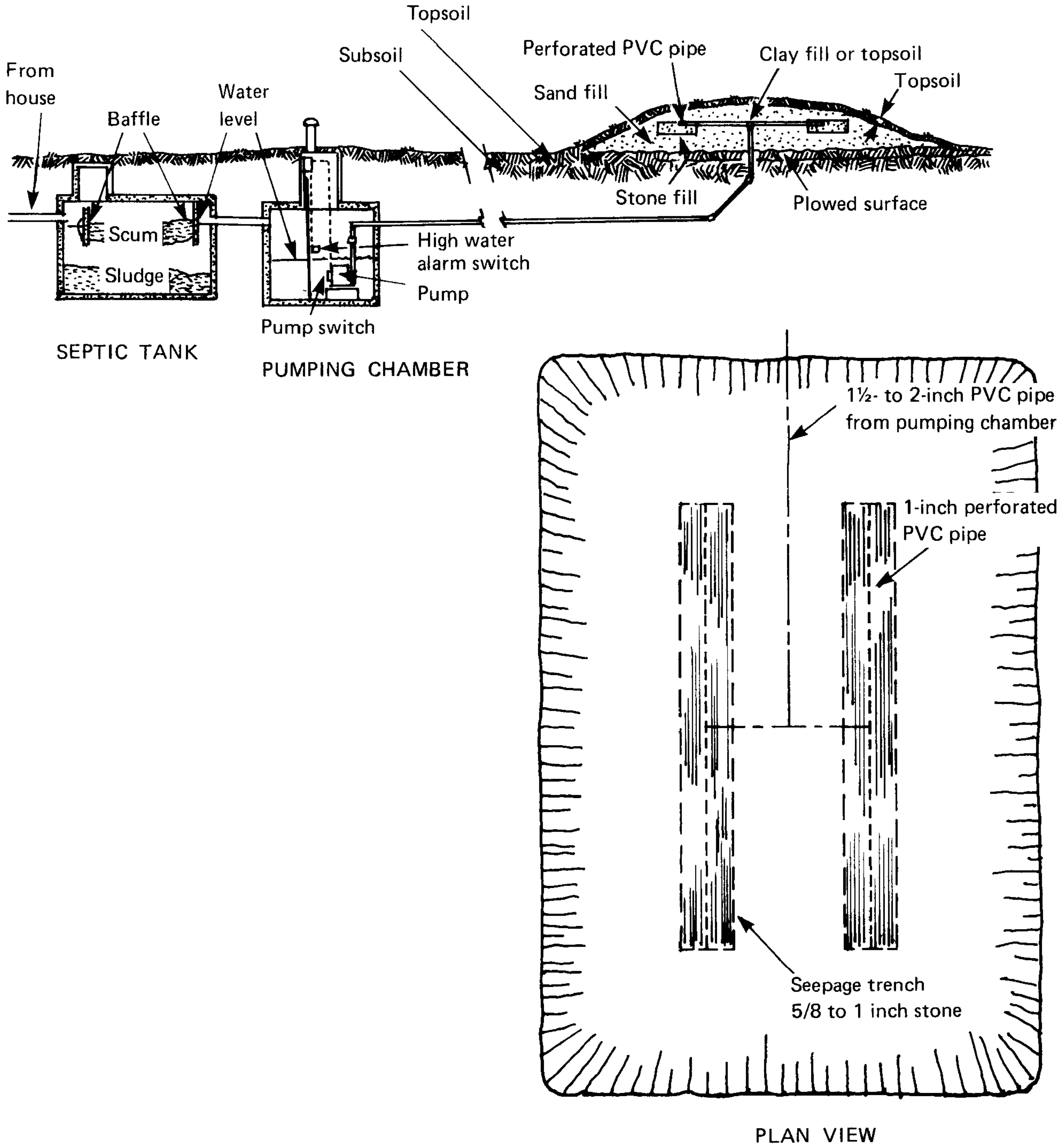


Figure I-19. Plan view and cross section of mound system for problem soils.

liquid contributed by the upslope trench. Infiltration rates into the natural soil and basal area required for the mound are based on the *K* characteristics of the least permeable soil horizon below the proposed site.

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.[67] Proper site criteria design and construction techniques, which have been described in detail,[66] are critical for satisfactory performance. Not all sites are acceptable for the mound design.

Shallow, Permeable Soils Over Creviced or Porous Bedrock. These soils constitute a major group of problem soils because adequate soil is not available to purify the percolating waste before it reaches the porous bedrock, which leads directly to the ground water. To overcome these limitations, the absorption field can be raised above the natural soil by using the mound system. (See fig. I-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 ground water.[29, 33, 68, 69] Nitrates, however, will not be removed.

The mound design procedures are the same as with slowly permeable soils. The seepage system within the fill may be of nearly any shape because 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[70] should be followed for proper operation of the mound.

Permeable Soils With Seasonally High Ground Water. Homes should not be built in areas with permanently high ground-water tables. In some areas, however, homes are built where the water table is only occasionally high during the year. During high water table periods, a conventional ST-SAS cannot function properly because of 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 ground water (fig. I-19). The mound design procedures are the same procedures as those used with 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 prevent the perched water table from entering the base of the mound. Detailed site criteria, design, and construction procedures are described elsewhere[71] and should be followed to ensure 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 maintain a sufficient depth of unsaturated soil for purifying the wastewater to avoid short circuiting.[72, 73] Such systems are being evaluated.

Modifying the Treated Wastewater Characteristics. Although the search for improved methods of on-site disposal has centered on the soil absorption system, more emphasis recently has been on altering the characteristics of the effluent discharged to the soil. Improving effluent quality has been said to enhance soil infiltration, reduce dependence on soils for final treatment, and eliminate the need for soil in the system.

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

Flow reduction to produce lower wastewater volumes can be accomplished through water conservation and recycling. Reductions can be achieved by improving water-use habits or by simple modifications in water-use appliances and plumbing fixtures. With less wastewater to treat and dispose of, the life of the on-site disposal would increase.

Nearly 70 percent of the total household wastewater generated is derived from toilet, laundry, and bath.[57] Using low-flow toilets, "sudsaver" washing machines, and restricted-flow shower heads and recycling bath and laundry wastes for toilet flushing are four commonly mentioned ways to save water. 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 17 percent in rural Wisconsin homes.[57] Recycling bath and laundry wastes to flush toilets could increase the savings to 33 percent. (See table I-8.) These savings compare well with values from other studies.[74,75]

Waste segregation to eliminate pollutants from the waste stream improves the quality of the wastewater. Analysis of wastewaters generated from rural households suggests which water-use events should be modified for the most beneficial results.[58,59] (See table I-9.)

Table I-8.—*Average calculated wastewater reductions in eleven rural homes*[57]

[All volumes in gallons per capita per day]

Use	Number per capita per day	Average volume of event	Normal volume	With 3 gallon flush	With sudsaver @ $27.86	With 15 gallon bath shower	With all three used	Recycled bath/ laundry to toilet
Toilet	2.29	3.99	9.16	6.87	9.16	9.16	6.87	—
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
Water softener	.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	—	—	42.60	40.31	40.73	39.65	35.49	28.62
Percent savings				5.00	4.00	7.00	17.00	33.00

Table I-9.—*Mean household wastewater contributions*[58]

[mg per capita per day]

Parameter	Toilet		Garbage disposal	Kitchen sink	Automatic dishwasher	Washing machine		Bath/ shower
	Fecal flush	Nonfecal flush				Wash	Rinse	
BOD_5 U	4,340	6,380	10,900	8,340	12,600	10,800	4,010	3,090
BOD_5 F	2,340	3,980	2,570	4,580	7,840	6,970	2,840	1,870
TOC U	3,530	4,250	7,320	5,000	7,280	7,700	2,610	1,750
TOC F	1,580	3,170	3,910	4,110	4,690	5,380	1,910	1,130
TS	10,700	17,800	25,800	13,800	18,200	37,500	10,900	4,590
TVS	7,760	12,000	24,000	9,730	10,500	14,700	4,800	3,600
TSS	6,240	6,280	15,800	4,110	5,270	7,930	3,040	2,260
TVSS	5,090	5,120	13,500	3,840	4,460	4,700	1,810	1,580
TOT-N	1,500	2,640	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	1,600	550	36
ORTHO-P	120	190	90	180	380	410	110	20
Temperature °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
Number sample	32 to 40	24 to 37	4 to 7	7 to 11	13 to 15	24 to 27	24 to 28	18 to 24

Recently, the concept of segregating toilet wastes (black water) from the other household wastewaters (gray waters) for separate treatment and disposal has drawn attention. Those planning development in water-short areas have raised serious questions regarding the use of valuable drinking water to transport body wastes and the comingling of black and gray waters before on-site treatment and disposal. Segregating black water from other household wastewater by using a nonwater-carrying 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 possible reductions are shown in table I-10.[57-59,74]

If the toilet wastes can be segregated and adequately disposed of, attention must then be directed toward the disposal of the gray water. Gray water is relatively uncontaminated, compared with black water. Gray water, however, contains substantial quantities of physical and chemical pollutants as well as pathogenic indicator organisms.[76] (See tables I-11 and I-12.)

Pollutant concentrations of gray water are similar to those of black water or combined wastes in rural homes.[59] Black water contains high concentrations of SS, nitrogen, and bacteria; gray water also contains sufficient quantities of pollutants and pathogenic indicators to cause concern.

Little research has been done on treatment and disposal of household gray water. One method is the ST-SAS, but simple alternatives might be more desirable in certain applications. The SSWMP is evaluating alternative methods for gray water treatment and disposal.

Reducing flow or waste strength may increase the life of a soil absorption field, but it is not known by what factor. For new installations, smaller absorption fields could perhaps be allowed, if water-saving devices were used. Unless assurances are made that flow reduction or waste-segregation facilities could not fail or be removed, however, reducing the size of the field is not recommended.

Effluent Quality and Soil Clogging. Improving 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 SS and 5-day biochemical oxygen demand (BOD_5) concentrations in the effluent.[77] The relationship found was:

$$\text{Adjusted area required} = \text{Area required for standard septic tank system} \times 3\ BOD_5 + TSS/250$$

Assuming an average effluent quality of 40 mg/l BOD_5 and 40 mg/l SS, the calculated adjusted absorption area is approximately two-thirds of the standard area.

In several other studies[23,50,78] on effects of effluent quality on soil infiltration, however, only slight differences in clogging rates were found over a range of qualities tested. These differences were small in hand-packed lysimeters of sand and sandy loam loaded with septic tank effluent (74 mg/l BOD_5 and 51 mg/l SS) and aerobic unit effluent (81 mg/l BOD_5 and 75 mg/l SS).[78] The

Table I-10.—*Effect of toilet waste segregation on household wastewater*

Parameter[a]	Percent reduction
Flow	22 to 31
BOD_5	22 to 49
SS	36 to 67
Total phosphorus	14 to 42
Total Kjeldahl nitrogen	68 to 99

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

Table I-11.—*Pollutant contributions by black and gray wastewater streams*[76]

Parameter	Gray				Black			
	Mean, percent	Range, percent	Mean		Mean, percent	Range, percent	Mean	
			(gal/cap/day)	(mg/l)			(gal/cap/day)	(mg/l)
BOD_5	63	51-80	28.5	255	37	20-49	16.7	280
SS	39	23-64	17.2	155	61	36-77	27.0	450
Nitrogen	18	1-33	1.9	17	82	67-99	8.7	145
Phosphorus	70	58-86	2.8	25	30	14-42	1.2	20
Flow	65	53-81	29.4	—	35	19-47	15.9	—

Note.—Values based on results of several studies.[59,74,75] Results are average values for households with typical conventional appliances, excluding the garbage disposal.

Table I-12.—*Selected bacteriological characteristics of bath and laundry wastewaters*[76]

Event	Bacteria	Data points	Mean[a] number per 100 ml	95 percent confidence interval per 100 ml
Bath/shower	Total coliforms	32	1,810	530 to 6,160
	Fecal coliforms	32	1,210	330 to 4,410
	Fecal streptococci	32	326	70 to 1,510
Laundry	Total coliforms	41	215	45 to 1,020
	Fecal coliforms	41	107	28 to 405
	Fecal streptococci	41	77	19 to 305

[a]Log-normal data.

aerobic unit effluent produced earlier, but less intense, clogging in the sand; the reverse was true in the sandy loam. After resting, the soils receiving aerobic unit effluent recovered more quickly. In general, however, there was little difference in 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.[23] The aerobic effluent had a significantly lower biodegradable organic concentration than the septic tank effluent-chemical oxygen demand (COD) concentrations of 150 mg/l and 60 mg/l, respectively—but SS 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. It was hypothesized that finely divided SS in the aerobically treated wastewater entered the small pores in the soil, clogging it with depth and creating a more effective barrier to flow.

Subsequent studies designed to test the solids clogging hypothesis found initial K_{sat} to be more significant than effluent quality.[79] Undisturbed cores of Almena silt loam were paired according to their initial K_{sat}; one pair represented a high and one a low initial K_{sat}. Three sets of four replicates each were dosed with 1 cm/d of septic tank effluent (48 mg/l BOD_5, 27 mg/l total suspended solids (TSS)), aerobic unit effluent (27 mg/l BOD_5, 61 mg/l TSS), and tapwater. The length of time until 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 tapwater 18.3 weeks. When the three sets' initial K_{sat} values were compared, mean ponding time for the high K_{sat} columns was 28 weeks and for the low K_{sat} columns was 14.8 weeks.

These studies 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.

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 on soil disposal, but which discharge the treated wastewater to surface waters or the atmosphere, are necessary.

Systems designed to discharge treated wastewaters to surface waters must produce a high-quality effluent. The EPA has set concentration maximums of 30 mg/l BOD_5 and 30 mg/l SS for municipal treatment plants that discharge to water courses. Lower maximums might be set for scattered individual systems that discharge to small, intermittent streams. It might be expected that bacteria counts in effluents from individual systems could not exceed total and fecal coliform maximums of 1,000/100 ml and 200/100 ml, respectively, recommended for recreational waters.[80] Limitations on the nutrients nitrogen and phosphorus are likely to be required for discharges to lakes or impoundments.

Aerobic Processes. Although a number of the options outlined have been field evaluated, many others have not been tried with small flows. Aerobic treatment processes have received the greatest attention as an alternative to the septic tank. More than 75 years' experience with this biological process in larger scale applications makes it a logical candidate for small-flow, on-site treatment.

Small trickling filters were among the first controlled aerobic processes for household wastewater treatment. Nichols[81] described the use of aerated pebble filters following septic tanks, and Frank and Rhymus,[82] 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 reported with fixed-media biological filters, but the potential of rotary and stationary synthetic filters is most promising. The SSWMP's most recent experience with a rotating biological contactor has not been good, owing primarily to shaft breakage in laboratory and field units.[83] An anaerobic, submerged-media system, initially designed for ships, has proved very stable and relatively maintenance-free.[83]

The first notable research on adapting the activated sludge process (extended aeration) to household use was conducted at Purdue University in the early 1950's.[84] A very simple prototype, receiving toilet wastes only, produced an effluent with average BOD_5 and SS concentrations of 28 mg/l and 42 mg/l, respectively. Similar studies were conducted at Ohio State University using a proprietary extended aeration package plant. Average BOD_5 and SS concentrations of 24 mg/l and 43 mg/l, respectively, and relative trouble-free service were reported during a 23-month period.[85]

In 1970, the National Sanitation Foundation (NSF) issued its "Standard No. 40" on individual aerobic treatment units.[86] It outlines criteria for evaluating such units and presents a procedure for testing and certification. Several States require NSF certification for aerobic units.

In addition to controlled studies of aerobic units, there are several reports on how they function under actual conditions.[87-90] 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.[88] Their findings indicated a mean value for BOD_5 and SS of 150 mg/l. They cited lack of proper maintenance and an adverse effect of surge flows for this poor performance. Voell and Vance[89] provided data on a large number of field-operated aerobic units in a New York county. Average values for BOD_5 and SS were about 90 mg/l. The lack of proper maintenance was again cited. Six aerobic treatment units were installed and monitored over an 8-month period by Glasser.[90] Average values for BOD_5 and SS were reported to be 48 mg/l and 85 mg/l, respectively. Glasser recommended maintenance and supervision at least four times per year.

The performance of several aerobic units of different designs has been compared with septic tanks operating under both laboratory and field conditions.[87,91] Two years' evaluation showed aerobic-unit removals of biodegradable organic material from the waste were significantly higher than those achieved by septic tanks; SS concentrations in all effluents were nearly identical. (See figs. I-20 and I-21; table I-13.)[92] The septic tanks were more stable, however. Periodic upsets resulted in substantial variability in aerobic-unit effluent quality.

Periodic carryover of solids was the major reason for effluent quality deterioration from aerobic units. Bulking sludge (sludge that will not settle), toxic chemical additions from the home, and instability because of excessive buildups of sludge, seemed to be the most common causes of carryover. Several design modifications have been suggested to help prevent some operational problems,

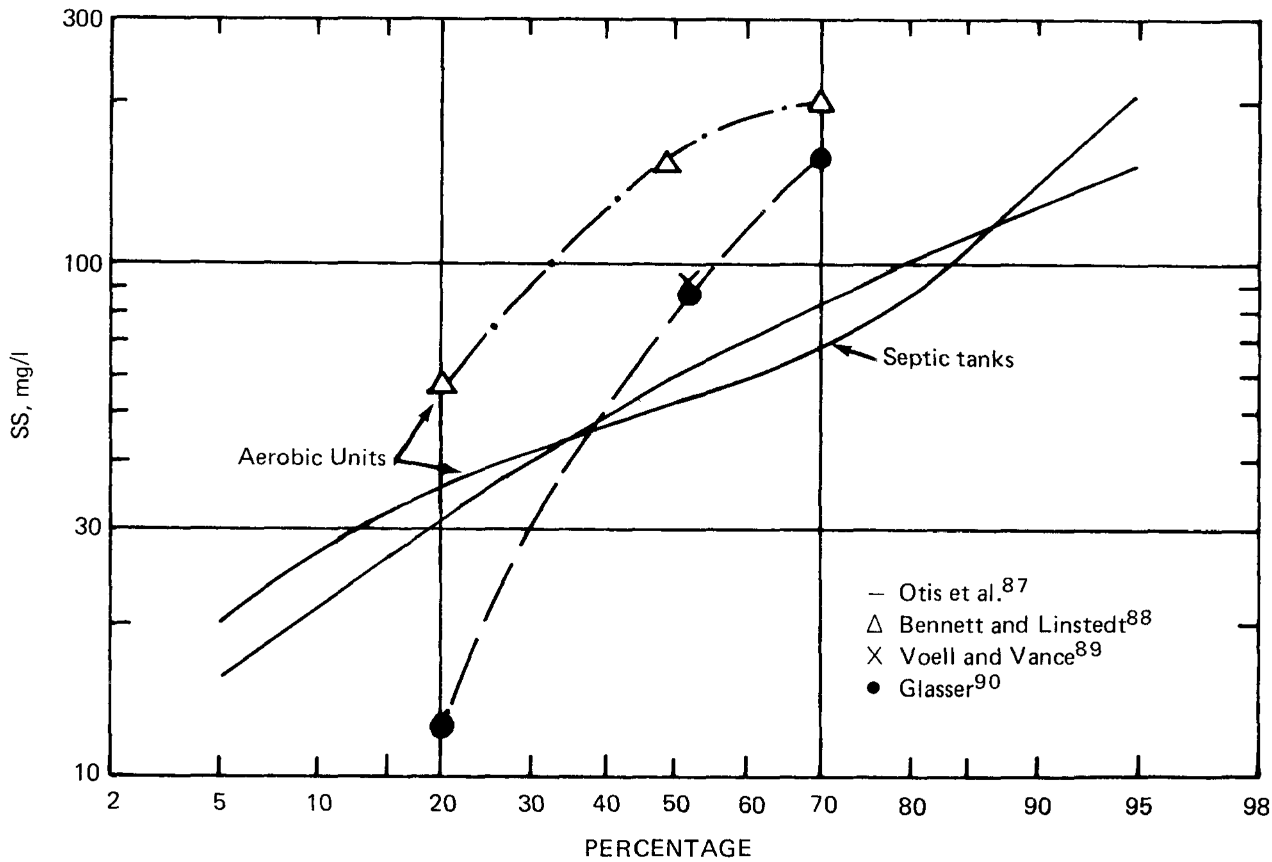


Figure I-20. Comparison of septic tank and aerobic unit effluent concentrations of suspended solids (SS).

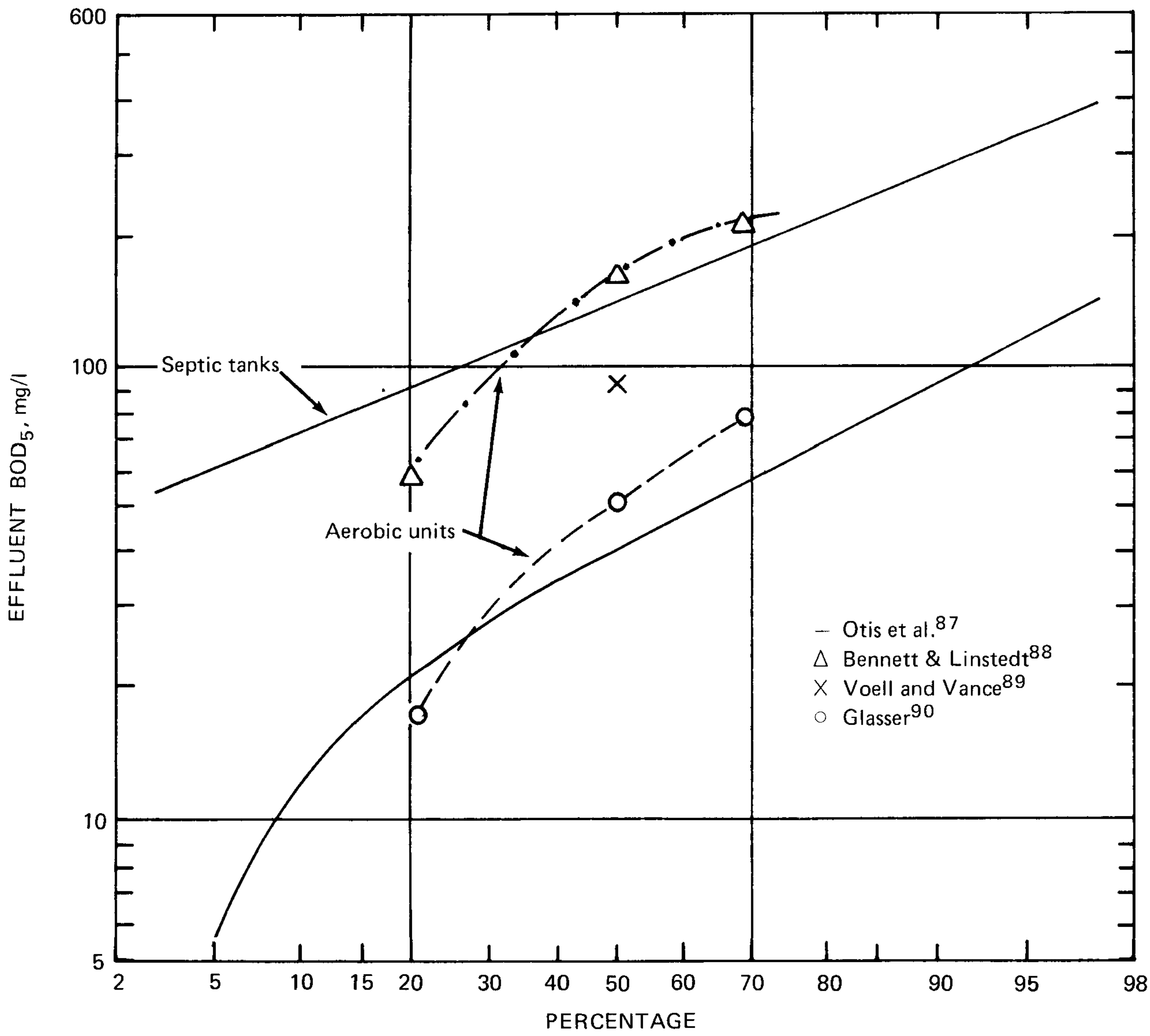


Figure I-21. Comparison of septic tank and aerobic effluent concentrations of biodegradable organic material.

Table I-13.—*Comparison of septic tank and aerobic unit effluents*

Characteristic	Treatment system	
	Aerobic units	Septic tank
BOD_5, mg/l:		
Mean	47 (63)[a]	158 (94)[a]
Coefficient of variant	0.79	0.50
95-percent confidence interval	38 to 57	142-174
Range	0 to 208	20-480
COD (unfiltered), mg/l:		
Mean	136 (69)[a]	360 (97)[a]
Coefficient of variant	0.45	0.36
95-percent confidence interval	121-150	335 to 386
Range	26 to 349	66 to 780

Table I-13.--*Comparison of septic tank and aerobic unit effluents*—Continued

Characteristic	Treatment system	
	Aerobic units	Septic tank
COD (filtered), mg/l:		
Mean	75 (68)[a]	285 (93)[a]
Coefficient of variant	0.52	0.36
95-percent confidence interval	65 to 84	264 to 306
Range	7 to 210	47 to 531
TSS[b], mg/l:		
Mean	53 (69)[a]	54 (93)[a]
Coefficient of variant	0.19	0.17
95-percent confidence interval	45 to 64	47 to 62
Range	4 to 252	11 to 695
FC[b]:		
Mean	107 (67)[a]	4,210 (94)[a]
Coefficient of variant	0.32	0.22
95-percent confidence interval	74 to 153	2,879 to 6,158
Range	1 to 2,200	5 to 180,000
FS[b]:		
Mean	32.9 (70)[a]	38.2 (97)[a]
Coefficient	0.44	0.87
95-percent confidence interval	22.7 to 47.6	20.1 to 72.4
Range	1 to 1,930	0 to 11,200
Total nitrogen, mg-N/l:		
Mean	37.6 (38)[a]	55.3 (53)[a]
Coefficient of variant	0.36	0.42
95-percent confidence interval	32.2 to 42	48.9 to 61.6
Range	15.8 to 77.6	9.7 to 124.9
Ammonium-N, mg-N/l:		
Mean	.02[b] (44)[a]	38.7 (63)[a]
Coefficient of variant	1.10	0.45
95-percent confident interval	.01 to .08	34.3 to 43
Range	0 to .08	.1 to 90.7
Nitrite-nitrate-N, mg-N/l:		
Mean	30.1 (46)[a]	0.56[b] (67)[a]
Coefficient of variant	0.52	2.63
95-percent confidence interval	25.5 to 34.8	.39 to .82
Range	.3 to 71.6	0 to 74.5
Total phosphorus, mg-P/l:		
Mean	35.2 (36)[a]	14.6 (54)[a]
Coefficient of variant	.69	.80
95-percent confidence interval	27 to 43.5	11.4 to 17.7
Range	6.8 to 140	3.8 to 90
Orthophosphate, mg-P/l·		
Mean	28.9 (32)[a]	11.5 (43)[a]
Coefficient of variant	.42	.38
95-percent confidence interval	24.5 to 33.3	10.2 to 12.8
Range	6.8 to 51.2	20.4

[a]Number of samples.
[b]Log-normal distribution.

but regular servicing is necessary to ensure proper functioning.[87] Inspections should be made at least once every 2 months and excess solids pumped every 8 to 12 months.[87, 91]

Intermittent Granular Filtration. Field experience with on-site aerobic treatment processes indicates that additional polishing of effluents will be necessary before surface discharge to meet current EPA effluent guidelines. Filtration appears to be one of the most promising alternatives available for polishing. Whether the filtration is provided by granular beds or by mechanical filter systems used as a part of the biological process or as a separate process depends on economics, effectiveness, and maintenance requirements.

Granular filtration seems particularly well suited to on-site system design. At least three basic flow configurations have been successfully tested in the field: the buried sand filter, the recirculating sand filter, and the intermittent sand filter.

The buried sand filter is constructed below the soil surface. A bed is excavated and underdrain collectors are installed and covered with approximately 12 inches of gravel. Normally, 24 inches of sand (usually greater than 0.4 mm effective size with a uniformity coefficient of less than 4.0) are placed over the gravel, followed by influent drain tile in 10 to 12 inches of gravel. The bed is covered with at least 6 inches of topsoil. Allowable wastewater loadings vary from State to State but range between 0.75 and 1.5 gal/d/ft^2, depending on appliance loading and on pretreatment. These systems usually perform excellently unless overloaded, but the bed's inaccessibility dictates a more conservative sizing than might be otherwise used for open systems.

The recirculating sand filter system consists of a septic tank, a recirculation tank, and an open sand filter.[93] Wastewater is dosed onto the filter by a submersible pump in the recirculation tank. The sump pump is activated by a timer and is sized to pump 5 to 10 gallons per minute for single households. A recirculation ratio of 4:1 (recycle to forward flow) is recommended. The recirculation tank, usually 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 before recycling. Filter-effluent recycled flow is controlled by a floating rubber 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 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 gal/d/ft^2, based on raw septic tank flow. It is estimated that approximately 1 inch of sand should be removed once a year to avoid serious ponding conditions. After 12 inches of sand have been removed, new sand should be added. Results of a household system study indicate that effluent BOD_5 values average less than 5 mg/l and TSS values less than 6 mg/l.[93]

In the intermittent sand filter, pretreated wastewater is applied over a bed of sand 2- to 3-feet deep and the filtrate collected by underdrains. The sand remains aerobic and serves as a biological filter, removing SS and dissolved organics. A summary of filter performance based on a review of the literature is given in figure I-22.[94]

Filters receiving septic tank or aerobic unit effluent have been tested under field and laboratory conditions. A typical filter system is depicted in figure I-23. Of major concern in sizing intermittent granular filters are the trade-offs between effluent quality and maintenance requirements (see fig. I-22).

The quality of sand-filtered septic tank and aerobic unit effluents appears in tables I-14 and I-15 for field systems operated for over 2 years.[95, 96] Relatively little difference is shown between aerobic-unit, sand-filtered effluent and septic-tank, sand-filtered effluent for comparable loading

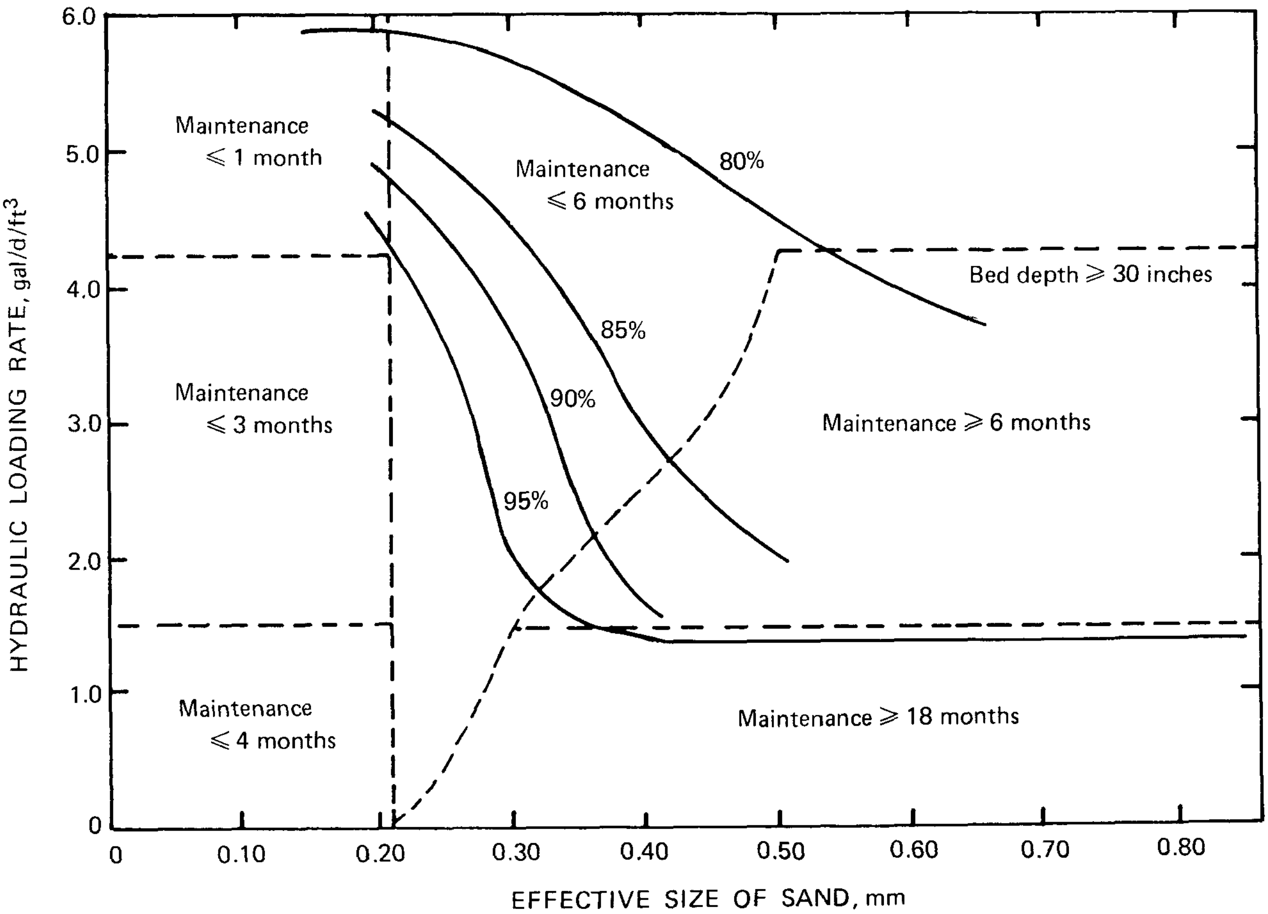


Figure I-22. Trends in percent of BOD reduction and required maintenance of intermittent sand filters treating septic tank wastewater (average BOD, 94 mg/l).[94]

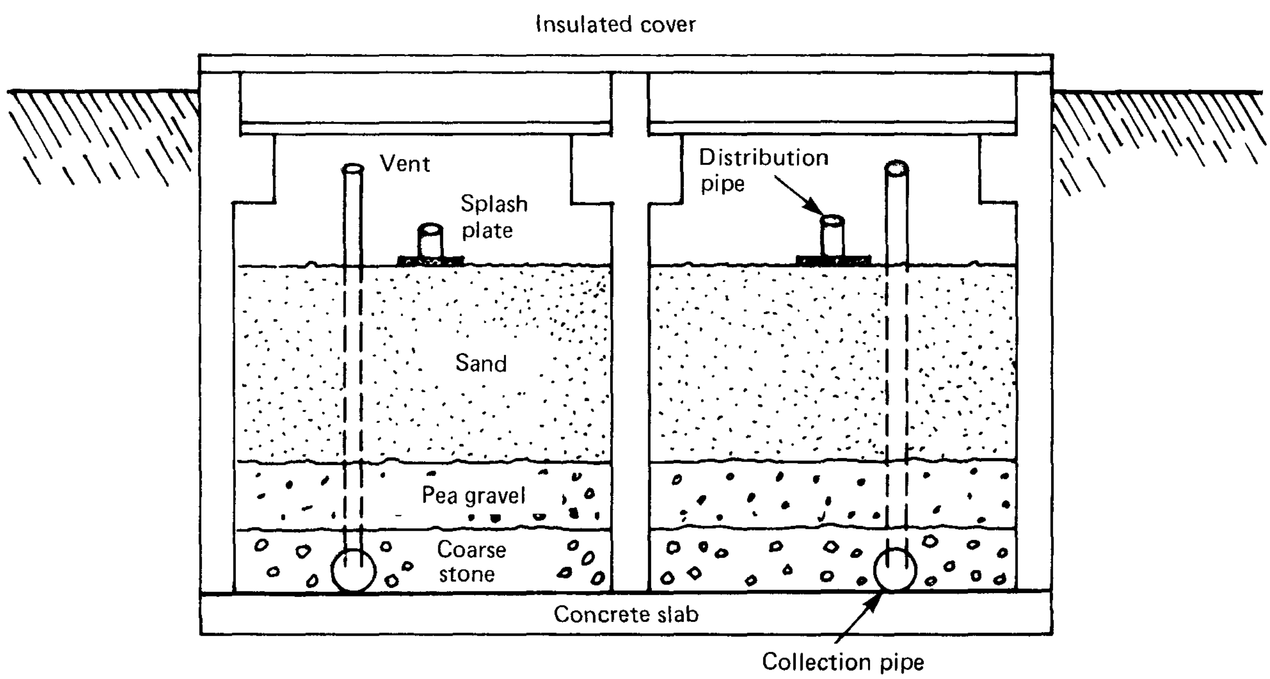


Figure I-23. Profile of intermittent sand filter.

Table I-14.—*Septic tank-sand filter effluent quality data*

Parameter	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-nitrogen, mg/l	19.2	1.0	1.6
Nitrate-nitrogen, mg/l	.3	20.0	18.9
Total phosphorus (P), mg/l	10.2	9.0	8.4
Orthophosphate-phosphorus, mg/l	8.7	7.0	7.9
Fecal coliforms (number per 100 ml)	5.9×10^5	6.5×10^3	2
Total coliforms (number per 100 ml)	9.0×10^5	1.3×10^3	3

Note.—Loading rate average, 5 gal/d/ft^2 (0.2 m/d; effective size, 0.45 mm; uniformity coefficient, 3.0.

Table I-15.—*Aerobic unit-sand filter effluent quality data*

Parameter	Aerobic unit effluent	Sand filter effluent	Chlorinated effluent
BOD_5 mg/l	31.0	3.5	4.0
TSS, mg/l	41.0	9.4	7.0
Total nitrogen (N), mg/l	37.8	34.8	38.3
Ammonia-nitrogen, mg/l	1.4	0.3	0.4
Nitrate-nitrogen, mg/l	32.3	33.8	37.6
Total phosphorus (P), mg/l	29.5	20.3	24.0
Orthophosphate-phosphorus, mg/l	25.0	18.9	23.4
Fecal coliforms $\log_{10}$#/l	5.3	4.0	0.9
Fecal streptococci $\log_{10}$#/l	4.4	3.2	1.5

Note.—Loading rate average, 3.8 gal/day/ft^2 (0.15 m/day); effective size, 0.19 mm; uniformity coefficient, 3.31.

conditions, although the aerobic system used a finer sand (0.19 mm, compared with 0.45 mm). Such filtered effluents could meet current EPA standards for BOD and TSS but would require further pretreatment for coliforms or phosphorus. Excellent ammonia conversion is produced by both systems.

Filter runs depend on 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, effluent-loaded filters.[83,95,96] Recommended filter operation schedules for a septic tank-sand filter system are presented in table I-16. It is recommended that there be two alternating filters, each designed for a hydraulic loading rate of 5 gal/d/ft^2. When one filter ponds, it can be taken out of service, raked to a depth of 2 to 4 inches, and rested before wastewater is reapplied. After a second loading period, the top 4 inches of sand from the filter should be replaced with clean sand.

Aerobic-unit, sand-filter systems do not normally require a second filter.[96] Suggested application rate is 5 gal/d/ft^2. Removing the solids mat and 1 inch of sand and adding 1 inch of clean sand are the only required maintenance steps. These should be done every 6 months. Wastewater can be reapplied immediately after maintenance. Experience shows that periodic biological and hydraulic

Table I-16.—*Septic tank-sand filter operation schedule*

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

upsets of the biological process can be handled by the sand filters; however, extended periods of upset will lead to shorter filter runs.

Disinfection Alternatives. If the final effluent requires disinfecting, several alternative systems have proved effective. The use of dry feed chlorinators will normally produce effluents that meet EPA standards (tables I-14 and I-15). Unfortunately, a major problem associated with the use of dry feed chlorinators is controlling the dose to the wastewater. High chlorine concentrations were found periodically during extended field testing.[94] There are ways to effectively control hypochlorite feed, however.[83] In light of the toxicity of chlorine, consideration must also be given to dechlorination of effluent before final surface water discharge.

Initial studies with ultraviolet (UV) irradiation of sand-filtered household effluents have been most promising. Four months of operating data with a commercial UV unit are presented in table I-17.[83] Long-term field tests are continuing with these units. One major drawback to UV irradiation is high initial capital investment. As greater demand for this type of system increases, however, costs are likely to decrease.

Iodine as a disinfectant for on-lot disposal may prove very practical. Because iodine is only slightly soluble in water, a saturator holding iodine crystals can serve as the solution feed device. The appropriate aliquot of feed solution added to the treated wastewater can be controlled by a pump, electronic signal, or pressure valve. Iodine is an excellent bactericide, virucide, and cysticide. Also, it does not combine with ammonia or react readily with other trace organics in wastewaters. Iodine is expensive, but proper feed control should result in cost-effective disinfection for on-lot treatment.[83]

Other disinfectants include bromine salts, formaldehyde, and ozone. The feeding of bromine salts appears to be too complex for small-flows application; ozone treatment also involves relatively complex equipment. Little experience has 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 mainte-

Table I-17.—*Coliform analysis of effluents from sand filter and ultraviolet (UV) units*

[*N* per 100 ml]

Coliform	Aerobic unit		Septic tank	
	Sand filter	Ultraviolet	Sand filter	Ultraviolet
Fecal	11 to 13	<1	$(2.6\text{-}4.4) \times 10^3$	<1
Total	64 to 75	<1	$(3.6\text{-}5.1) \times 10^3$	<1

nance is relatively high.[91] The use of ion exchange and carbon adsorption techniques is feasible, but maintenance and operation requirements are high, as are costs. Treatment packages using a number of these unit operations are being field tested at the University of Wisconsin on both combined and gray water effluents from households.

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

If evaporation is to be continuous, three conditions must be met.[97] First, there must be a continuous supply of heat to meet the latent heat requirement (approximately 590 cal/g 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 by diffusion, convection, or both. Meteorological factors, such as air temperature, humidity, wind velocity, radiation, and vegetative cover, influence both energy supply and vapor removal. Energy can also be added from heat in the water itself or from biological activity. Third, there must be a continuous supply of water to the evaporative surface. This depends on the soil's matric potential and *K*. The soil material must be fine textured enough to draw up the water 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.

A typical ET bed system consists of a 1½- to 3-foot depth of selected sand over an impermeable plastic liner.[98] A perforated plastic piping system with rock cover is often used to distribute septic tank effluent in the bed. The bed may be square-shaped on relatively flat land, or a series of trenches on slopes. A cross section of a typical bed is depicted in figure I-24. The surface area of the bed must be large enough for sufficient ET to occur to prevent the water level in the bed from rising to the surface. This means that annual evaporation rate must be significantly higher than 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 uniform sand in the size

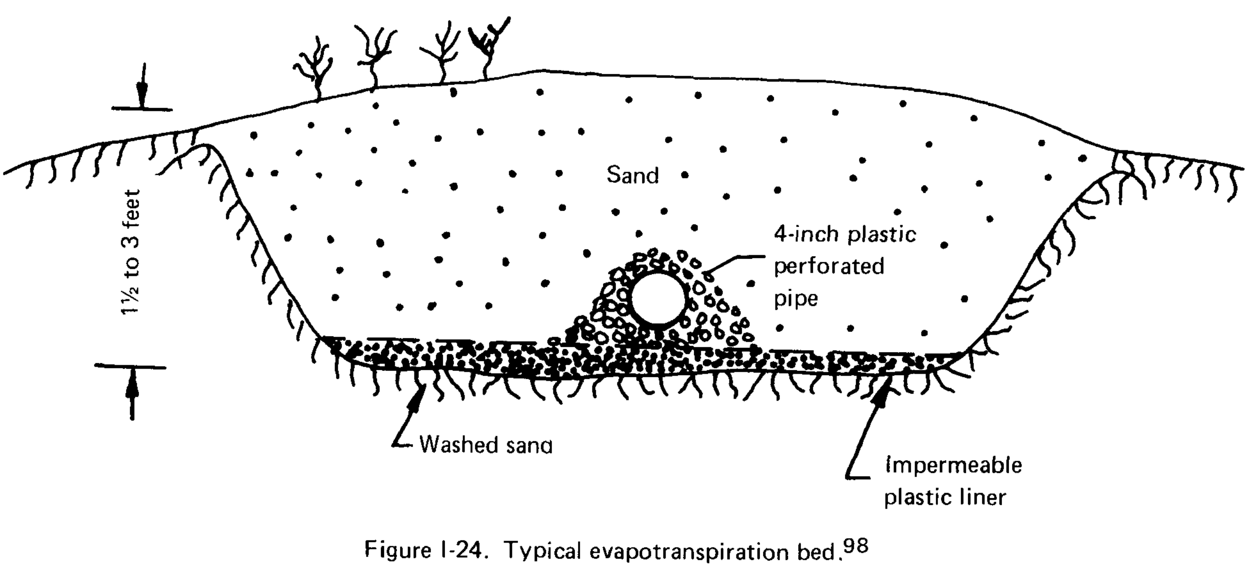


Figure I-24. Typical evapotranspiration bed.[98]

range of D_{50} (0.10 mm). This size sand is capable of raising water about 3 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 vegetation on the disposal field. Trees and bushes with large silhouettes received more advective heat. When vegetation is dormant, ET is greatly reduced. Snow cover reflects solar radiation, which reduces ET. In addition, when temperatures are below freezing more heat is required to change frozen water to vapor. Thus, care must be taken in selecting a site. A procedure has been outlined to estimate the maximum ET that can be expected from disposal fields east of the Mississippi River.[99]

The Department of Civil, Environmental and Architectural Engineering at the University of Colorado at Boulder[98] is conducting a study of design parameters for nondischarging ET systems. The study involves 28 outdoor lysimeter units, each 2 feet in diameter and 28 inches deep. Several full-scale ET systems in use at private homes are also being monitored so that data can be correlated.

Design of an ET bed is based on the local annual weather cycle. Evaporation rates are highest during the summer, but the Colorado study shows that winter evaporation rates are extremely important to the functioning of the system. Summer evaporation has been found to be approximately 40 percent of the pan value; winter values are about 70 percent. The average design evaporation value can be established from the annual pattern, as shown in figure I-25. This rate can be matched with the total expected inflow based on household wastewater generation rate and on rainfall. A mass diagram is used to establish the storage requirements of the bed.[98]

Vegetative cover can increase the ET rate during the summer growing season; but if this increased rate is to be used, the bed must provide additional storage in winter. Lawn grass has been found to increase evaporation rates slightly during June, July, and August; in winter, when the ground is bare, evaporation rates are reduced.

Alfalfa can produce an evaporation rate of 0.6 gal/d/ft^2 at peak growing season. Design year-round sewage ET rates have been found in the range of 0.04 gal/d/ft^2 of bed in the Boulder, Colo., area.[98] This results in a bed area of approximately 5,000 square feet for one 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. In practice, nondischarging ET bed systems are limited to areas of the country where pan evaporation exceeds annual rainfall by at least 24 inches and where winter evaporation exceeds precipitation by a value of 2 inches every month. The decrease of ET in winter at middle and high latitudes greatly limits its use; under freezing conditions ET would be inadequate.[99]

Evapotranspiration disposal systems could work in semiarid regions, such as those in 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.[98]

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.[98] When the costs of the septic tank and piping are included, the total system costs range from $3,000 to $5,000 per house (about $1.00 per square foot). Studies are under way at the University of Colorado to develop a mechanical wastewater evaporation system that will have a greater range of application.[98] 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

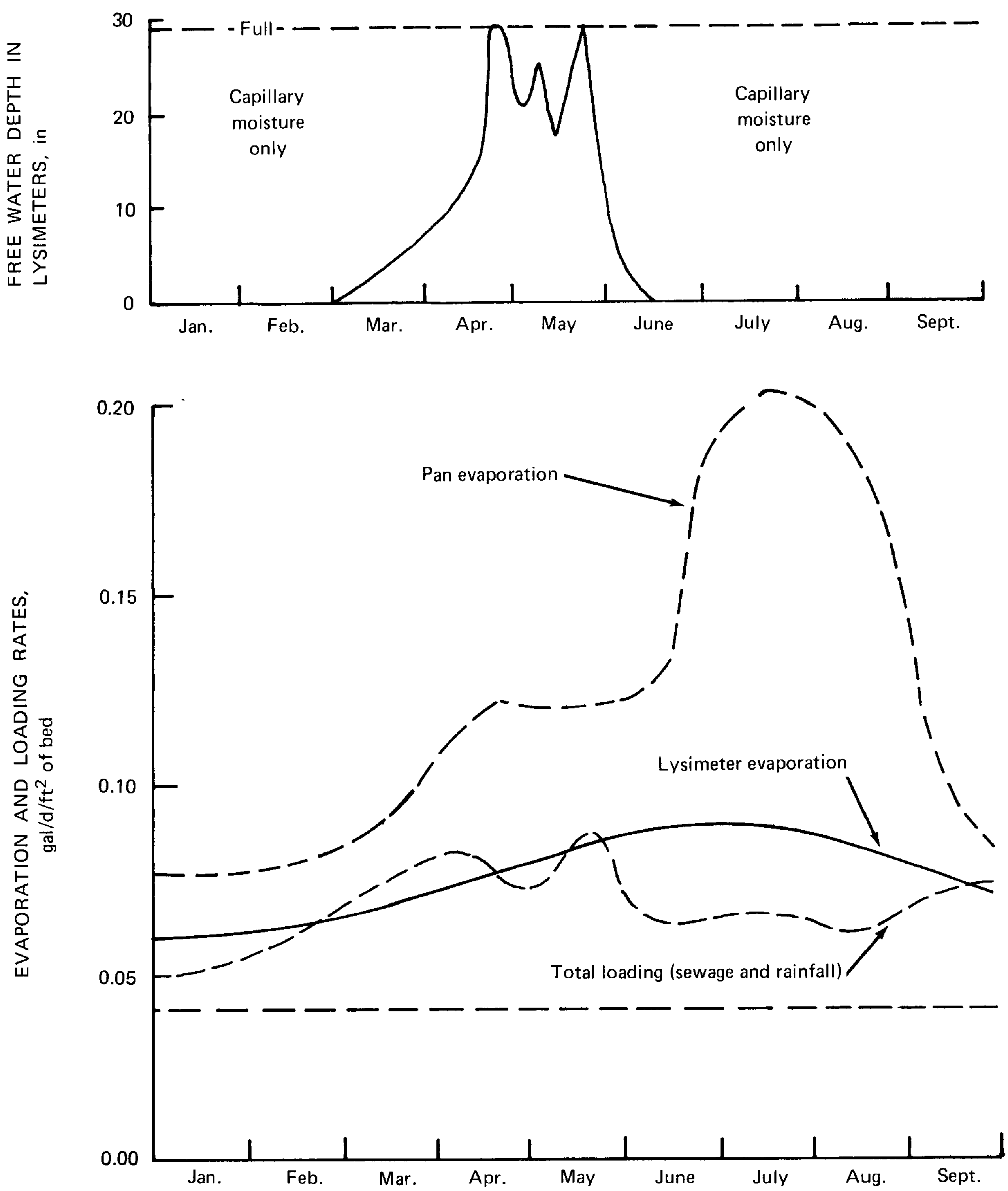


Figure I-25. Evapotranspiration bed water-balance characteristics.

rainfall precludes the use of soil-based ET systems, mechanical devices might be applied. The ultimate value of this approach depends on the cost of commercially manufactured units.

The kind of unit being evaluated in the prototype studies at the University of Colorado is a multiple, concentric disc system rotating on a common shaft. Preliminary studies are under way to determine optimum rotation speed and disc size, submergence, material, and surface characteristics.

ALTERNATIVE SELECTION

The choices available to the individual for wastewater disposal are numerous, yet only a few will prove economically and environmentally acceptable. The selection process should involve evaluation of technical, economic, and administrative feasibility. All three are extremely important to the successful execution of a project.

Table I-18 lists some options available. It is first necessary to evaluate design constraints, such as soil types, site topography, geological characteristics, climate, and water-quality objectives. Once

Table I-18.—*On-site system alternatives*

Process	Objective
A. In-house:	
1. Water conservation:	
a. Flow control	Reduce water
b. Reuse	
2. Waste segregation:	
a. Non-water-carriage toilet:	
(1) Chemical	
(2) Biological	
(3) Recycle	Reduce pathogens, flow, BOD, N, P, solids
(4) Incinerating	
Very low flow toilet	
b. Gray-water treatment	Reduce P, solids, BOD, pathogens
3. Household product selection	Control waste products, P, toxic metals, chlorinated compounds
4. Appliance selection	Reduce flow, waste constituents
B. Wastewater carriage:	
1. Gravity sewer	
2. Small-diameter gravity sewer	Carry wastewater
3. Pressure sewer	
4. Vacuum sewer	
C. Anaerobic:	
1. Septic tanks	Reduce solids, grease, pathogens?
2. Fixed media:	
a. Sand/granular	Reduce solids, BOD, pathogens, N
b. Synthetic media	Reduce solids, BOD, pathogens?
D. Aerobic:	
1. Suspended growth:	
a. Activated sludge:	
(1) Batch	Reduce solids, BOD, nitrification, pathogens
(2) Continuous	
b. Lagoons	Reduce solids, BOD, pathogens
2. Fixed media:	
a. Sand-intermittent filter	Reduce solids, BOD, pathogens, nitrification
b. Sand expanded	
c. Coarse media:	
(1) Trickling filter	
(2) Rotating disks	Reduce solids, BOD, pathogens, nitrification
(3) Tray/media contractors	
3. Emergent vegetation	Reduce N, P, BOD, pathogens

Table I-18.–*On-site system alternatives*–Continued

Process	Objective
E. Physical-chemical:	
1. Adsorption	Polish organics
2. Ion exchange	NH_3, M^+
3. Chemical precipitation:	
a. Suspended	Reduce P, solids, BOD, pathogens
b. Fixed	
4. Disinfection:	
a. Halogens	
b. Ultraviolet unit	Reduce pathogens
c. Ozone	
5. Holding tank	Storage
F. Land application:	
1. Soil absorption	
2. Mounds	
3. Irrigation	Dispose solids, BOD, pathogens, P, nitrification
4. Lagoons (adsorption)	
5. Evapotranspiration	

these have been delineated, an orderly selection of options can be evaluated through cost analysis. Finally, an appropriate institutional framework must be developed to ensure appropriate construction, operation, and maintenance of the system. Sample flowsheets are depicted in table I-19. Although many of these systems have been proved technically feasible, extensive field testing to determine process reliability and effectiveness of institutional controls is still needed.

Costs of Alternative On-Site Treatment and Disposal Systems

The costs of alternative on-site treatment and disposal systems are not well documented, owing to an insufficient data base. Unit costs, based primarily on Wisconsin studies, are presented in table I-20. These costs are estimates; systems should be evaluated on a site-by-site basis.

WISCONSIN SMALL SCALE WASTE MANAGEMENT PROJECT

In 1971, the State of Wisconsin provided research funds to the University of Wisconsin to commence investigations into the on-site disposal of wastewater. The SSWMP was established with the objectives to determine and understand the causes of failure of septic tank-soil absorption fields; to improve methods of site characterization, system design, and system construction for on-site disposal of wastewater; to search out effective alternatives to the septic tank-soil absorption field in problem soils; to develop more effective management techniques to on-site wastewater disposal systems; and to assess the implications of new wastewater disposal technologies for land use planning.

Since then, substantial funding also has been provided by the Wisconsin Department of Natural Resources, the Upper Great Lakes Regional Commission, and the United States Environmental Protection Agency. A major portion of Part I of this publication is based on research efforts conducted

Table I-19.—*Sample flowsheets*

Environmental design constraints	Process[a]	Cost	Maintenance	Reliability
Good soil:				
Single home	C1-F1	Low	Low	Excellent if properly constructed and maintained
High bedrock:				
Single home	C1-F2	High	Low	Excellent if properly constructed and maintained
	or			
	A1a-C1-D2a^{E}4b	High	High	Data unavailable
Poor soil topography:				
Surface water with BOD 10 mg/l, SS 10 mg/l, FC < 200/100:				
Single home	A1a-D1-D2a^{E}4a	High	High	Data unavailable
	or			
	A1a-C1-D2a^{E}4b	High	High	Data unavailable
Lake with BOD 10, SS 10, M < 1, P < 1, FC < 100/100:				
Single home	A1-C1-D2a-C2a-E3a^{E}4b	High	High	Data unavailable
	or			
	A2b-C1-E2-D2a-E3b^{E}4a	High	High	Data unavailable
Surface water with BOD 10 mg/l, SS 10 mg/l, FC < 200/100:				
Small community	A1a-C1-B2-F1	Moderate[b]	Moderate[b]	Depends on institutional control
	or			
	A2b-(E-5)-C1-B3-D1-D2a^{E}4b	Moderate[b]	Moderate[b]	Depends on institutional control

[a]See also table I-18.
[b]Relative to centralized system.

Table I-20.—*Capital costs and operation and maintenance costs*

Unit	Cost, dollars
Septic tank (1,000 gal):	
Installed cost	350 to 450
Operation cost per year	10
Aerobic:	
Installed cost	1,500 to 2,500
Maintenance per year	65 to 110
Power per year, 2.4 to 7.4 kwh/d (@ 4¢/kwh)	35 to 60
Sand filter:	
Installed cost	15 to 20/ft^2
Maintenance per year	1/ft^2
Pump chamber:	
Installed cost:	
Chamber	200 to 250
½-hp sump pump and controls	300 to 350
Operation cost[a] per year, 0.1 kwh/d @ 4¢ per kwh	21
Chlorine and settling chamber:	
Installed cost	1,000 to 1,200
Operation cost[a] per year (chemical)	26
Ultraviolet irradiation:	
Installed cost	1,100 to 1,500
Power, 1½ kwh/d @ 4¢ per kwh	20
Maintenance per year	40 to 60
Soil absorption system:	
Installed cost	1 to 1.25/ft^2
Maintenance per year	–
Mound:	
Installed cost	2,500 to 4,000
Maintenance per year	18 to 25
Power per year	2
Evapotranspiration:	
Installed cost	75¢ to 1.00/ft^2
Maintenance per year	–

[a]Does not include pump replacement.

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Part II

SEPTAGE TREATMENT AND DISPOSAL

GENERAL

The first priority of the EPA's program to abate water pollution has been to provide adequate wastewater treatment for sewered communities. According to the 1970 census,[1] however, 16.6 million housing units, or more than 24.5 percent of the housing units in the United States, relied on septic systems for wastewater disposal.

Users

The geographical distribution of septic systems, as seen in figure II-1 and table II-1, shows that States with more than 35 percent use are located in New England, the Southeast, and the Pacific Northwest. Most North-Central, Northeastern, and Southeastern States have only a slightly lower use of these on-site disposal facilities. The Southwestern States' use of septic tanks is between 10

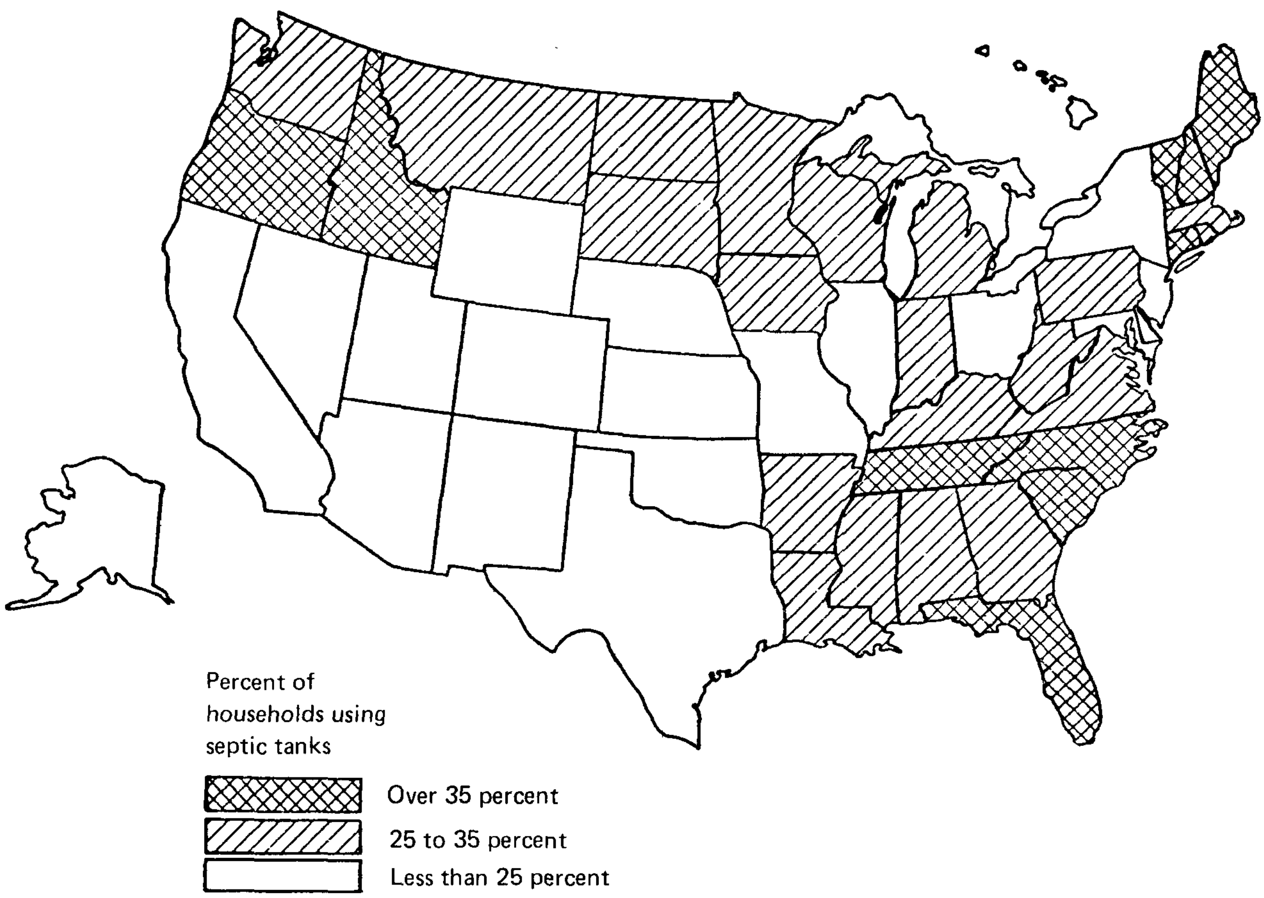

Figure II-1. Distribution of on-site septic systems, by State.[1]

Table II-1.—*Sewage disposal characteristics for the United States*[a,1]

State	Housing units on public sewers		Housing units with septic tanks		Housing units with other	
	Number	Percent of total	Number	Percent of total	Number	Percent of total
Alabama	566,307	50.80	385,345	34.56	163,139	14.63
Alaska	55,511	62.69	18,629	21.03	14,423	16.29
Arizona	446,304	77.11	114,433	19.77	18,013	3.11
Arkansas	355,684	52.85	220,287	32.73	96,999	14.41
California	6,084,632	87.22	853,013	12.23	38,324	0.55
Colorado	612,659	82.47	113,290	15.25	16,689	2.25
Connecticut	608,603	62.82	354,585	36.60	5,633	0.58
Delaware	130,259	74.44	39,860	22.78	4,870	2.78
Washington, D.C.	277,068	99.52	454	0.16	871	0.31
Florida	1,509,682	60.61	938,352	37.67	42,743	1.72
Georgia	848,516	57.85	474,455	32.35	143,654	9.79
Hawaii	161,438	74.78	50,558	23.42	3,844	1.78
Idaho	137,891	57.87	93,146	39.09	7,266	3.05
Illinois	3,072,266	83.20	554,603	15.02	65,080	1.76
Indiana	1,060,942	61.97	589,794	34.45	61,061	3.57
Iowa	662,320	69.35	257,889	27.00	34,829	3.65
Kansas	594,758	75.52	163,918	20.82	28,808	3.66
Kentucky	536,388	50.57	312,856	29.50	211,328	19.93
Louisiana	778,247	67.90	287,481	25.08	80,245	7.01
Maine	169,975	50.11	140,409	41.39	28,817	8.50
Maryland	953,470	77.23	243,728	19.74	37,271	3.02
Massachusetts	1,339,304	72.83	490,365	26.67	9,120	0.50
Michigan	1,947,137	68.43	847,433	29.78	50,509	1.78
Minnesota	864,984	70.92	307,441	25.21	47,070	3.86
Mississippi	338,581	48.56	209,115	30.00	149,514	21.44
Missouri	1,173,688	70.47	359,278	21.57	132,617	7.96
Montana	154,581	64.21	74,198	30.82	11,974	4.97
Nebraska	385,860	75.44	105,320	20.59	20,266	3.96
Nevada	147,743	86.07	21,988	12.81	1,951	1.13
New Hampshire	132,475	53.26	109,015	43.83	7,231	2.91
New Jersey	1,890,977	82.03	404,241	17.53	10,123	0.44
New Mexico	230,737	71.60	65,781	20.42	25,722	7.98
New York	4,824,525	98.34	1,289,253	20.93	44,883	0.73
North Carolina	733,848	45.32	687,572	42.46	197,859	12.22
North Dakota	128,967	64.32	53,074	26.47	18,457	9.21
Ohio	2,565,317	74.41	779,510	22.61	102,566	2.98
Oklahoma	686,240	73.17	203,174	21.66	48,413	5.16
Oregon	448,967	61.04	275,944	37.52	10,559	1.44
Pennsylvania	2,798,522	72.13	985,014	25.39	96,502	2.48
Rhode Island	197,947	64.41	107,544	34.99	1,843	0.60
South Carolina	363,611	45.18	334,210	41.53	106,996	13.29
South Dakota	140,258	63.30	62,366	28.14	18,970	8.56
Tennessee	671,248	51.76	457,008	35.24	168,672	13.00
Texas	2,989,684	78.49	654,283	17.18	164,950	4.33
Utah	258,649	82.93	49,249	15.79	3,976	1.28
Vermont	72,264	48.23	68,265	45.56	9,315	6.21

Table II-1.—*Sewage disposal characteristics for the United States*[a,1]—Continued

State	Housing units on public sewers		Housing units with septic tanks		Housing units with other	
	Number	Percent of total	Number	Percent of total	Number	Percent of total
Virginia	906,030	71.02	408,213	27.49	170,580	11.49
Washington	786,551	65.28	403,909	33.52	14,464	1.20
West Virginia	304,151	51.31	187,028	31.55	101,600	17.14
Wisconsin	994,926	70.26	371,567	26.24	49,549	3.50
Wyoming	86,983	75.94	23,349	20.38	4,217	3.68
Total	48,187,675	71.18	6,601,792	24.52	2,904,375	4.30

[a]Total housing units = 67,693,842.

and 20 percent. On a local level, many counties in New Jersey, New York, California, and other States have over 50,000 housing units that use on-site waste disposal systems, but statewide use appears less significant. Areas with over 100,000 housing units using on-site waste disposal systems include suburban New York, Los Angeles, and Miami.[2]

The use of a septic system requires periodic maintenance that includes pumping out the accumulated scum and sludge, which is called septage. Kolega[3] has reported a septage buildup of between 65 and 70 gallons per capita per year in properly functioning septic systems.

Various recommendations exist for time periods between pumping out a septic tank, most between 2 and 5 years. After a hauler pumps out a homeowner's septage, it must be disposed of in a safe, cost-effective, and convenient manner. Table II-2 shows the estimated statewide septage generation per year, based on pumping the average 1,000-gallon septic tank every 4 years.

Septage Characteristics

Septage is a highly variable anaerobic slurry having large quantities of grit and grease; a highly offensive odor; the ability to foam; poor settling and dewatering abilities; high solids and organic content; and, quite often, an accumulation of heavy metals. Tables II-3, II-4, and II-5 present the results of previous research work compiled by EPA's Cincinnati research group,[4] as well as extreme values reported in the literature.

Graner[6] reports septage in Nassau and Suffolk (N.Y.) counties with characteristics similar to medium to strong wastewater; in Maine Goodenow[9] found samples with total solids (TS) and suspended solids (SS) over 130,000 mg/l and 93,000 mg/l, respectively. In Alaska, Tilsworth[7] obtained septage samples with 5-day biochemical oxygen demand (BOD_5) over 78,000 mg/l and chemical oxygen demand (COD) over 700,000 mg/l. The EPA mean concentrations are good indicators of septage concentrations when compared with other researchers' data.

The geometric mean heavy-metal content of residential septage from Lebanon, Ohio, was compared with geometric means found in raw and digested sludge from 33 U.S. sewage treatment plants and with metal content in Danish and Swedish sludge (see table II-5). On an mg/kg dry-weight basis, domestic septage contains one-half to two orders of magnitude less heavy metal than does municipal sludge.[4]

Table II-2.—*Estimated household septage generation by State*[a,1]

[Millions]

State	m³/yr	gal/yr	State	m³/yr	gal/yr
Alabama	0.36	96.3	Montana	0.07	18.5
Alaska	0.02	4.7	Nebraska	0.10	26.3
Arizona	0.11	28.6	Nevada	0.02	5.5
Arkansas	0.21	55.1	New Hampshire	0.10	27.3
California	0.81	213.3	New Jersey	0.38	101.1
Colorado	0.11	28.3	New Mexico	0.06	16.4
Connecticut	0.34	88.6	New York	1.22	322.3
Delaware	0.00	1.0	North Carolina	0.65	171.9
Washington, D.C.	0.00	0.11	North Dakota	0.05	13.3
Florida	0.89	234.6	Ohio	0.74	194.9
Georgia	0.45	118.6	Oklahoma	0.19	50.8
Hawaii	0.05	12.6	Oregon	0.26	69.0
Idaho	0.09	23.3	Pennsylvania	0.93	246.3
Illinois	0.52	138.7	Rhode Island	0.10	26.9
Indiana	0.56	147.4	South Carolina	0.32	83.6
Iowa	0.24	64.5	South Dakota	0.06	15.6
Kansas	0.16	41.0	Tennessee	0.43	114.3
Kentucky	0.30	78.2	Texas	0.62	163.6
Louisiana	0.27	71.9	Utah	0.05	12.3
Maine	0.13	35.1	Vermont	0.06	17.1
Maryland	0.23	60.9	Virginia	0.39	102.1
Massachusetts	0.46	122.6	Washington	0.38	101.0
Michigan	0.80	211.9	West Virginia	0.18	46.8
Minnesota	0.29	76.9	Wisconsin	0.35	92.9
Mississippi	0.20	52.3	Wyoming	0.02	5.8
Missouri	0.34	89.8			
			Total	15.67	4,141.91

[a]Based on pumping a 1,000-gallon septic tank every 4 years.

Bacteriology

Septage contains predominately gram-negative, nonlactose fermenters. Many of these microorganisms, such as *Pseudomonas*, are considered aerobic and have been found in septic tanks. Numerous obligate anaerobes are present; but only spore-forming types, including *Clostridium lituseburence* and *Clostridium perfringens*, have been recovered. Calabro[12] was unsuccessful at isolating non-spore-forming obligate anaerobes, such as bacteriodes; because they are exceedingly oxygen-sensitive, the pumping operation may expose anaerobes to incident oxygen, thereby killing them. Figure II-2 compares specific types of microorganisms from 12 septage and septic tank sewage samples, with 95-percent confidence limits. The standard plate count per millilitre was determined after 48 hours incubation under aerobic and anaerobic conditions at 24° C ± 1°. When the septic tank is pumped, sludge, intermediate wastewater, and upper layer of scum are mixed, yielding aerobes and anaerobes.

The presence of aerobes in a septic tank can be explained by either the dissolved oxygen of the incoming sewage providing sufficient oxygen to allow limited aerobic growth, or by chemostatic displacement of effluent by the influent furnishing a relatively constant number of aerobic microorganisms. It is fortunate that *Pseudomonas* and similar aerobic bacteria are found in the septic tank, as they add limited lipid and detergent degradation capabilities.

Table II-3.—*Septage characteristics*[a,4]

Parameter	EPA mean concentration	Minimum reported	Maximum reported	Variability[b]
Total solids	40,000.0	[6]1,132.0	[9]130,475.0	115
Total VS	26,000.0	[5]4,500.0	[9]71,402.0	16
Total SS	15,000.0	[7]310.0	[9]93,378.0	301
VSS	[5]18,100.0	[5]3,660.0	[10]51,500.0	14
BOD_5	5,000.0	[6]440.0	[7]78,600.0	179
COD	45,000.0	[7]500.0	[7]703,000.0	469
TOC	15,000.0	[4]316.0	[8]96,000.0	73
TKN	600.0	[4]66.0	[5]1,900.0	29
NH_3-(N)	150.0	[4]6.0	[8]380.0	63
NO_2	[5]0.7	[8]<0.1	[8]1.3	13
NO_3	[5]3.2	[8]<0.1	[11]11.0	110
Total P	150.0	[5]20.0	[4]760.0	38
PO_4	[5]64.0	[5]10.0	[5]170.0	17
Alkalinity	[5]1,020.0	[7]522.0	[7]4,190.0	8
Grease	9,561.0	[4]604.0	[4]23,368.0	39
pH (units)	6 to 9.0	[6]1.5	[6]12.6	8
LAS	150.0	[4]110.0	[4]200.0	2

[a]All values in mg/l, except where noted.
[b]Values represent ratio of maximum to minimum values.

Table II-4.—*Septage metal concentrations*[a,4]

Metal	EPA mean concentration	Minimum reported	Maximum reported	Variability[b]
Al	50.0	[4]2.0	[4]200.0	100
As	0.1	[4]0.03	[4]0.5	17
Cd	0.5	[4]0.05	[4]10.8	216
Cr	1.0	[4]0.30	[8]3.0	10
Cu	8.5	[8]0.30	[4]34.0	113
Fe	200.0	[4]3.0	[4]750.0	250
Hg	0.1	[4]0.0002	[4]4.0	20,000
Mn	5.0	[4]0.50	[4]32.0	64
Ni	1.0	[4]0.20	[8]28.0	140
Pb	2.0	[4]1.50	[4]31.0	21
Se	0.1	[4]0.02	[4]0.3	15
Zn	50.0	[8]33.00	[4]153.0	5

[a]All values in mg/l.
[b]Values represent maximum to minimum values.

Calabro[12] estimated the gross relative stability of septage, septic tank sewage, and domestic wastewater using methylene blue as a redox indicator of biological activity. Septage samples changed color in 5 hours, septic tank sewage in 6 to 21 hours, and raw domestic sewage in 17 to 21 hours.

Table II-5.—*Heavy metal content of septage and municipal sludge*[a,4]

Metal	Lebanon, Ohio	Salotto	Other U.S.	Denmark	Sweden
Cd	5.5	43.0	69.0	10.0	9.3
Cr	21.0	1,050.0	840.0	110.0	170.0
Cu	28.1	1,270.0	960.0	340.0	670.0
Hg	0.24	6.5	28.0	7.8	5.8
Mn	106.0	475.0	400.0	350.0	400.0
Ni	28.5	530.0	240.0	37.0	65.0
Zn	1,280.0	2,900.0	2,600.0	2,600.0	1,900.0

[a]All values in mg/kg dry solids.

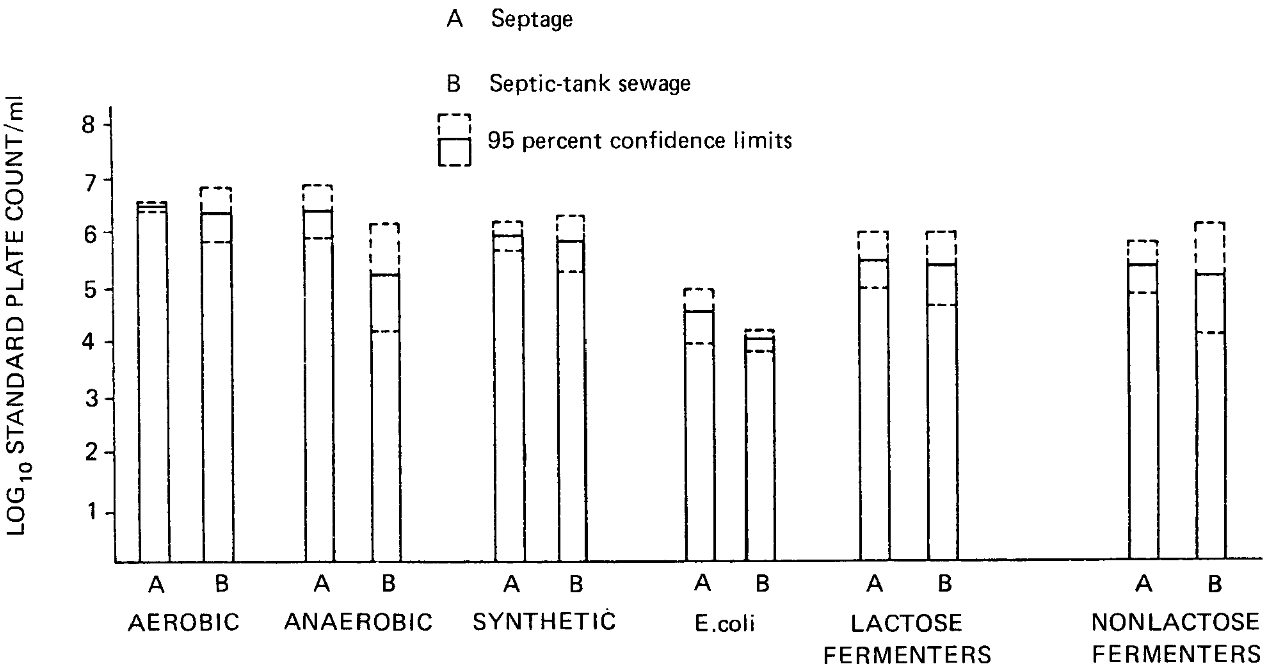

Figure II-2. Comparison of specific types of micro-organisms, with 95 percent confidence limits (after Calabro).[12]

LAND DISPOSAL

Septage disposal on land can include surface spreading and sub-sod injection (SSI), spray irrigation, trench and fill, sanitary landfills, and lagooning. All land disposal alternatives require analyses of soil characteristics, seasonal groundwater levels, neighboring land use, groundwater and surface water protection and monitoring, climate, and site protectors such as signs and fences.

Land spreading requires a knowledge of land slopes, which are often limited to 8 percent, and of runoff conditions. Other requirements may include storage facilities for times when land application is inadvisable, crop management techniques, odor control procedures, and loading criteria. Loading criteria generally are determined by agricultural considerations that result in the limiting of organic and heavy metals.

Loading Factors

Nitrogen. In most agricultural areas, available existing nitrogen (N) is far below levels needed for optimum crop yield. As a result, artificial sources of nitrogen, such as commercial fertilizer, are generally added. Nitrogen is available as a plant nutrient in the form of the ammonium ion, which is retained on negatively charged soil particles.[13] Septage is rich in available ammonia, with about 25 percent of the total 5 to 8 lb/1,000 gal of nitrogen occurring in this form. Soil bacteria will transform NH_3-N to NO_3-N, but much of this nitrogen may not be available for plant use if hydraulic loadings cause the highly soluble NO_3-N to be leached below plant roots. Nitrogen also may be lost if poor drainage conditions exist. This causes rapid denitrification, converting nitrate to nitrogen gas.

Nitrogen must be applied at rates less than or equal to plant nitrogen uptake requirements. Otherwise, excess nitrates could form and contaminate groundwater or surface water through leaching or runoff. Nitrate concentrations above 10 mg/l in drinking water may cause health problems, particularly infant methemoglobinemia (nitrate cyanosis). Nitrate pollution in surface waters also will lead to accelerated eutrophication of lakes and streams.

Maine has reported in "Maine Guidelines for Septic Tank Sludge Disposal on the Land"[14] that a loading criteria of 62,500 gallons per acre per year on well-drained soils and 37,500 gallons per acre per year on moderately well-drained soils will not result in pollution caused by excess nitrogen. These loadings result in an application of 500 pounds per acre per year in well-drained soils and 300 pounds per acre per year in moderately well-drained soils. Maine officials report that wells monitored at sites that follow these criteria show no signs of pollution.

Phosphorus and Potassium. Both phosphorous and potassium are basic requirements for plant growth. Land application of septage usually results in a phosphorous surplus and a potassium deficiency. Both elements, however, tend to become fixed in the soil and are not likely to leach out. For this reason, nitrogen requirements usually govern the organic considerations in application rates.

Heavy Metals. The phytotoxic metals—zinc (Zn), nickel (Ni), and copper (Cu) and cadmium (Cd) are foliage-limiting factors in the amount of sludge that may be applied to the land. How these metals are retained in the soil is a complex and poorly understood process, but workable estimates of limits based on soil cation exchange capacity (CEC) have been proposed by researchers in Wisconsin.[13]

The CEC can be estimated by a displacement procedure that yields an exchange capacity in milliequivalents (meq) per 100 grams of soil. A lifetime application has been proposed[13] that would limit the amounts of phytotoxic metal applied in terms of Zn equivalent. Further research on lifetime metal-loading limits is underway, as some heavy metals seem to become tied up in the soil over time. This is the result of a reversion effect linked with a solid state diffusion of metal into crystalline soil structures. Attenuation of the effects of phytotoxic metals in sludges when they are overapplied to the land may be attributed to this mechanism.

The Wisconsin metal-loading criterion limits the Zn equivalents to 10 percent of the CEC, based on Cu being twice as toxic as Zn and Ni four times as toxic as Zn. Other researchers have proposed relative toxication ratios other than 1:2:4.

The calculation of permitted lifetime loading of metal from septage is expressed as:[13]

$$ML = 65(CEC)/\frac{(Zn) + 2(Cu) + 4(Ni)}{500}$$

where:

ML = maximum loading to soil, tons of sludge per acre
CEC = cation exchange capacity of soil, meq/100 g
Zn = zinc content of sludge, mg/l
Cu = copper content of sludge, mg/l
Ni = nickel content of sludge, mg/l

Cadmium presents a special problem because of its mobility and its potential for accumulating in the edible portions of plants. Effects are cumulative and insidious. For example, excessive alimentary cadmium intake manifests itself in humans as Itai-Itai (Ouch-Ouch) disease.[15] This hazard is endemic to the Jintsu River region in Japan. The affected area is situated below the Kamioka mine. High concentrations of cadmium, lead, and zinc have been traced to the mine's effluent, which drains into the Jintsu River. The area has shown high concentrations of cadmium in rice, fish, and river water. Drainage from the mine varied from 0.005 mg/l to 0.6 mg/l Cd at a pH of 7 to 8. Farther downstream, the river water contained almost no cadmium, yet suspended material had concentrated the cadmium, showing a concentration of 363 pm to 382 pm. Solids had carried over into the rice paddies where rice roots concentrated the cadmium.[16] Rice roots were analyzed and found to contain up to 1,300 pm Cd.[17] People complaining of renal dysfunctions were diagnosed as having Itai-Itai disease. Other symptoms include advanced skeletal deformations and weakened bones that fracture easily. Friberg[16] hypothesized that bone structure weakness was caused by cadmium having replaced calcium in bone material.

One recommendation for Cd limits is based on Wisconsin findings that 2 pounds or more per acre showed a significant increase in metal concentration in plants over control plants. The proposed limits are 2 pounds per acre per year and a lifetime loading of 20 pounds per acre.[13]

The proposed limits of phytotoxic metals and cadmium are reported to be low enough to protect reasonably well-chosen disposal sites. Based on Lebanon, Ohio, septage and Salotto findings, as shown in table II-5, approximately eight times as much septage as municipal sludge could be applied to the land, using cadmium as the limiting factor. Using the phytotoxic metals limit, approximately five times as much septage could be applied.[4]

Metal-Loading Calculation.[13] A sample metal-loading calculation for septage application rates based on a combination of phytotoxic metals and cadmium, again assuming average Lebanon, Ohio, septage and a soil CEC of 10 meq/100g, follows.

Septage Concentration:

Zn = 50 mg/l
Cu = 8.5 mg/l
Ni = 1.0 mg/l
Cd = 0.5 mg/l

Total metal equivalent loading:

$$65 \times \text{CEC} = 650 \text{ lb per acre}$$

Septage metal equivalent per ton:

$$\frac{50 + 2(8.5) + 4(1.0)}{500} = \frac{71.0}{500} = 0.142 \text{ lb metal equivalent per ton septage}$$

Lifetime loading permitted:

$$\frac{650}{0.142} = 4{,}577.5 \text{ tons per acre}$$

Yearly loading limit due to Cd:

$$\frac{2 \times 500}{\text{pm Cd}} = \frac{1{,}000}{0.5} = 2{,}000.0 \text{ tons septage per acre for 2 lb Cd}$$

Lifetime Cd loading permitted:

$$20 \text{ lb per acre} \times \frac{2{,}000.0}{2 \text{ lb}} = 20{,}000.0 \text{ tons per acre}$$

Cadmium loading, therefore, is limiting on a yearly basis (2,000.0 tons per acre per year), and phytotoxic metal equivalents are limiting on the lifetime of the site (4577.5 tons per acre per year).

The yearly loading based on cadmium of 2,000 tons per acre per year translates to 0.469 million gallons per acre per year, or 4.8 times the application rate based on the limiting nitrogen loading in this example of 500 pounds per acre per year. A well-drained site receiving this septage would have a phytotoxic metal-loading lifetime of 11 years at the nitrogen application rate.

Pathogens. The natural digestion process in a septic tank does not always result in a pathogen-free material, as related by Calabro,[12] who found *salmonella* and other potentially dangerous organisms in septage. For this reason, care must always be taken in handling it.

Evidence that pathogens are reduced when septage is exposed to atmospheric conditions is based on work by the Metropolitan Sanitary District of Greater Chicago and others. Table II-6 shows that after 7 days only 1 percent of the original coliforms survived. Table II-7 shows basically the same reduction for sludge cake applied to the land. Table II-8 shows the number of days of storage required in a laboratory study[14] to reduce several viruses and bacteria to 99.9 percent of the original values at various temperatures.

The soil reportedly has removed pathogens by various mechanisms, predominately filtration, soil inactivation, and die-off. Pathogen travel is usually restricted to the order of feet from point of application unless runoff or channeling occur, potentially polluting surface and ground water.

Table II-6.—*Fecal coliform counts of stored digester supernatant exposed to atmospheric conditions*[18]

Days	Fecal coliform counts (per 100 ml)	Percent survival
0	[a]800,000	100.00
2	[b]20,000	2.50
7	8,000	1.00
14	6,000	0.75
21	<2,000	<0.25
35	<20	<0.01

[a]Fecal coliform count just before lagooning.
[b]Fecal coliform count after lagooning.

Table II-7.—*Disappearance of fecal coliforms in sludge cake covering a soil surface*[19]

Days after sludge application	Fecal coliforms/gm sludge cake, dry weight
1	3,680,000
2	655,000
3	590,000
5	45,000
7	30,000
12	700

Table II-8.—*Laboratory study on number of days' storage required for 99.9 percent reduction of virus and bacteria in sludge*[20]

Organism	Number of days		
	4° C	20° C	28° C
Poliovirus 1	110	23	17
Echo virus 7	130	41	28
Echo virus 12	60	32	20
Coxsackie virus A9	12	—	6
Aerobacter aerogenes	56	21	10
Escherichia coli	48	20	12
Streptococcus faecalis	48	26	14

The Wisconsin guidelines[13] for sludge disposal on agricultural land do not recommend raw sludge spread without treatment. Partially digested septage may be applied if some preventive measures are followed, such as lagooning before land disposal or immediate liming of septage.

Disposal Methods

Surface Application. This method of septage disposal is perhaps the most frequently used in the United States today. Future studies should give consideration to stabilization and additional pathogen reduction before surface application of septage to land because no discussion of health hazards in this respect is available. With surface application techniques, some nitrogen loss occurs through ammonia volatilization; the highest losses occur with spray irrigation.

Land Spreading. The hauler truck that pumps out the septic tank is frequently the vehicle that applies septage to the land. Consideration should be given to intermediate holding facilities before land application. Storage is necessary during or just before precipitation to prevent runoff of contaminated water. In colder climates, land application should be limited to unfrozen surfaces to prevent runoff during thaws. Pathogen die-off during storage, as mentioned before, also indicates the necessity of storage.

With a storage facility, disposal can be performed by the hauler truck or by a tank wagon, usually pulled by a farm tractor. The choice is one of economics. A larger operation may choose to have its trucks on the road with septage spreading performed by a separate crew, thus freeing the more expensive tank truck to perform cleanout functions. A smaller septage hauler may prefer to use one vehicle to perform both tasks, thus leveling the workload by spreading septage during slack hauling periods. In some instances, soil conditions may require the use of flotation-type tires that are not suitable for long-distance highway use. This would dictate the use of separate collection and spreading vehicles.

Ridge-and-Furrow. This method has been used to dispose of sludges on relatively level land, usually that limited to 1.5-percent slopes. No instances of this method were found during this study. Although this method can be used to distribute septage to row crops during their growth, care should be taken to ensure these crops are not for human consumption.

Spray Irrigation. Spray irrigation of septage necessitates storing it in a lagoon before disposal. Portable pipes and nozzle guns are used rather than fixed or solid ones. Because the septage must be pumped at 80 to 100 psi through 3/4-inch to 2-inch nozzle openings, a screening device at the lagoon's pump suction is mandatory. Spray irrigation also offers the greatest potential for offensive odors; thus a knowledge of wind patterns and a well-located site are important during design stages.

Subsurface Application. Soil incorporation techniques offer better odor and pest control than surface spreading techniques and reduce the likelihood of inadvertent pathogen contamination to humans. Disadvantages include full incorporation of all nitrogen because ammonia volatilization is eliminated; this action reduces any nitrogen-loading safety factor from ammonia loss in surface spreading. Costs are greater than for surface spreading because a storage lagoon or tank and subsurface injection equipment are necessary. A resting period of 1 to 2 weeks is required before equipment can be driven over the waste-incorporated land.[13] Three methods have been used to inject septage into the land.

Plow furrow cover (PFC). A typical setup consists of a moldboard, a furrow wheel, and a colter. Septage is placed in a narrow furrow and immediately plowed over.

Sub-sod injection. This technique uses a device that injects a wide band or several narrow bands of septage into a cavity 6 to 8 inches below the surface. Some equipment forces the injection swath closed.

Terreator. This is a patented device that drills a 9.5-cm hole with an oscillating chisel point. A tube places the septage as deep as 50 cm below the surface at a rate of 24.8 litres per linear metre (2 gallons per linear foot).[21] Kolega[21] found that subsurface application of 300 pounds of nitrogen per year in a well-drained soil did not produce any noticeable ground water quality variation with PFC, SSI, or terreator methods.

Burial. Methods include disposal in trenches, sanitary landfills, leaching lagoons, or settling lagoons with infiltration-percolation beds. Foul odors are endemic to these operations until a final soil cover is placed over the open surfaces of trenches or landfills. Lagoon management practices, such as proper inlet design, site location, and liming, minimize these problems.

Site selection is important, not only to control odor but also to minimize potential ground water and surface water pollution problems. Many States require wells' sampling and ground water monitoring as operational checks.

Trenches. Disposing septage in trenches is similar to disposing it in lagoons, except that trenches are usually a smaller scale alternative. Septage is placed sequentially in one of many trenches in small lifts, 6 to 8 inches, to minimize drying time.[22] When a trench is filled with septage, 2 feet of soil should be placed as a final covering, and a new trench opened. New York recommends trenches be a maximum of 7 feet deep. Sufficient room must be left between trenches for movement of heavy equipment. The trench-and-fill technique is often used at sanitary landfills.

Sanitary Landfill. When a sanitary landfill accepts septage, leachate production and treatment must be investigated. For moisture absorption, New Jersey recommends a starting value of 10 gallons of septage to each cubic yard of solid wastes. Septage should be prevented from entering landfills in areas with more than 35 in/yr rainfall if leaching prevention and control facilities or an isolated hydrogeological rock stratum are not present.

A 6-inch earth cover should be applied daily to each area that was dosed with septage. A 2-foot final cover should be placed within a week after the placement of the final lift.[23] Many designers suggest a maximum cell height of 8 feet.[24] Using the New Jersey criterion and an 8-foot cell height, 1,000 gallons of septage could be distributed on 340 ft^2.

Leaching Lagoons. Connecticut[23] has been advocating leaching lagoon systems of earthen anaerobic-aerobic sludge digestion cells. Septage is discharged into a manhole at the edge of a lagoon and exits about one-third the distance into the cell near the bottom. The lagoon bottom is not sealed, and at least one-third of the lagoon is above ground level to facilitate liquid removal by hydrologic gradient and envirotranspiration. The minimum depth of the lagoon is 3 to 5 feet. Sludge is periodically removed, and effluent from this anaerobic lagoon flows through a controlled outlet to an aerobic leaching lagoon. Adding lime is suggested to maintain pH at 6.8 to 7.2. When introduced with the septage into the manhole, lime settles at the end of the anaerobic leaching lagoon influent pipe and exerts little or no effect on lagoon pH.[23] Parallel sets of these series lagoons are recommended. The capacity of each cell is equal to 0.1 of yearly volume, based on 50 to 70 gallons per capita per year of contributing population.

Massachusetts[23] requires a minimum of a 6-foot-deep anaerobic lagoon and six percolation beds, each having 1 ft^2/gal/d of design capacity. The lagoon design requirements call for a sizing of 1 gallon per capita per day, with a minimum of 20 days' retention at average flow. The recommended discharge below the liquid level can stir up sediment and release foul odors. Acton, Mass., now allows haulers to discharge over riprap into the lagoon, which, they report, lessens odor problems.

Disposal Lagoons. These are usually a maximum of 6 feet deep, allow no effluent or underdrain system, and require small (6- to 12-inch) application rates and sequential loading of lagoons for optimum drying. Series or parallel series lagoons with 2 years' capacity each and a 2-foot maximum depth are called for in New York State guidelines.[11] After drying, solids can be bucketed out and disposed in a sanitary landfill and the lagoon used for further applications, or 2 feet of soil may be placed over the solids as a final cover. Many States report odors having been controlled by placing the lagoon inlet pipe below the liquid level and having water available for haulers to immediately wash spills into the lagoon inlet pipe.

Comparison of Land Disposal Practices

The land disposal methods discussed are compared in table II-9. The comparison assumes a moderately well-drained soil, nitrogen-loading requirements, and northern climatic conditions (requiring use of holding tanks or lagoons during inclement weather). In surface-spreading techniques, approximately one-quarter to one-half of the ammonia nitrogen may be lost, raising the amount of nitrogen that can be added.

Cost comparisons are not included because only very limited information is available and costs of existing systems vary widely depending on whether land must be bought and amortized over the life of the project, be rented, or already is municipally owned; the amount of regrading, clearing, and grubbing, if necessary; and access requirements.

Table II-9.—*Land disposal characteristics*

Land disposal method	Acres required @ 10,000 gal/d, 250 d/yr	Characteristics	Advantages	Disadvantages
Surface: Application				
Spray irrigation............	370, plus storage and buffer zone	Large orifices for nozzle; irrigation lines to be drained after irrigation season	Can be used on steep or rough land	High power requirements; odor problems; possible pathogen dispersal Storage lagoon needed for pathogen destruction and when ground is wet or frozen
Ridge and furrow	400, plus storage	Land preparation	Lower power requirements than spray irrigation; can be used in furrows, on crops not grown for human consumption	Limited to 1.5 percent slopes; storage lagoon; some odor
Hauler truck spreading..	400, plus storage	Larger volume trucks require flotable tires; 500- to 2000-gallon trucks ok; 800- to 3000-gallon capacity	Same truck can be used for transport and disposal	Some odor immediately after spreading; storage lagoon; limited to 8 percent slopes
Farm tractor with tank wagon spreading	400, plus storage	Requires additional equipment	Frees hauler truck during high usage periods	Some odor immediately after dispersal; storage lagoon; limited to 8 percent slopes
Subsurface application:				
Tank truck with plow-furrow-cover (PFC)..	420	Single plow mounted on truck; not usable on wet or frozen ground	Minimal odor; storage lagoon optional for pathogen control	Limited to 8 percent slopes; longer time needed for disposal operation than for surface disposal

Table II-9.—*Land disposal characteristics*—Continued

Land disposal method	Acres required @ 10,000 gal/d, 250 d/yr	Characteristics	Advantages	Disadvantages
Farm tractor with PFC..	420	Septage discharge into furrow behind single plow; septage spread in narrow swath and immediately plowed; not usable on wet or frozen ground	Minimal odor; storage lagoon optional for pathogen control	Limited to 8 percent slopes; more time needed for application than in surface disposal
Sub-sod injection (SSI)..	420	Septage placed in opening created by tillage tool; not usable in wet, frozen, or hard ground	Injector can be mounted on rear of some trucks; minimal odor; storage lagoon optional for pathogen control	Limit land to 8 percent; more time needed for application than in surface disposal; keeps vehicles off area for 1 to 2 weeks after injection
Burial:				
Trench	15	New trenches opened when old ones filled; long-term land commitment after operations end	Simplest operation; no slope limits; no climatological limits	Odor problems; high groundwater restrictions; vector problem
Lagoon	30	Sludge bucketed out to landfill from bottom of lagoon; settled water usually flows to percolation/infiltration beds	No slope limits; no climatological limits	Odor problems; high groundwater restrictions; vector problem
Sanitary landfill	195, working surface	Septage mixed with garbage at controlled rates; possible leachate and collection requirements	No topographic limits, simple operation	Odor problems; rodent and vector problems; limited to areas with less than 35 inches yearly rainfall or have leachate collection or be isolated from groundwater

SEPARATE TREATMENT FACILITIES–SEPTAGE ONLY

Alternatives for treating septage at a separate treatment facility include aerated lagoons, anaerobic-aerobic processing, composting, the BIF Purifax Process, and chemical treatment.

Aerated Lagoon

Aerated lagoons can be used to treat septage if the aerators have the required oxygen transfer capacity and create sufficient turbulence to prevent solids deposition. Howley[17] reported severe foaming problems, but he did obtain a volatile suspended solids (VSS) reduction of 23.8 percent and a COD reduction of 73.9 percent, using hydraulic retention times of 1 to 30 days in bench scale units. Howley found 1.8 pounds of oxygen was required to destroy 1 pound of VSS at loadings between 0.03 and 1.3 pounds VSS/f^3/day.[17] He reported that 18,500 gallons per day per million gallons of aerated lagoon design capacity, operating at 50 percent design sewage flow, should not cause overloading.

Brookhaven, Long Island, N.Y., using lagoon treatment of septage, experienced reductions of 62.5 percent in BOD, 51 percent in TS, and 49 percent in SS from influent strengths averaging 5,600 mg/l, 3,700 mg/l, and 2,700 mg/l, respectively. Without equalization facilities, this process was prone to biological upsets. Grit and scum chambers and three large settling lagoons now buffer flow to the 50,000 gal/d septage system. The effluent from a final settling lagoon is chlorinated and discharged to sand recharge beds. Accumulated sludge is removed to a nearby landfill.

Anaerobic-Aerobic Process

The anaerobic-aerobic process uses an anaerobic lagoon or digester, then an aerated lagoon. A pilot anaerobic-aerobic treatment process, with sand beds for filtering final effluent, reported 99 percent BOD, COD, and SS removal, and 90 percent removal of total nitrogen and total phosphorous.[25] Anaerobic digesters, which are useful in reducing high concentrations of volatile solids (VS) and BOD, are addressed later in this section.

Composting

Composting is an alternate septage disposal technique offering good bactericidal action[26-28] and a 25-percent reduction in organic carbon.

In aerobic composting, septage is mixed with dry organic matter for moisture control and for easier air penetration so that aerobic conditions can be maintained. Aerobic composting is generally recognized as superior to anaerobic composting because it provides better odor control, higher temperatures for pathogen control, and requires shorter periods for stabilization.

Process Stages. There are three stages in composting. In the initial stage temperatures go from cryophilic (5° C to 10° C) to mesophilic (10° C to 40° C) regions. Active composting can begin within days and operates in the thermophilic (40° C to 80° C) region, which tends to be self-limiting because of competing mechanisms. When there is an abundance of substrates, bacterial populations increase, thereby raising temperatures. Temperatures above 60° C inhibit microbial growth, lowering population but also lowering temperatures to the point where they are optimum for renewed growth. The third stage is substrate limiting. This curing stage operates under two successive temperature regions—mesophilic (40° C to 10° C) and cryophilic (10° C to 5° C).

Design. Composting sites should have ample room for movement of heavy equipment and should have a receiving tank to equalize septage and collect leachate and surface water. Primary screening for removal of larger unwanted material is advised. After it is mixed with dry organic matter, compost is shaped into windrows, cubes, or hemispheres. Moisture level is controlled by either controlling dry, organic material/septage ratios or by aeration. Pile aeration can be achieved by natural draft, mechanical mixing, forced (bottom) aeration, or turning over the compost.

Lebo System. The Lebo System is in operation in South Tacoma and Bremerton, Wash., and is being constructed for Lewis County and Kent, Wash. A patented preaeration process is used before septage is sprayed on piles of sawdust, wood shavings, or other dry organic material. A 1- to 2-inch application is covered with additional sawdust, and front-end loaders form the mixture into piles to minimize heat loss. Natural draft aeration, possible because the mixture is bulky, eliminates the need for turning or forced aeration. The 50- to 60-percent moisture content material is said to attain a pile temperature of 65° C.[29] The pile is cured in 3 months.

Beltsville System. The Beltsville System, devised by the United States Department of Agriculture, is operating on dewatered sludge in Washington, D.C.; Bangor, Maine; Durham, N.H.; Orange County, Calif.; and Johnson City, Tenn. Camden, N.J., will use the Beltsville forced aeration system on 8-percent sewage sludge, with licorice root as the bulking. The Beltsville System usually mixes sludge with wood chips in long windrows and has piping facilities to alternately blow and suck air through 0.66-cm (0.25-in) holes in 15-cm (6-in) pipe, covered by 30 cm (1 foot) of wood chips or screened compost, to maintain aerobic bacterial action.[26,27] Some turning of the windrows is suggested. After several weeks, the compost can be screened and the wood chips recycled for further composting.

End Use. Some composting facilities attempt to market their end product. This method has rarely been successful because of a lack of public acceptance and other factors. Using end products from municipal facilities as soil conditioners in parks and on golf courses has been acceptable.

In a study conducted by Western Washington Research and Extension Center,[29] Lebo compost applied to sweet corn initially produced no significant change in yield; however, there was an increased yield in subsequent years when compost and fertilizer were added to commercial fertilizer.

Purifax

The BIF Purifax Process oxidizes screened, degritted, and equalized septage with dosages of chlorine, from 700 mg/l to 3,000 mg/l, under moderate pressure. Chlorine replaces oxygen in organic molecules, rendering this material unavailable to bacteria as a food source, thereby stabilizing and deodorizing the septage. The Purifaxed septage changes color from black or deep brown to straw. The process initially releases CO_2, which separates liquids and solids quickly by causing the solids to float.

Purifax treatment results in a highly acidic slurry, pH 1.7 to 3.8. If mechanical dewatering or lagoon separation of the liquids or solids is contemplated, chemicals should be added for pH control of the resultant liquid fraction.

Locations using the Purifax Process in treating septage and sludge in lagoons for liquid-solids separation have had periodic solids separation and odor problems. Sand drying beds appear to be the most efficient method of liquid-solids separation of Purifaxed septage. Adequate ventilation of covered sand drying beds is mandatory to prevent operators from inhaling any NCl_3 released.[22]

Chemical Treatment

Raw septage is chemically treated with lime and ferric chloride at an Islip, Long Island, N.Y., facility. After the septage is screened, degritted, and equalized, about 190 pounds of lime per ton dry solids and 50 gallons per ton dry solids of a standard strength ferric chloride solution are flash-mixed with the septage. The solids-liquid separation step occurs in a clariflocculator. An observed significant-solids carryover problem indicates the separation unit may have been undersize. The liquid fraction is chlorinated and discharged to ground water recharge beds, and the underflow solids from the clariflocculator are vacuum filtered. Long-term relative stability of the lime-ferric, chloride-septage mixture is unknown.

Tilsworth[7] found good liquid-solids separation only after adding huge amounts of chemicals. Separation occurred with 10,000 mg/l lime, 10,000 mg/l ferric sulfate, 4,000 mg/l lime and ferric sulfate mixture, or a 3-percent concentration of a cationic polymer.

Feige et al.[8] added similar quantities of lime (±180 pounds per ton dry solids) to obtain acceptable septage dewatering on sand drying beds. The long-term fate of limed, dewatered septage in landfills and on land needs to be addressed.

SEPTAGE-SEWAGE TREATMENT FACILITIES

Because of their number and location, sewage treatment plants are one of the most frequent acceptors of septage and must be included in any comprehensive study of alternate treatment schemes. Septage can be disposed of in a treatment facility by adding it to the liquid stream or the sludge stream. In either case, a properly designed septage handling facility, including screening, degritting, and equalization, is recommended.

Septage frequently is considered a high-strength wastewater and is dumped into an upstream sewer or placed directly into various unit processes in a treatment plant (fig. II-3). At several facili-

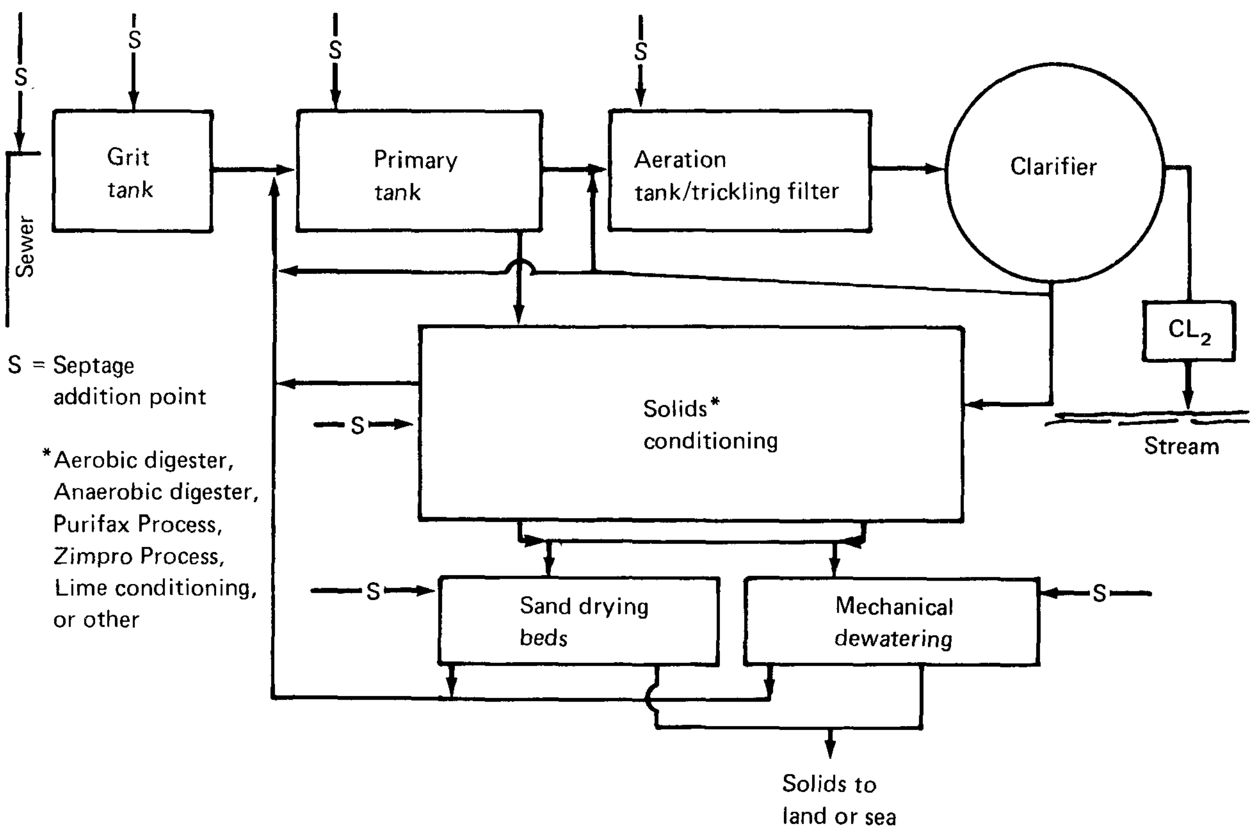

Figure II-3. Septage addition points in wastewater treatment plants.

ties, septage is considered a sludge because it is the product of an anaerobic settling/digestion tank, and it has approximately the same TS concentration as raw municipal sludge. The septage application points, if treated as a sludge, may include sludge stabilization, sand bed drying, or a mechanical dewatering process. The decision of where to apply the septage should be determined after a statistically significant sampling and analysis of a locale's septage, including:

- Solids loading
- Oxygen demand
- Toxic substances
- Foaming potential
- Nutrient loading (N and P), where required

These factors, combined with a plant's layout, design capacity, present loading, and the following criteria, provide the design professional with sufficient information for a reasonable septage treatment scheme for a wastewater treatment facility.

When septage is added to an upstream sewer or discharged at a treatment plant, there should be a suitable hauler truck discharge facility. It should include a hard-surfaced ramp that leads to an inlet port and is able to accept a quick-disconnect coupling directly attached to the truck's outlet. This significantly reduces odor problems. Washdown water should also be provided for the hauler so that spills can be cleaned up. Recording the time and volume and the name of the hauler is vital for operation and billing purposes. The Columbia Avenue Plant in Portland, Oreg., and Seattle Metro's Renton facility use a plastic charge plate or magnetically coded card and card reader to obtain such information.

Pretreatment

Treatment plants handling septage have experienced better operation when septage is pretreated. Pretreatment generally includes screening, using bar screens with 3/4- to 1-inch openings; grit removal; and pre-aeration or prechlorination if it is an aerobic process. Grit removal by cyclone classifiers has been done successfully in Babylon and is included in the new Bay Shore plant, both on Long Island, N.Y. Usually, separation of inorganic matter larger than 150 mesh is sufficient. Equalization/storage tanks with 2 days' average septage flow and mixing capability should also be provided. To further attenuate odors, enclosing the storage tanks and ozonization in tank vent lines should be considered. Pumping equipment should be used to apply a continuous dose of septage into the desired unit. Operators report slug or intermittent doses of septage are difficult to treat and may seriously upset biological treatment systems.

Primary Treatment

Feige's[8] report for the EPA indicated that neither natural settling nor adding lime or polyelectrolytes resulted in consistent liquid-solids septage separation. Tilsworth[7] characterized raw septage as relatively nonsettleable, as determined by a settleable-solids volume test, from 0 to 90 percent with 24.7 percent as the average volume.

Tawa[30] found that poor settling characteristics generally could be expected from septage and that it could be divided into three types. Type 1, from septic tanks pumped before necessary, settled well and was found in 25 percent of the samples. Type 2, from normally operating systems, showed intermediate settling characteristics and was found in 50 percent of the samples. Type 3, which exhibited poor settling, was found in 25 percent of the samples and was from tanks overdue for pumping. All samples were between 1 and 6 years old.

Unless chemicals are added to it, septage settles very poorly. In a study on treatment of Alaskan septage, Tilsworth[7] found that only 50 percent of the samples settled by more than 10 percent after 30 minutes, as shown in figure II-4. About one-third of the samples are not represented in the figure because they settled less than 1 percent during the 30-minute test period.

Elutriation, in terms of the settling of septage in a septage-sewage mixture, is reported to yield better results. Carroll[31] reports that up to 75 percent of septage SS can be expected to settle in a sewage treatment plant's primary sedimentation basins. An EPA study found 55 to 65 percent SS removals in a primary clarifier, but only 15 to 25 percent BOD removals.[32]

Activated Sludge

Septage may be added to the activated sludge process if additional aeration capacity is available, the plant is organically and hydraulically loaded below design capacity, the septage metals content can be diluted sufficiently, and foaming potential is low or controllable. Very limited quantities of septage may be added without changing the sludge-wasting rates.

At the Weaverville Wastewater Treatment Plant in Trinity County, Calif., 400 gal/d slug dumps were handled without significant upset at a 0.5 million gallons per day plant flowing at 40-percent capacity.

In a report to the U.S. Forest Service,[31] CH_2M/Hill recommended various levels of septage addition for several kinds of activated sludge plants. This information, modified by authors' field investigations,[22] is presented in figures II-5 and II-6.

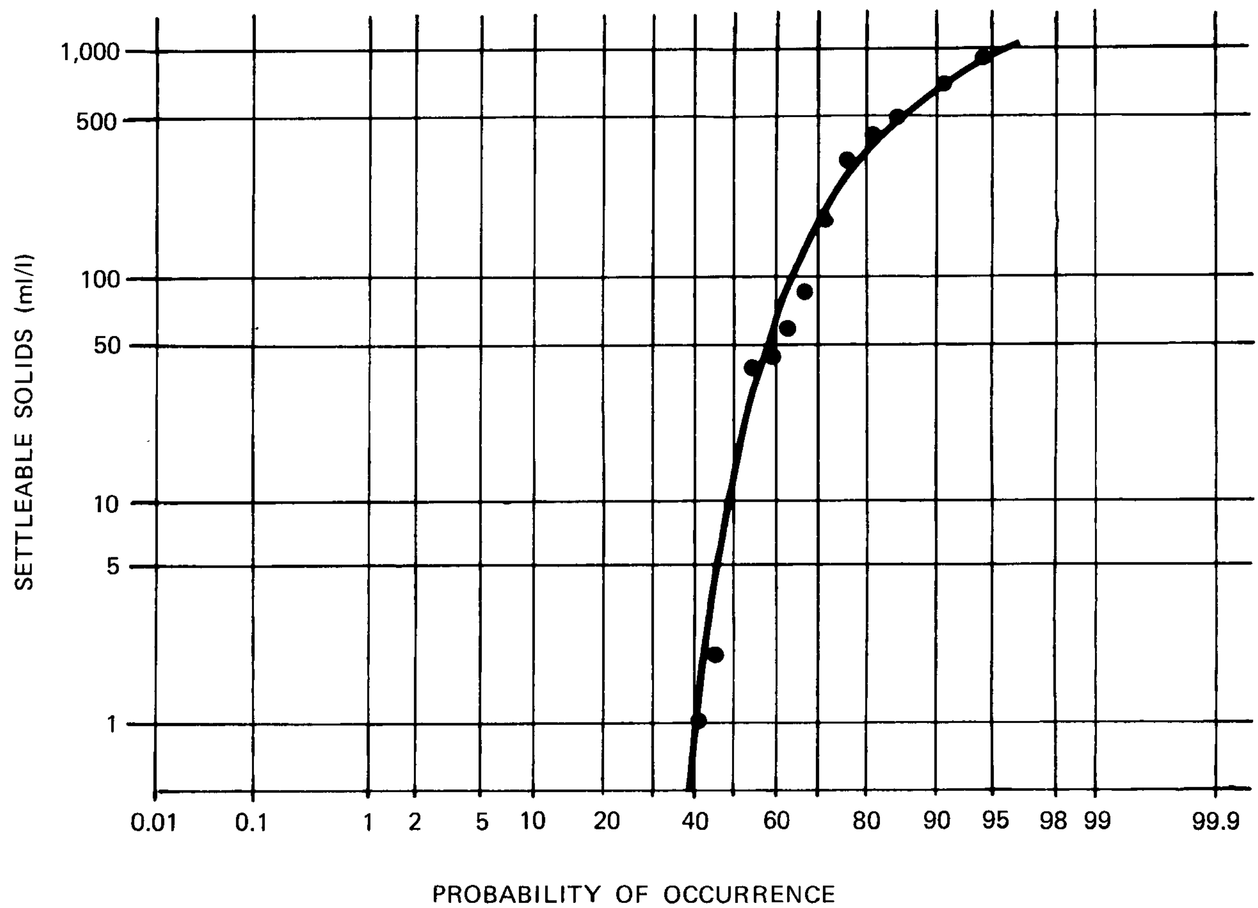

Figure II-4. Probability of solids reduction—liquid interface height after 30 minutes settling of Alaskan septage samples.

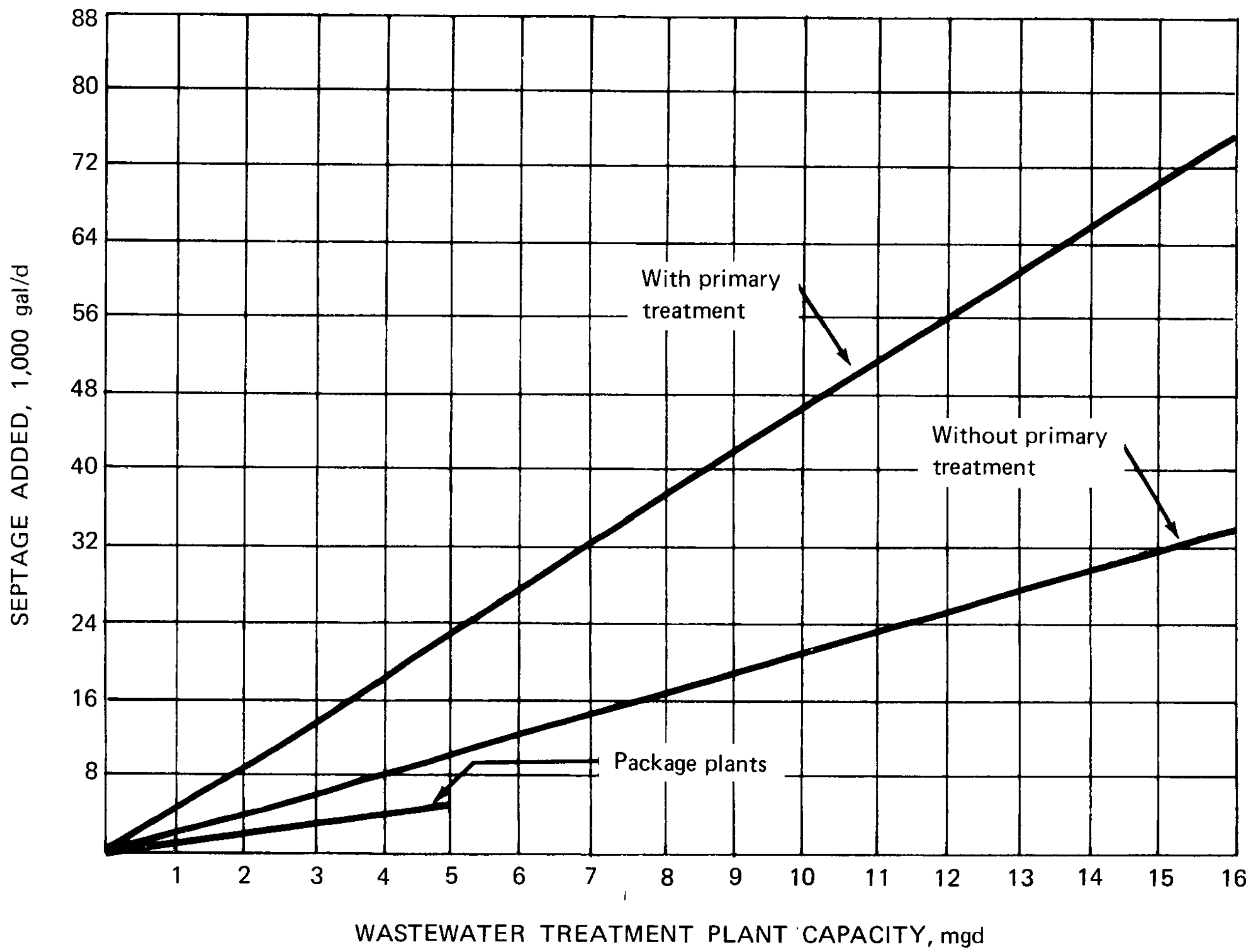

Figure II-5. Septage additions to activated-sludge wastewater treatment plants (no equalization facilities).[31]

The use of slug dumping of septage may depend on limiting the increase in mixed-liquid suspended solids (MLSS) to 10 percent per day to maintain a relatively stable sludge, as shown in figure II-5. Higher loadings and wasting rates than the resident aquatic biomass is acclimated to may result in a poor-settling sludge.[33] Severe temporary changes in loading beyond the 10- to 15-percent MLSS increase may cause a total loss of the system's biomass.[31]

Package treatment plants should not accept septage for slug dumping if their design capacity is less than 100,000 gal/d.[31] In a study for the U.S. Forest Service, CH_2M/Hill determined that package treatment plants can treat septage at approximately 0.1 percent of the plant design capacity, whereas modified activated sludge can treat septage at twice the rate of a package plant. Conventional activated sludge plants can treat septage at about four times the rate of package plants.[31]

In plants with holding and metering facilities, septage may be bled into the waste-flow stream at considerably greater rates than would be allowable if only slug-dumping procedures were available.

An EPA study[32] fed septage at a controlled rate of 2 to 13 percent of the total influent flow to one of two activated sludge units. With a control unit food-to-microorganism (F/M) ratio of 0.4 and a septage-sewage unit F/M of 0.8, effluent BOD and SS characteristics were similar. Effluent COD of the unit receiving septage increased when septage was loaded at 10 to 13 percent of plant

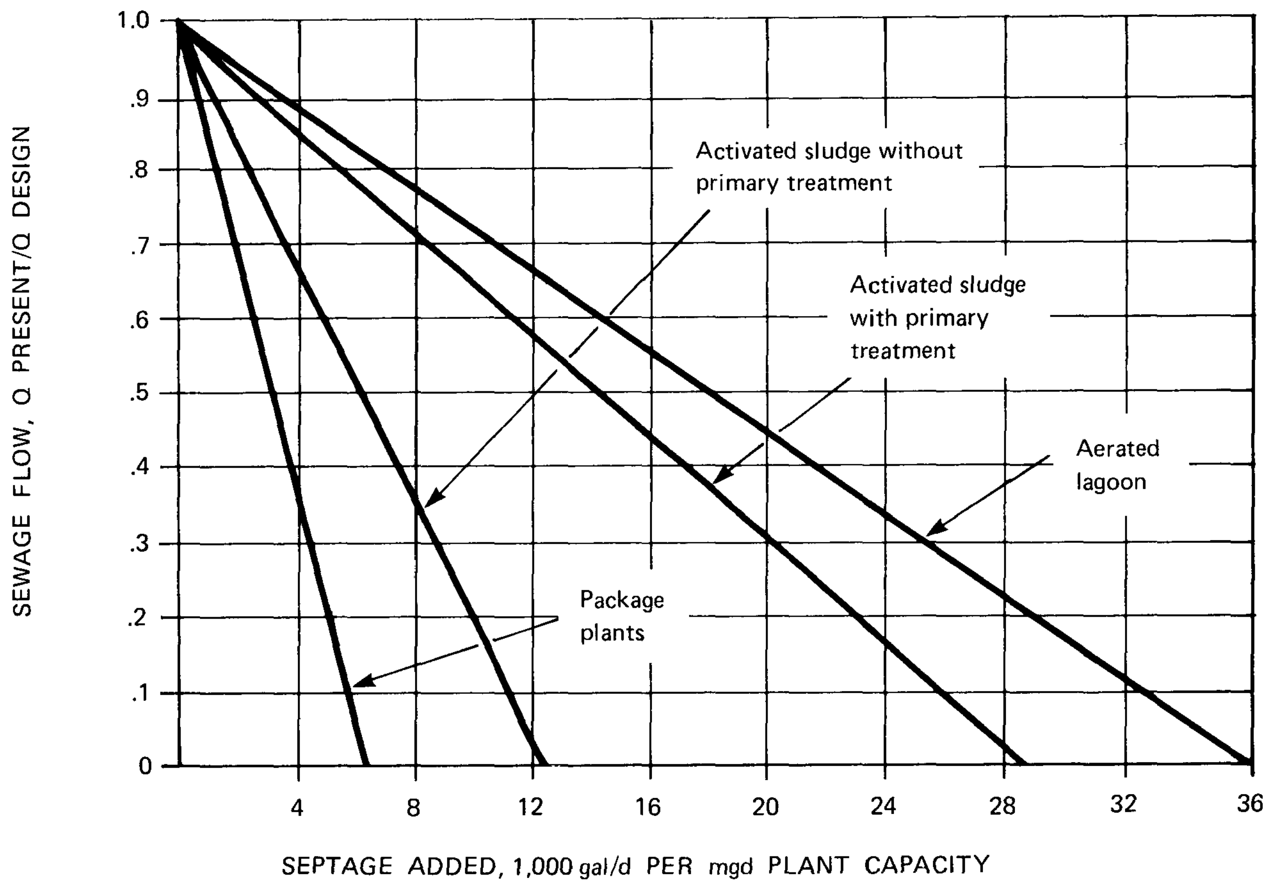

Figure II-6. Septage addition to wastewater treatment plants (with equalization facilities).

flow. When a lower F/M ratio of 0.5 to 0.6 was used in the septage unit, it had a superior performance because nocardia, a procaryotic filamentous actinomycete often associated with bulking, was controlled.

Figure II-6 is based on research reported in the literature and field investigations.[22] Again, it demonstrates that package plants with design capacities under 100,000 gal/d should not accept septage. Depending on the present plant flow compared with the design plant flow, a biological treatment reserve can be estimated that will allow for a certain level of septage to be adequately treated. Under identical loading conditions, the ratio of septage addition to various kinds of treatment plants would be:

Treatment plants	Relative volumes of septage addition
Package plants	1.00
Activated sludge (no primary treatment)	2.08
Activated sludge (conventional)	4.83
Aerated lagoons	6.00

Figure II-6 represents continuous septage addition to a facility for a fully acclimated biomass. It is recommended that an initial septage feed to an unacclimated system should be substantially less than shown, that is, on the order of 10 percent of the graph values. Further gradual increases in daily septage loading should be made over a 2- to 3-week period up to the maximum amount shown in the figure. Oxygen capacity must be checked continuously and gradual changes made in sludge age.

Figure II-7 shows the additional oxygen requirements when septage is added in activated sludge treatment plants. Treatment facilities should be analyzed to determine if oxygen requirements or mixing requirements are controlling factors.

Because septage has higher oxygen demands than raw sewage on a unit BOD_5 basis, an additional oxygen supply for activated sludge plants that accept septage having primary treatment would be 40 pounds of O_2 per 1,000 gallons of septage added. For plants without primary treatment, an additional 80 pounds of O_2 per 1,000 gallons of septage added should be provided. Package treatment plants have an oxygen requirement similar to plants without primary treatment.

Feng[34] has shown higher sludge ages (10 days versus 4 days) result in higher percentage BOD removal and less sludge production than do lower sludge ages (fig. II-8). Wasting must be adjusted gradually with increased loads to obtain a sludge age that produces the optimum balance between

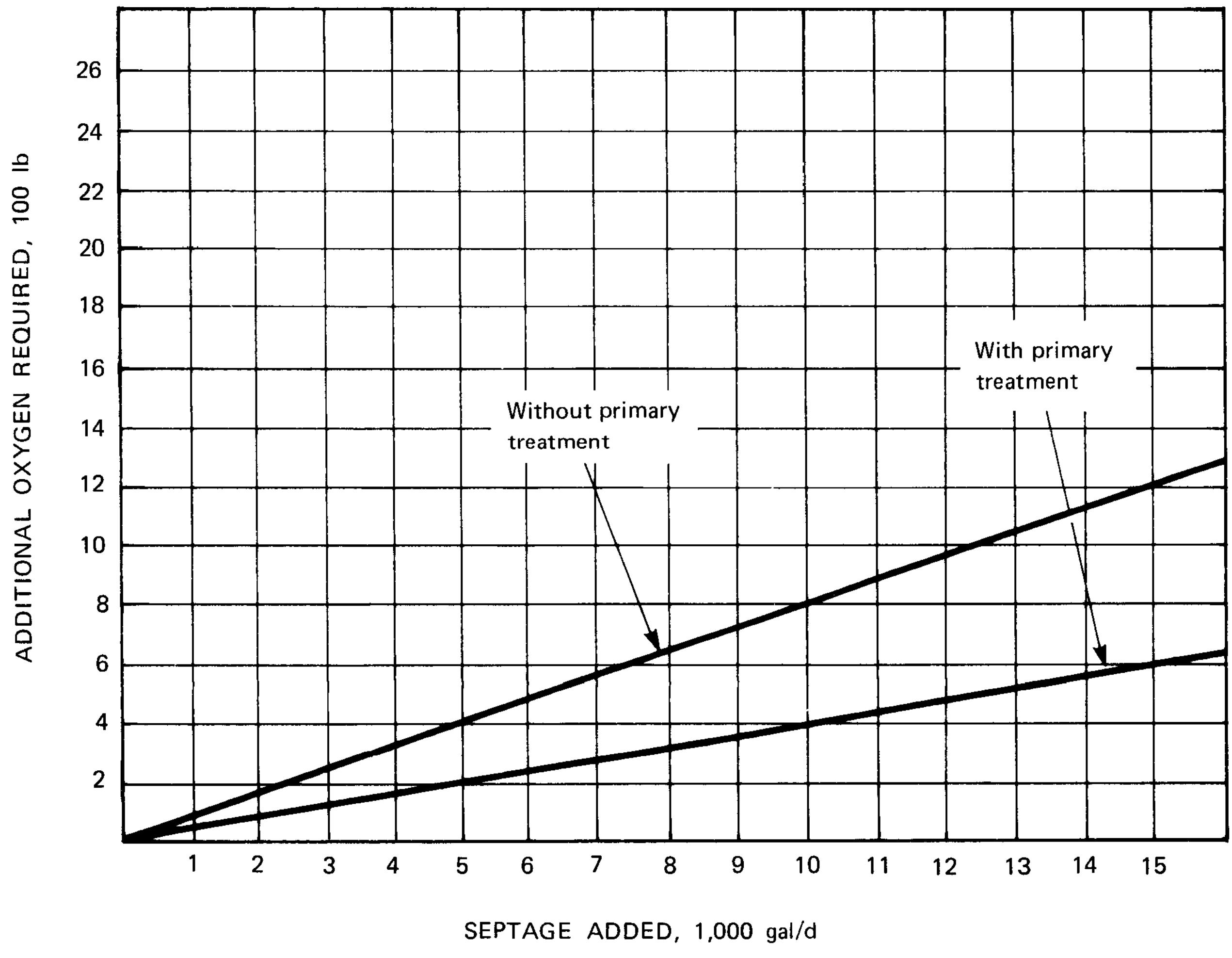

Figure II-7. Additional oxygen required for septage additions in activated-sludge treatment plants.[31]

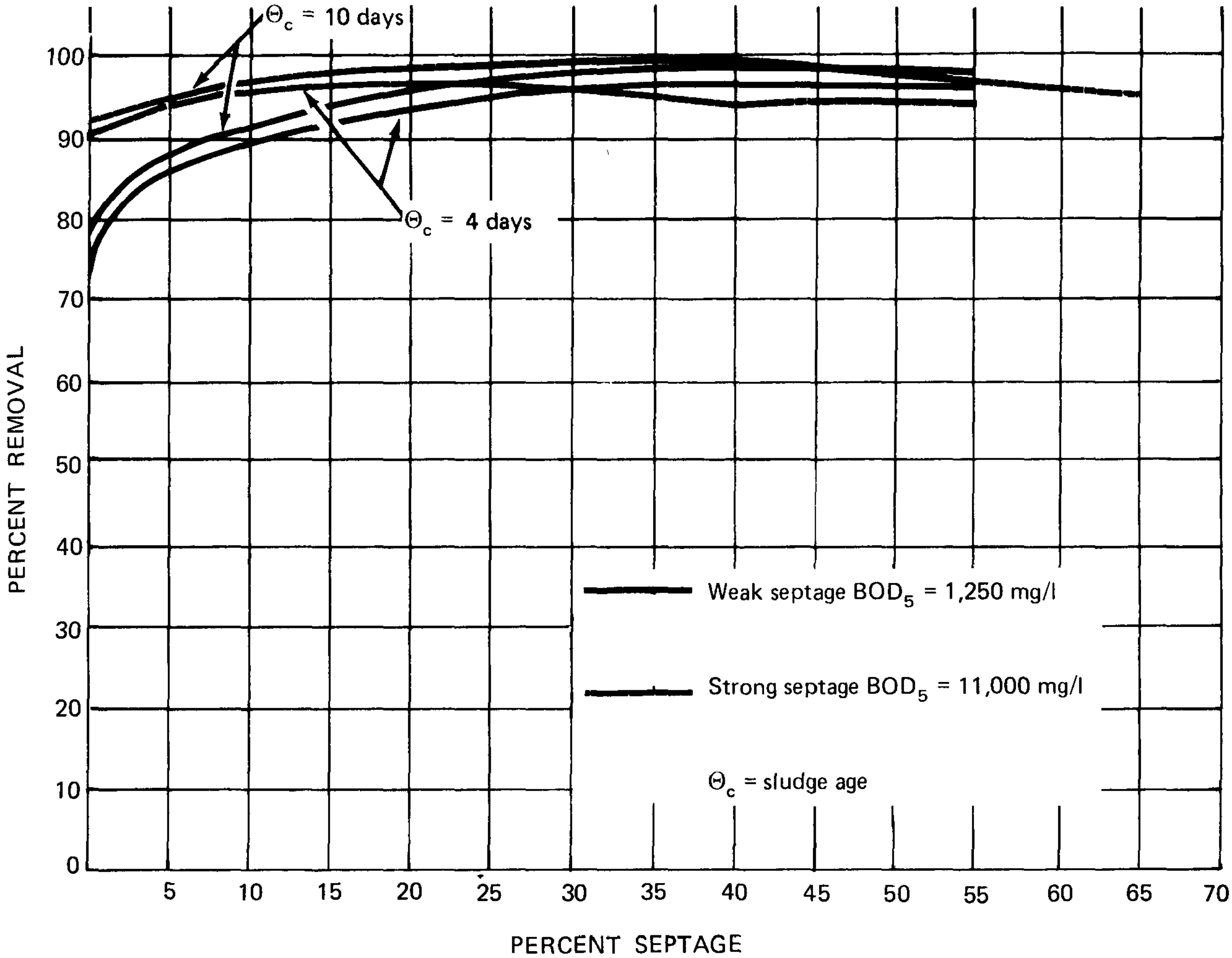

Figure II-8. BOD_5 removal from septage-sewage mixtures in batch activated sludge process.[34]

aeration tank efficiency and good settling characteristics. A high sludge age generally produces a light sludge with poor settling ability but good substrate removal characteristics. The reverse is often true for a very young sludge.

At one New York plant, septage is bled into the liquid stream at a rate inversely proportional to that of the sewage flow.[23] The procedure takes advantage of a larger excess aeration capacity during lower loading times. Orange County, Fla.,[35] added septage proportionate to sewage flow rates. Both plants have experienced some operational problems.

Some odor and foaming problems have been reported in aeration systems; however, the odor usually dissipated within 6 to 24 hours,[7,10] and foaming was not apparent in all cases. Commercial defoamers, such as decyl alcohol, and aeration-tank spray water have been used to reduce foaming.

Attached Growth Systems

Systems that use attached growth aerobic treatment processes, such as trickling filters and rotating biological contactors, are usually more resistant to upsets from changes in organic or hydraulic loadings and are suitable for septage treatment.[23,36,37]

In trickling filters, additional recirculation has been shown to adequately dilute septage concentrations and diminish chances of plugging the media. At Huntington, Long Island, N.Y., 30,000 gal/d septage is treated at a 1.9 million gallons per day facility.[23] BOD_5 reductions of 85 to 90 percent have been observed concurrent with SS reductions of 85 percent.

Rotating biological contactors use a long detention time and a continually rotating biological medium that is reportedly resistant to upsets. At Ridge, Long Island, N.Y., a BOD reduction of 90 percent, a COD reduction of 67 percent, and a total suspended solids reduction of 70 percent were reported. Flow equalization of a low-strength septage and a surface loading of 2 gal/d/ft^2 produced these results.

Aerobic Digestion

An alternative to considering septage as a concentrated wastewater is to assume it is the product of an unheated digester and, therefore, a sludge. Many researchers have reported good results in aerobic digestion of septage or septage-sewage mixtures. Jewell[10] reports odors diminished, but time needed to produce an odor-free sludge varied up to 7 days.

Tilsworth[7] reported a high degree of septage biodegradability at a 10-day aeration time, resulting in a BOD reduction of 80 percent and a VSS reduction of 41 percent. Chuang,[25] treating anaerobically digested septage with an aerobic digester, reported 36-percent VS removal at 40 days' aeration under a loading of 0.0016 pound VS per cubic foot per day. After 22 to 63 days' aeration, Howley[17] found a 43-percent VSS reduction and a 75-percent COD reduction.

Orange County, Fla., adds septage to aerobic digesters at the rate of 5 percent of total sludge flow and obtains good reductions at a loading of 0.15 pound VS per cubic foot per day. Bend, Oreg., obtained good removal by adding 13 percent septage to 87 percent sludge at a loading of 0.02 pound VS per cubic foot per day, with 15 to 18 days' aeration time.[38]

Tilsworth[7] observed gas transfer characteristics for septage and found that α, the ratio of gas transfer efficiency to tapwater, and β, the ratio of O_2 saturation concentration to tapwater, approached unity after 1 to 2 days' aeration. Before 1 day, α and β were in the range of 0.4 to 0.6.

Jewell[39] found both dewatering and settleability improved with aeration, but that aeration time required to effect significant improvement varied.

Before adding septage to the aerobic digestion process, aeration capacity, toxic metal or chemical accumulations, and increased solids to be disposed of should be investigated. Investigators consistently have reported initial repulsive odors and foaming problems.[7,17,25,35,39]

When considering septage addition to aerobic digesters, recommendations should include screening, degritting, flow equalization, and analyses of excess digestion capacity and peripheral effects on other processes such as solids handling. An initial septage addition should be limited to approximately 5 percent of the existing sludge flow. Further septage additions should be gradual.

Studies in high-temperature auto-oxidation of septage are planned[40] and may prove promising as a low-cost, efficient, solids-destruction technique.

Anaerobic Digestion

Septage in Tallahassee, Fla., is treated in an unheated (20° C to 30° C) anaerobic digester. With an influent septage concentration of 17,700 mg/l TS, a VS reduction of 56 percent was reported after an 82-day retention time at a loading of 0.01 pound VSS per cubic foot per day. Large quantities of grit in the septage required draining and cleaning of the open digester after only 3 years' operation. Leseman and Swanson[41] analyzed volatile acid distribution concentrations in the digester contents. The volatile acid-to-alkalinity ratio varied from 0.34 to 0.83. The 8-month volatile acid concentration averaged 703 mg/l and ranged from 408 mg/l to 1,117 mg/l at a consistent pH of 6.0. The progression of volatile acid concentrations in the digester, from two to five carbon acids, showed acetic as 276 mg/l, propionic as 294 mg/l, isobutyric as 14 mg/l, butyric as 49 mg/l, isovaleric as 28 mg/l, and valeric as 42 mg/l. The digester had an open cover, so gas production could not be monitored. Supernatant from this digester is pumped to the sewage sludge anaerobic digester.

Jewell[10] reported a 45-percent reduction in VSS from a bench-scale digester loaded at 0.05 pound VSS per cubic foot per day, with a 15-day hydraulic retention time. Gas production varied from 4.2 to 7.6 cubic feet per pound COD up to a loading of 0.08 pound VSS per cubic foot per day, at which point gas production fell off dramatically, possibly indicating poisoning of the system by a toxic chemical concentration from an unknown source.

Chuang[25] reported a 92-percent VS removal from a heated anaerobic digester loaded at 0.08 pound VSS per cubic foot per day with a 15-day hydraulic retention time. Incoming solids ranged from 0.3 percent to 8 percent, and TS reduction was more than 93 percent. BOD reductions averaged 75 percent, from 6,100 mg/l influent to 1,500 mg/l effluent.

Howley[17] recommends a maximum septage addition of 2,130 gal/d to each 14,500 gallons sewage sludge added per day per million gallons of digester capacity, with a detention time of 30 days and a loading of 0.08 pound VSS per cubic foot per day. Good operation of anaerobic digesters requires that toxic materials be limited.

Septage should be screened, degritted, and equalized before it is added to single-stage digesters. Digesters should be cleaned on a regular schedule, such as every 2 to 3 years, or as required.

Monitoring digester performance includes long-term evaluation of volatile acid/alkalinity ratios and gas production. Mixing is vital to preventing a sour digester from developing point-source failure from a septage load containing high volatile acid concentrations.

In systems with multiple tanks, all the preceding suggestions should be followed. Spreading the septage among a number of digesters reduces septage concentrations. Recycling material from the bottom of a secondary digester or from another well-buffered primary digester at a rate of up to 50 percent of the raw feed per day has been found helpful. Temperature and mixing should also be adjusted for maximum performance.[42]

Mechanical Dewatering

Islip, Long Island, N.Y., uses a vacuum filter to dewater 100,000 gal/d of chemically conditioned septage. A design basis of 6 pounds per hour per square foot of surface area was used and appears to be satisfactory.[43] Adding lime at a rate of about 190 lb/ton of dry solids and 50 gal/ton of dry solids standard concentration ferric chloride solution are added before vacuum filtering.

In a study at Clarkson College, Crowe[44] had successful results with vacuum filtration of mixtures of raw septage and digested sludge with up to 20 percent raw septage by volume. Chemical preconditioning with lime, ferric chloride, and polymers was required at doses typical of domestic sludge. He observed dewatering characteristics similar to those of mixtures without septage. The filtrate contained only 5 to 10 percent of the raw septage COD.

Sand Drying Beds

Sand drying has been used to dewater septage with varying success. Anaerobically digested septage is reported to require two to three times the drying period of digested sludge.[41] After treatment in aerated lagoons and batch aerobic digesters, dewatering simulation studies yielded a septage capillary suction time (CST) on the order of 200 seconds versus about 70 seconds for sewage treatment plant sludges. A lower CST can be correlated to a faster dewatering time. The CST's of raw septage were found to range from 120 to 825 seconds; the mean was 450 seconds.[45] Adding lime to septage before sand bed dewatering has vastly improved dewatering characteristics. Feige[8] found that adding 180 pounds of lime per ton dry solids, or 30 pounds per 1,000 gallons of septage based on 40,000 mg/l TS, raised the pH to 11.5 and dried to 25 percent solids in 6 days and 38 percent solids in 19 days. An application depth of greater than 8 inches is not recommended because it slowed the drying process. The filtrate analysis showed that most heavy metals were tied up in the solids, fecal coliforms were killed effectively, fecal streptococci were more resistant than fecal coliforms, and odors were significantly reduced. Filtrate quality was generally good, but further treatment before discharge was recommended.[23]

Perrin found other chemicals worked well in modifying the ability of septage to dewater. From a mean initial CST of 450 seconds, septage showed a dewatering ability of 50 seconds after adding an average of either 1,360 mg/l ferric chloride, 1,260 mg/l alum, 1,360 mg/l Purifloc C-31, or 2,480 mg/l Purifloc C-41.[45] Perrin also studied the effects of freezing on dewatered samples of septage after treatment in aerated lagoons or batch aerobic digesters. Freezing lowered the CST from 225 seconds to 42 seconds, an 80-percent decrease in dewatering time.

If septage is to be placed on sand drying beds, treatment to a consistent CST range of 50 to 70 seconds is recommended. Further treatment of underdrainage would be required in most cases.

COSTS

Of all the alternatives investigated, land disposal was reported to have the lowest operation and maintenance costs, from $1.50 to $5.00 per 1,000 gallons, exclusive of the cost of the land. Lagoon treatment is reported to cost between $5.00 and $10.00 per 1,000 gallons. The cost of septage treatment in sewage treatment plants varies widely, but typically runs about $15.00 per 1,000 gallons. Composting by the Lebo process is reported to cost approximately the same as disposal in wastewater treatment plants. Physical chemical treatments, such as the Purifax Process and chemical stabilization, range from average costs similar to those found in disposal at treatment plants to double or triple that figure.

A nationwide survey of 42 wastewater treatment plants[22] determined that only about one-half charged for septage disposal based on treatment costs (fig. II-9). Some charge prohibitive rates to avoid septage; others place a minimal charge on septage to ensure against illegal dumping at an unauthorized site. At those plants surveyed, the average charge for septage was $15.18 per 1,000 gallons. An additional 20 to 30 plants contacted, however, either placed no charge on septage disposal or levied only a yearly fee, most often in the range of $50 to $300 per truck.

Many variables affect treatment costs, including local funding requirements; eligibility for State or Federal funds; necessity for industrial cost recovery formats; local taxes assessed in lieu of, or to offset, treatment plant expenses; level of pollutant removal capacity; climate; present loading versus design plant capacity; and cost of land. It is easy to understand, therefore, the broad range of charges for treatment plant septage disposal.

An estimate was performed to determine a reasonable charge a homeowner could expect to pay for having a 1,000-gallon septic tank cleaned, assuming no additional work was needed. It was based on a 15-mile haul to the disposal point, 2 hours travel time per load, vehicle depreciation and insurance of $4,000 per year, and estimated union wages. Depending on the level of profit and a disposal cost not exceeding $15, a reasonable charge appears to be $40 to $60.

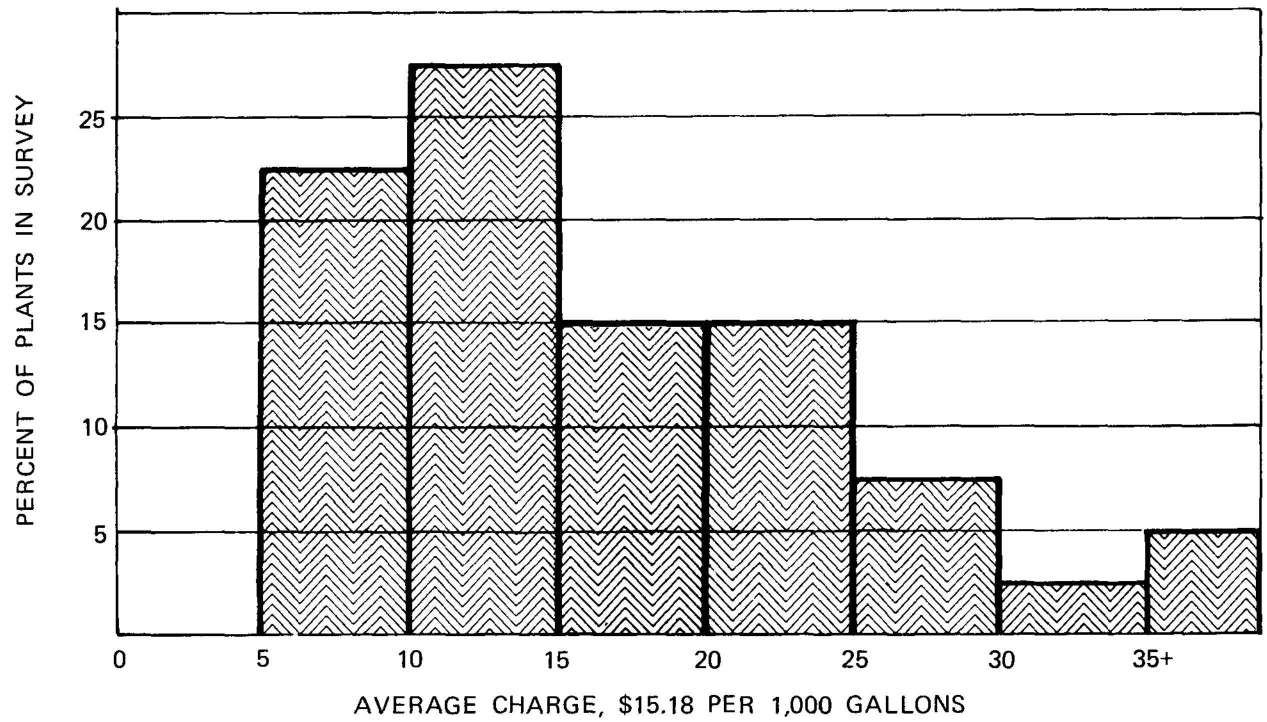

Figure II-9. Septage disposal charges at 42 wastewater treatment plants, in dollars per 1,000 gallons of septage.

Fees charged to homeowners ranged from a low of $20 to $25 per 1,000 gallons in parts of Long Island, N.Y., to around $100 per 1,000 gallons in areas of New Jersey, Connecticut, and Oregon. Rural areas in New England had slightly lower charges, $25 to $40 per 1,000 gallons; in the rest of the country, charges were mostly in the range of $40 to $60 per 1,000 gallons. These charges depend on the distance from the septic tank to the disposal point (especially if more than 15 miles) and the disposal fee.

SUMMARY AND CONCLUSIONS

Various alternatives for septage disposal have been presented. Good design practices and conscientious operation are necessary to preclude septage from polluting the environment.

The method chosen should depend on an evaluation of local needs by the design professional, cost/effectiveness of solutions, and environmental weighing of impact factors.

For example, although land disposal appears most cost/effective, local constraints concerning land use, odors, or poor soil may preclude this option. Similarly, a more expensive option, such as composting, may prove viable if it meets such local requirements as those on land restrictions or odor prevention and conversion of excess wood waste into a marketable product.

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METRIC CONVERSION TABLES

Recommended Units

Description	Unit	Symbol	Comments	Customary Equivalents*
Length	meter	m	*Basic SI unit*	39 37 m = 3 281 ft = 1 094 yd
	kilometer	km		0 6214 mi
	millimeter	mm		0 03937 in
	micrometer or micron	μm or μ		3 937 X 10^{5} in = 1 X 10^{4} Å
Area	square meter	m^2		10 76 sq ft = 1 196 sq yd
	square kilometer	km^2		0 3861 sq mi = 247 1 acres
	square millimeter	mm^2		0 001550 sq in
	hectare	ha	The hectare (10,000 m^2) is a recognized multiple unit and will remain in international use	2 471 acres
Volume	cubic meter	m^3		35 31 cu ft = 1 308 cu yd
	litre	l		1 057 qt = 0 2642 gal = 0 8107 X 10^{-4} acre ft
Mass	kilogram	kg	*Basic SI unit*	2 205 lb
	gram	g		0 03527 oz = 15 43 gr
	milligram	mg		0 01543 gr
	tonne	t	1 tonne = 1,000 kg	0 9842 ton (long) = 1 102 ton (short)
Force	newton	N	The newton is that force that produces an acceleration of 1 m/s^2 in a mass of 1 kg	0 2248 lb = 7 233 poundals
Moment or torque	newton meter	N·m	The meter is measured perpendicular to the line of action of the force N Not a joule	0 7375 lb ft 23 73 poundal ft
Flow (volumetric)	cubic meter per second	m^3/s		15 850 gpm = 2,119 cfm
	liter per second	l/s		15 85 gpm

Recommended Units

Description	Unit	Symbol	Comments	Customary Equivalents*
Velocity linear	meter per second	m/s		3.281 fps
	millimeter per second	mm/s		0 003281 fps
	kilometers per second	km/s		2,237 mph
angular	radians per second	rad/s		9 549 rpm
Viscosity	pascal second	Pa s		0 6722 poundal(s)/sq ft
	centipoise	Z		1 450 X 10^{7} Reyn (μ)
Pressure or stress	newton per square meter or pascal	N/m^2 or Pa		0 0001450 lb/sq in
	kilonewton per square meter or kilopascal	kN/m^2 or kPa		0 14507 lb/sq in
	bar	bar		14 50 lb/sq in
Temperature	Celsius (centigrade)	°C		(°F−32)/1 8
	Kelvin (abs)	°K		°C + 273 2
Work, energy, quantity of heat	joule	J	1 joule = 1 N m where meters are measured along the line of action of force N	2 778 X 10^{7} kw hr = 3 725 X 10^{7} hp-hr = 0 7376 ft-lb = 9 478 X 10^{-4} Btu
	kilojoule	kJ		2 778 X 10^{-4} kw hr
Power	watt	W	1 watt = 1 J/s	44 25 ft lbs/min
	kilowatt	kW		1,341 hp
	joule per second	J/s		3 412 Btu/hr

Application of Units

Description	Unit	Symbol	Comments	Customary Equivalents*
Precipitation, run-off, evaporation	millimeter	mm	For meteorological purposes, it may be convenient to measure precipitation in terms of mass/unit area (kg/m^2) 1 mm of rain = 1 kg/m^2	
Flow	cubic meter per second	m^3/s		35 31 cfs
	liter per second	l/s		15 85 gpm
Discharges or abstractions, yields	cubic meter per day	m^3/d	1 l/s = 86 4 m^3/d	0 1835 gpm
	cubic meter per year	m^3/year		264 2 gal/year
Usage of water	liter per person per day	l/person/day		0 2642 gcpd

Application of Units

Description	Unit	Symbol	Comments	Customary Equivalents*
Density	kilogram per cubic meter	kg/m^3	The density of water under standard conditions is 1,000 kg/m^3 or 1,000 g/l or 1 g/ml	0 06242 lb/cu ft
Concentration	milligram per liter (water)	mg/l		1 ppm
BOD loading	kilogram per cubic meter per day	$kg/m^3/d$		0 06242 lb/cu ft/day
Hydraulic load per unit area, e g , filtration rates	cubic meter per square meter per day	$m^3/m^2/d$	If this is converted to a velocity, it should be expressed in mm/s (1mm/s = 86 4 m^3/m^2/day)	3 281 cu ft/sq ft/day
Air supply	cubic meter or liter of free air per second	m^3/s l/s		
Optical units	lumen per square meter	lumen/m^2		0 09294 ft candle/sq ft

*Miles are U S statute, qt and gal are U S liquid, and oz and lb are avoirdupois

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