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Wastewater Management Alternatives for Rural Lakefront Communities

Robert H. Griffin
Research Assistant

and

Richard R. Noss
Assistant Professor of Civil Engineering

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Technical Report

WASTEWATER MANAGEMENT ALTERNATIVES FOR RURAL LAKEFRONT COMMUNITIES

Robert Griffin, Graduate Research Assistant
Richard R. Noss, Assistant Professor

Department of Civil Engineering
Environmental Engineering Program
University of Massachusetts
Amherst, MA 01003

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Executive Summary

This project report presents an in-depth review of several wastewater management techniques particularly suitable for implementation at unsewered, rural lakefront communities in Massachusetts.

Rural communities, because of necessarily lower housing densities than their urban counterparts, often present difficult financial problems when attempting to apply conventional wastewater management technology (centralized collection and treatment). In the absence of community wastewater removal systems, on-site treatment becomes necessary for habitation of that region. Traditionally this has meant septic tanks followed by soil absorption systems for treatment and disposal of sewage.

Septic tank - soil absorption systems, unfortunately, have not always provided reliable or adequate treatment of wastewater, especially when applied to lake shore development. The occasional high failure rates of soil absorption systems can be attributed to improper application of soil absorption technology rather than inadequacies inherent to the technology. Improper application has been the result of inadequate site evaluation techniques, poor regulatory design criteria, and inadequate construction procedures.

In addition to traditional septic tank systems, there are a multitude of wastewater management systems potentially applicable to rural lakefront communities with site conditions such as those found in Massachusetts. The purpose of this report is to identify and evaluate a manageable set of alternatives appropriate for implementation at Massachusetts rural lakefront communities. The decision criteria used in this screening process included: (1) reliability of performance, (2) adequacy of treatment performance, (3) acceptability without requiring significant cultural or sociological change by the user, (4) suitability for implementation at some Massachusetts rural lakefront locations, (5) maintenance and operational requirements, and (6) a need for review. For example, systems relying on evapotranspiration appear unsuitable for regular use in Massachusetts. Extreme water conservation systems, alternative toilets and the like were rejected for questions about performance, social acceptance and long term maintenance. In the future, progressive disposal systems such as these may be desirable. Today however, systems that remove and treat wastewater at reasonable cost with little attention required of the homeowner seem more favorable.

In short, in the authors' judgment, the only systems that can be considered for on-lot wastewater treatment are those that require practically no maintenance. Thus a large portion of this report evaluates and discusses only traditional septic

tank-soil absorption systems and variations of this system. If collection of wastewater is feasible such that systems can be designed to serve clusters of homes, then formally delegated maintenance responsibilities become possible and "higher technology" systems become feasible. The last two chapters of this report look at alternatives for reducing the cost of small scale collection and treatment systems so that such cluster treatment schemes become feasible.

Another means of escaping from the "no maintenance" restriction on individual systems is to develop innovative operation and maintenance arrangements such as community responsibility. Such poolings of resources allow a professional to be hired to manage and maintain wastewater disposal systems, thereby allowing higher technology and higher maintenance systems to be used. Such operation and maintenance arrangements are the exception at present. Consideration of this approach to rural wastewater management was beyond the scope of this project. The discussion in this report emphasizes the mechanisms governing small flow wastewater management system behavior, for it is the authors' opinion that understanding these mechanisms is a necessary step towards rational evaluation of wastewater management systems.

Septic tanks and on-site soil absorption systems, when properly designed, constructed and maintained, provide satisfactory renovation of wastewater. Where soils are unsuitable for absorption system use, either due to excessive or insufficient permeability, a modification of traditional soil absorption systems, the wastewater disposal mound, often presents a viable alternative. Unfortunately, lake shore developments are often plagued by inadequate on-lot disposal systems. Old developments often do not have any significant wastewater treatment system; newer systems are often improperly designed or located. A common result is excessive lake eutrophication due to phosphorus introduction from these disposal systems. A section of the report is devoted to on-site phosphorus retention processes within the soil matrix. In some cases, installation of a new, properly designed, soil absorption system will sufficiently mitigate introduction of phosphorus to a waterbody from soil disposal systems.

Where on-site systems are not the answer, perhaps because proper site conditions do not exist and the cost to create suitable conditions is prohibitive, a more traditional treatment scheme, centralized collection and treatment, is a remaining alternative.

Sewage collection in traditional gravity flow pipelines is constrained by minimum velocity requirements, designed to keep solids suspended and prevent clogging of the pipeline. To reduce the depth of construction, pumping stations may be constructed periodically along the flow path. As a result, traditional collection systems can become very complex and expensive construction projects when applied to lakeshore development.

Alternative sewage collection systems are now available that may make collection systems to centralized or sub-regional treatment facilities economically feasible. Three such systems are evaluated and presented in this report: pressure collection systems, vacuum collection systems and small diameter gravity sewers (including variable grade design). Each system is described and its design, construction, and maintenance reviewed. These alternative systems generally require more maintenance than traditional sewerage systems, but the move to collective rather than individual wastewater treatment makes this acceptable.

When a centralized collection system is used, biological wastewater treatment schemes (a type of "higher technology" treatment) become feasible or necessary, especially if suitable soils cannot be located nearby. Small flow systems that provide biological wastewater treatment are commonly known as "package plants" for they are often prefabricated and delivered to a site ready to be hooked up to influent sewer, power supply, and effluent discharge. Two biological wastewater treatment processes employed in package plants, suspended growth and attached growth, are reviewed in Chapter Seven.

CHAPTER 1

INTRODUCTION

This project report presents an in-depth review of several wastewater management techniques particularly suitable for implementation at unsewered, rural lakefront communities in Massachusetts.

Rural communities, because of necessarily lower housing densities than their urban counterparts, often present difficult financial problems when attempting to apply conventional wastewater management technology (centralized collection and treatment). In the absence of community wastewater removal systems, on-site treatment becomes necessary for habitation of that region. Traditionally this has meant septic tanks followed by soil absorption systems for treatment and disposal of sewage.

Septic tank - soil absorption systems, unfortunately, have not always provided reliable or adequate treatment of wastewater, especially when applied to lake shore develoment. The occasional high failure rates of soil absorption systems can be attributed to improper application of soil absorption technology rather than inadequacies inherent to the technology. Improper application has been the result of inadequate site evaluation techniques, poor regulatory design criteria, and inadequate construction procedures.

There currently exists a multitude of wastewater management systems potentially applicable to rural lakefront communities with site conditions such as those found in Massachusetts. For example, the U. S. EPA has published several documents (1977b; 1977f; 1978; 1980b; 1982) that provide an overview of many on-site wastewater disposal systems. A preliminary review of these documents and many others rejected many of these systems from further consideration. The purpose of this report is to identify and evaluate a manageable set of alternatives appropriate for implementation at Massachusetts rural lakefront communities. The decision criteria used in this screening process included: (1) reliability of performance, (2) adequacy of treatment performance, (3) acceptability without requiring significant cultural or sociological change by the user, (4) suitability for implementation at some Massachusetts rural lakefront locations, (5) maintenance and operational requirements, and (6) a need for review. For example, systems relying on evapotranspiration appear unsuitable for regular use in Massachusetts because where evapotranspiration surfaces freeze, as would those in Massachusetts, their ability to function is doubtful (Beck, 1979). Further, impractical wastewater storage capabilities are required for systems relying on evapotranspiration alone where evapotranspiration does not exceed precipitation by two inches every month of the year (U. S. EPA, 1981a). The U. S. EPA (1980b) presents information indicating that in Massachusetts annual mean precipitation exceeds evapotranspiration by twenty inches annually.

Extreme water conservation systems, alternative toilets and the like were rejected for questions about performance, social acceptance and long term maintenance. In the future, progressive disposal systems such as these may be desirable. Today however, systems that remove and treat wastewater at reasonable cost with little attention required of the homeowner seem more favorable.

In short, in the authors' judgement, the only systems that can be considered for on-lot wastewater treatment are those that require practically no maintenance. Conversations with septage haulers and some literature (Eshwege, 1980; DeWalle, 1981; U. S. EPA, 1980f) reveal that practically no homeowners even pump their septic tank regularly, certainly not as often as the annual cleaning required by Massachusetts subsurface disposal regulations. Usually only when the tank is overloaded and sewage backs up into the home or surfaces outside the home is cleaning considered (DeWalle, 1981). Thus a large portion of this report evaluates and discusses only traditional septic tank-soil absorption systems and variations of this system. If collection of wastewater is feasible such that systems can be designed to serve clusters of homes, then formally delegated maintenance responsibilities become possible and "higher technology" systems become feasible. The last two chapters of this report look at alternatives for reducing the cost of small scale collection and treatment systems so that such cluster treatment schemes become feasible.

Another means of escaping from the "no maintenance" restriction on individual systems is to develop innovative operation and maintenance arrangements such as community responsibility. Such poolings of resources allow a professional to be hired to manage and maintain wastewater disposal systems, thereby allowing higher technology and higher maintenance systems to be used. Such operational and maintenance arrangements are the exception at present. Consideration of this approach to rural wastewater management was beyond the scope of this project.

Laboratory studies were not conducted as a part of this study. There currently exists a general excess of literature, much of it very good, reviewing on-site wastewater management systems. This provides, for most topics, a wealth of knowledge from which to draw upon. Evaluation of pertinent literature usually allows quite specific conclusions to be drawn. The large amount of literature also occasionally provides, as the reader might expect, conflicting viewpoints. In these situations, when both viewpoints can be scientifically justified, both viewpoints are presented. Generally however, small flow wastewater management systems are not "new technology" and the mechanisms governing small flow wastewater management system behavior are understood fairly well. Throughout this report, these mechanisms are presented, for it is the author's opinion that understanding these mechanisms is a necessary step towards rational evaluation of wastewater management systems. Where literature does not provide an adequate review of wastewater management topics, specific conclusions cannot be made. Generally, the limited knowledge is presented and weaknesses in the literature pointed out. Occasionally, suggestions for further research are made.

This report's next chapter, chapter two, discusses rural wastewater characteristics. The pattern of wastewater production and pollutant concentrations of rural domestic wastewater are different than wastewater characteristics of large municipal systems. These differences are significant to some wastewater management system designs. A description and, to a slight extent, evaluation of the validity of parameters used to describe wastewater is given in the appendix of this report.

Next, septic tanks, the most common on-site pretreatment process, are discussed. The reliability of many of the wastewater treatment or conveyance systems subsequently reviewed in this report depends heavily on the pretreatment performance provided by septic tanks. Septic tanks, properly designed and operated, remove solid material from and provide anaerobic degradation of wastewater. Alone, septic tanks do not provide adequate treatment to permit surface or subsurface discharge of wastewater. The many parameters affecting septic tank performance are reviewed so that a rational evaluation of septic tank design may be made. A septic tank design, suggested for incorporation into Massachusetts subsurface disposal regulations is presented. This septic tank, only slightly more difficult to construct than a conventional septic tank, provides better, more reliable treatment performance. More practical septic tank maintenance procedures are also suggested.

A discussion of soil absorption systems follows in chapter four. The physical, chemical and biological processes by which septic tank effluent is renovated within the soil are discussed. By understanding these processes and optimizing the conditions for their performance through design, improved disposal system performance can be achieved. Site conditions and soil properties necessary for adequate soil absorption system operation are reviewed. Inadequacies in current site evaluation techniques are reviewed and improved procedures, which better assess the ability of a site to accept septic tank effluent, are suggested. Modification of Massachusetts subsurface disposal regulations, to reflect the improved reliability and treatment performance resulting from these procedures, is recommended. Recommendations regarding construction techniques that reduce the probability of decreasing a site's permeability during the construction process are also presented. Methods to renovate failed absorption fields are reviewed. Finally, a design example, incorporating many of the suggested improvements is presented.

Where soils are unsuitable for absorption system use, either due to excessive or insufficient permeability, a modification of traditional soil absorption systems, the wastewater disposal mound, often presents a viable alternative. Design criteria for mounds has been adopted into many other states' subsurface disposal regulations; amendment of the Massachusetts subsurface disposal regulations to permit the use of mounds is recommended. Mounds provide an environmentally acceptable method of wastewater disposal, often at reasonable cost. Studies that evaluate mound design criteria and performance are reviewed within chapter four and a mound design, proven successful in other areas of the United States, is presented.

The chapters of Septic Tanks and On-Site Soil Absorption Systems describe technologies that, when properly designed, constructed and maintained, provide satisfactory renovation of wastewater. Unfortunately, lake shore developments are often plagued by inadequate on-lot disposal systems. Old developments often do not have any significant wastewater treatment system; newer systems are often improperly designed or located. A common result is excessive lake eutrophication due to phosphorus introduction from these disposal systems. Alternative phosphorus management systems such as phosphate detergent bans are discussed in chapter five. Particular attention is given to on-site phosphorus retention processes within the soil matrix. In some cases, installation of a new, properly designed, soil absorption system will sufficiently mitigate introduction of phosphorus to a waterbody from soil disposal systems.

Where on-site systems are not the answer, perhaps because proper site conditions do not exist and the cost to create suitable conditions is prohibitive, a more traditional treatment scheme, centralized collection and treatment, is a remaining alternative. A collection system can be designed to gather wastewater from homes along the lake perimeter (or clusters of homes) and discharge to a treatment system.

Sewage collection in traditional gravity flow pipelines is constrained by minimum velocity requirements, designed to keep solids suspended and prevent clogging of the pipeline. Deep excavation is often required to maintain minimum velocity requirements. To reduce the depth of construction, pumping stations may be constructed periodically along the flow path. These collection systems can become very complex and expensive construction projects. Along lakes, where shallow depth to ledge or groundwater are likely, construction costs of a traditional collection system become prohibitive. Environmental protection requirements along sensitive lakeshore areas may increase construction costs of these systems. Also, the natural topography of lakeshore regions works against traditional gravity flow collection systems. Most often, land around a lake slopes toward the waterbody, with houses located above and below a perimeter road. To collect sewage entirely by gravity flow, the sewer main can be placed either very deeply below the perimeter road surface, or much shallower along the lakeshore perimeter. While the shallower depth of main placement makes construction along the lakeshore attractive, it suffers from greater likelihood of high groundwater, shallow depth to ledge and environmental sensitivity. Thus lake water quality planners have often been faced with a difficult choice: Expensive, but adequate, wastewater treatment or continuation of inadequate, environmentally degrading disposal systems.

Alternative sewage collection systems are now available that may make collection systems to centralized or sub-regional treatment facilities economically feasible. Three such systems are evaluated and presented in chapter six: Pressure collection systems, vacuum collection systems and small diameter gravity sewers (including variable grade design). Each system is described and its design, construction, and maintenance reviewed. These alternative systems generally require more maintenance than traditional sewerage systems, but the move to collective rather than individual wastewater treatment makes this acceptable.

In the event of centralized collection, biological wastewater treatment schemes (a type of "higher technology" treatment) often become necessary, especially if suitable soils cannot be located near the wastewater generation region. Chapter seven of this report reviews the performance and types of biological wastewater treatment systems currently available for small flow applications. Small flow systems that provide biological wastewater treatment are commonly known as "package plants" for they are often prefabricated and delivered to a site ready to be hooked up to influent sewer, power supply, and effluent discharge. Two biological wastewater treatment processes employed in package plants, suspended growth and attached growth, are reviewed.

CHAPTER 2

Rural Wastewater Characteristics

The most suitable method of treating residential wastewater in a given instance depends on the treatment objectives, available resources and characteristics of the wastewater to be treated. Residential wastewater characteristics vary considerably. They depend most significantly on the lifestyle of the generator and to a lesser degree on diet, season, water pressure and plumbing fixtures. This section discusses parameters used to describe wastewater and suggests parameter values for design of small wastewater systems.

As part of a recent study (U. S. EPA, 1981a), a literature review of household wastewater characteristics was conducted. Each piece of literature was reviewed and weighted (based on type of study and amount of data) to develop a set of tables describing wastewater volumes and pollutant mass production. The average wastewater parameters developed by the 1981 study compare favorably with other literature not considered in their review (Ligman, Hutzler and Boyle, 1974; Siegrist, Witt and Boyle, 1976). Table 1 presents average mass pollutant production per capita-day and average household wastewater characteristics (based on their reported average total wastewater flow of 160 liters (43 gallons) per capita-day). Table 2 describes, based on a U. S. EPA report (1978), the added pollutant load home garbage grinders place on disposal systems.

In the appendix of this report, the reader will find a description, and evaluation of most of these wastewater parameters. Should greater detail be desired, the author suggests readers consult environmental engineering textbooks such as those written by Grady and Lim (1980), Metcalf and Eddy (1979), Clark, Viessman and Hammer (1977), reference manuals describing test procedures such as Standard Methods for the Examination of Water and Wastewater (American Public Health Association et al., 1981), or the journal articles and technical reports referenced by these sources.

The volume of wastewater produced is probably the most important wastewater characteristic to rural wastewater management for it often determines the size of conveyance or disposal systems. Rural wastewater generation is often estimated near 45 gallons per capita-day (Siegrist, 1976; Metcalf and Eddy, 1979; U. S. EPA, 1980b; U. S. EPA, 1981a). The effect of the standard of living of the generator on wastewater production is accounted for in estimating tables such as those found in Clark, Viessman and Hammer (1977; pg 127), developed for the Federal Housing Administration. These tables indicate that generators at

Table One

Average Rural Household Wastewater Characteristics

(U. S. EPA, 1981a)

Parameter	Pollutant Production (gm/cap-day)	Wastewater Concentration (mg/liter)
BOD_5	48	300
BOD_5 filtered	30	188
COD	120	750
TOC	32	200
TOC filtered	22	138
TS	125	780
TVS	70	440
SS	40	250
VSS	31	194
TKN	6	38
NH_3-N	2	13
NO_3-N	0.1	0.6
NO_2-N	---	---
Total P	4	25
PO_4-P	1.4	8.8
Oil and Grease	15	94
MBAS	3	19
Flow	160 lpcd 45 gpcd	

Table Two

Average Rural Household Wastewater Characteristics

Contribution Due to Use of Garbage Grinders

(U. S. EPA, 1978)

Parameter	Pollutant Production (gm/cap-day)	Wastewater Concentration (mg/liter)
BOD_5	11	1030
BOD_5 filtered	2.6	240
TOC	7.3	690
TOC filtered	3.9	370
TS	25.8	2430
TVS	24.0	2270
SS	15.8	1490
VSS	13.5	1270
TKN	0.6	60
NH_3-N	---	0.9
NO_3-N	---	---
Total P	0.13	12
PO_4-P	0.09	8
Flow	14.4 lpcd 3.8 gpcd	

locations of higher property value (i.e., standard of living) produce more wastewater.

Wastewater generation per capita typically increases during summer months. Seasonal wastewater generation fluctuations are attributed to more frequent bathing and increased human water consumption during warm weather. A lakefront community may as a whole have very large seasonal variations owing to its number of seasonal residents. Also, these seasonal residents may be from areas accustomed to greater wastewater generation.

Rural wastewater production varies diurnally and may vary within the week. Diurnal flow patterns are generally very similar to the potable water use profile of the generator, commonly showing peak water use rates during the morning and evening hours. Weekly flow variations in rural areas result from the residence pattern of that area. For example, wastewater production at recreational parks during summer weekends is often so much greater than the average daily flow that aerobic holding basins are constructed to dampen weekly variations (by releasing accumulated wastewater over several days) that might "flush out" a biological treatment system (CLOW Corporation, 1983). Design of any wastewater management system should consider wastewater production patterns.

Per capita pollutant mass loadings have also been studied. Residential pollutant mass loadings vary with diet and lifestyle. Several studies have analyzed wastewater production and characteristics by event (Ligman, Hutzler and Boyle, 1974; Siegrist, Witt and Boyle, 1976; U. S. EPA, 1978; U. S. EPA, 1981a). This information is important when designing wastewater disposal systems for non-residential sites such as schools, restaurants or factories. In these cases, the number of events per day would be estimated to determine wastewater composition.

This project concentrates on traditional gross wastewater parameters such as biochemical oxygen demand after five days of incubated digestion (BOD_5), suspended solids (SS), total nitrogen (N), and total phosphorus (P) concentrations. Wastewater treatment system performance can generally be evaluated in terms of their reduction of these parameter concentrations. More specific information is necessary for a complete evaluation of treatment system performance.

The next three chapters and chapter seven of this report describe wastewater treatment systems. All of these systems should provide, when properly designed, implemented, and maintained, adequate wastewater purification to meet the needs of Massachusett's rural lakefront communities. These systems do not "completely" renovate wastewater (for example, to drinking water quality) but do so sufficiently to protect public health and prevent significant environmental degradation.

CHAPTER 3

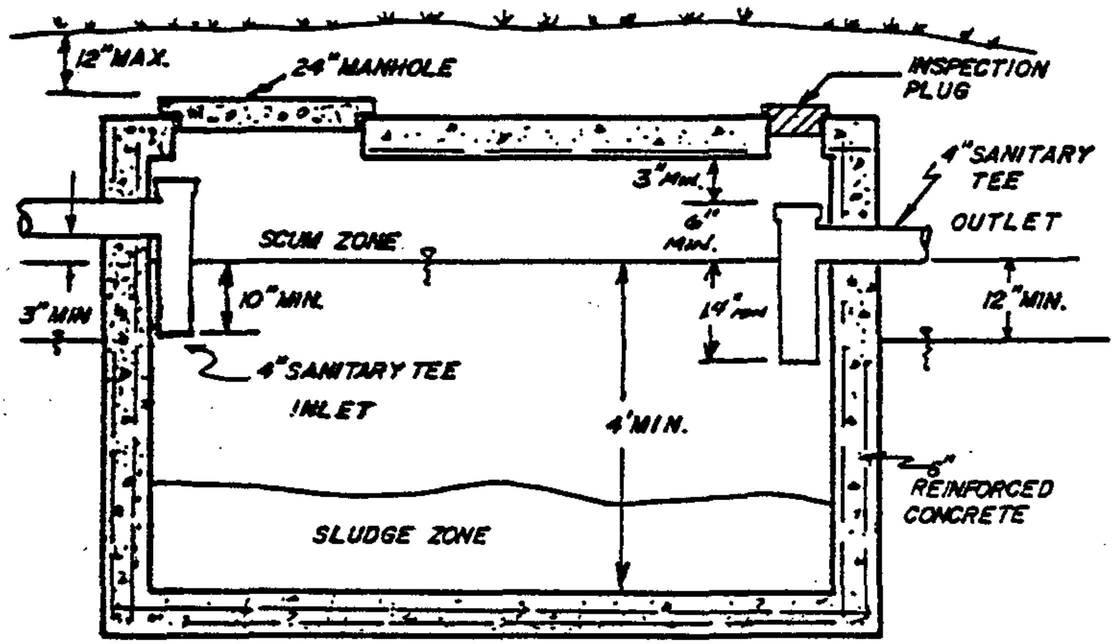
Septic Tanks

On-site wastewater management systems often require wastewater pretreatment to remove solid material, the presence of which may detract from subsequent treatment process performance. For many on-site systems, a septic tank serves this purpose. Septic tanks also provide flow equalization, retention of flotable materials, microbially mediated transformation of some chemical compounds (for example, transformation of organic and condensed phosphorus forms to orthophosphate forms) and an anaerobic environment for biological wastewater treatment.

Septic tanks operate entirely by gravity flow, they require no outside energy source. Although anaerobic digestion of organic material occurs in the tank, its primary purpose is sedimentation (Otis, 1982a). Septic tanks are large (usually 750 gallons or greater) rectangular boxes, normally placed below grade. They usually provide at least twenty four hours retention of sewage at average flow conditions. Approximately 25 percent of United States homes use septic tanks or cesspools for disposal of their domestic wastewater (U. S. Dept. of Commerce, 1980; U. S. EPA, 1980b). Septic tanks are used to pretreat residential wastewater before conveyance in small diameter gravity sewer systems and some pressure sewer systems. They commonly precede disposal to soil absorption or filtration systems. Figure 1 shows a septic tank conforming to current Massachusetts subsurface disposal requirements (Commonwealth of Mass, 1980).

Organic material stored in the septic tank undergoes anaerobic digestion, reducing organic molecules to soluble compounds and gases such as H_2, CO_2, NH_3, H_2S and CH_4 (Otis, 1982a). Digestion can reduce accumulated sludge volume by up to forty percent (Otis, 1982a). Gases that bubble up from the sludge layer as a result of digestion may disturb and resuspend nearby solids, decreasing septic tank performance. Outlet structures should be baffled to deflect away rising gases and their associated suspended solids. Venting of gases is important to remove toxic, noxious and explosive gases (Otis, 1982a).

Septic tanks significantly reduce wastewater biochemical oxygen demand (BOD) and suspended solids (SS) but not sufficiently to meet most point source surface discharge requirements, even if effluent disinfection is practiced. The U. S. EPA (1978) reviewed five studies and evaluated seven sites to report several septic tank effluent characteristics. Effluent BOD_5 concentrations ranged from 93 to 240 mg/1 (most reports near 140 mg/1). Suspended solids effluent concentrations ranged from 39 to 155



SECTIONAL VIEW

FIG. I : SEPTIC TANK (CONFORMS TO MASS. REGULATIONS) mg/l (most reports under 100 mg/l). Data presented in a U. S. EPA study (1978) indicates that a 1,000 gallon single compartment septic tank, receiving a wastewater loading characteristic of a 4 person rural residence, will average 25 percent BOD_5 and 82 percent SS removal. Poorer BOD_5 and SS removals occurred in smaller tanks receiving similar loadings. Table 3 summarizes septic tank effluent characteristics.

Septic tanks, as well as removing solid material, also alter the characteristics of solid materials present in wastewater (Ludwig, 1978). The nature of the solids in septic tank effluent are markedly changed from influent solids. Ludwig (1950, 1978) describes raw sewage solids as being of a "gummy gelatinous" nature, while those in septic tank effluent are discrete and non-gelatinous. Hence, solids in septic tank effluent are less likely to cause clogging of subsequent conveyance or treatment systems than raw sewage solids.

Nitrogen and phosphorus removals were not consistently reported in the literature, but generally, poor removals of these nutrients occur in the septic tank. Nitrogen is removed by storage in the sludge zone. Laak (1980a) estimates 20 percent total nitrogen removal. The predominant form of nitrogen in septic tank effluent is ammonia (U. S. EPA, 1978). Denitrification of any nitrates in the septic tank would be expected. However, since the septic tank is commonly the first component in a treatment system, nitrification of the wastewater (forming nitrates) has probably not occurred and therefore, denitrification cannot occur.

Phosphorus is also partially removed by accumulation in the sludge zone. Laak (1980a) reports 30 percent and the U. S. EPA (1980b) estimates 15 percent total phosphorus removal by sludge accumulation. The predominant form of phosphorus in septic tank effluent is orthophosphate (U. S. EPA, 1978).

Septic tanks do not significantly decrease microorganism concentrations of wastewater. They also cannot be relied on to remove pathogenic microorganisms from the waste stream (U. S. EPA, 1980b).

Septic tank effluent usually discharges to soil absorption fields where physical, chemical and biological processes (hopefully) renovate the wastewater as it percolates downward. The presence of excessive solids or grease in septic tank effluent will clog the distribution piping or soil absorption field. Such clogging will likely lead to hydraulic failure of the treatment system. Clogging of the soil absorption field may also result from organic overloading. When organic wastes are discharged to soil, a bacterial mat develops which restricts the percolation of wastewater. If an excessive bacterial mat develops, soil

Table Three

Characteristics of Septic Tank Effluent

Single Compartment Tank Receiving Residential Wastewater:

Based on: U. S. EPA (1978); Field and laboratory analysis of variously loaded and sized septic tanks.

Parameter	Average Concentration (mg/liter)	95 Percent Confidence Interval (mg/liter)
BOD_5	138	129-147
SS	49	44-54
Total P	13	12-14
Total N	45	41-49
Fecal Coliform*	6.7	6.4-7.0
Fecal Strep.*	4.6	3.9-5.3

* Log_{10} #/liter

Two Compartment Septic Tank Receiving Residential Wastewater:

Based on: Laak (1980b)

BOD_5	101	mg/liter
SS	40	mg/liter

absorption field clogging occurs. An improperly designed or operating septic tank may not sufficiently remove solids and grease or degrade the carbonaceous components of wastewater, contributing to absorption field failure. Increasing the efficiency of the septic tank is often the most cost effective method to decrease the probability of excessive clogging (Laak, 1980b) and hence, treatment system failure. Sufficiently increasing septic tank performance in some cases could eliminate the need to replace or expand a failed soil absorption field (Laak, 1980b).

The presence of inlet or outlet baffles improves septic tank performance. An inlet baffle dissipates energy of the influent wastewater, reducing turbulence and sludge upset in the septic tank. An exit baffle will deflect away from the discharge piping, many of the solids suspended by gas bubbles rising from the sludge zone (due to anaerobic digestion processes within this zone). Both inlet and outlet baffles may help prevent short circuiting in the tank. Septic tanks ideally should have baffles at the entrance and exit of each compartment.

The construction of inlet and outlet structures is important to prevent floating scum from entering (and potentially clogging) inlet or effluent piping. By extending their length below and venting them above the scum zone, this carry over can be prevented.

Upflow velocity of fluid is usually the critical parameter in sedimentation basin performance and as such, improvements in septic tank performance can generally be achieved by increasing septic tank surface area. For equal volumes of septic tank, shallow tanks are preferred (Otis, 1982a). Shallow tanks have larger surface areas, resulting in improved settling of suspended solids and better dampening of hydraulic surges (Otis, 1982a). Laak (1980b) also suggests maximizing septic tank surface area and describes this geometry by a surface area to depth ratio (surface area in square feet and depth in feet). Ratios greater than two are suggested for each compartment in multi-compartment tanks (Laak, 1980b). Sufficient depth should be present however, to provide for solids and grease accumulation and prevent turbulent flows from disturbing these stored materials. Otis (1982a) recommends that septic tanks be greater than three feet but no more than six to seven feet from effluent invert to bottom of tank.

Septic tank performance is also improved by compartmentalization. When a tank is properly divided, improved BOD and SS removal occur (U. S. EPA, 1980b). Laak (1980a,b) recommends the use of two compartment septic tanks. Reviewing work by others and himself, Laak (1980b) indicates that two compartment tanks perform better than single or triple compartment tanks of equal volume. Improved performance over single compartment tanks is attributed to preventing solids carry over to the effluent piping. Poorer performance of triple (and greater number) compartment tanks can perhaps be attributed to decreasing compartment quiescence as the the number of compartments in a constant volume and area system increase. Laak (1980b) estimates two compartment tanks have 50 percent better BOD and SS removal than single compartment tanks. He points out (1980b) that even small improvements in SS removal (for example, from 75 to 80 percent removal) can significantly reduce the suspended solids load (20 percent reduction in this example) to subsequent treatment units, perhaps significantly increasing their useful life. The U. S. EPA (1980b) also recommends two compartment tanks, attributing improved performance to hydraulic isolation and reduced mixing within the tank. The second compartment receives wastewater at a lower hydraulic rate and with less turbulence than the first compartment (due to flow equalization provided by the first compartment), increasing the removal of low density solids (U. S. EPA, 1980b). Wastewater treatment or conveyance systems employing two compartment tanks may not fail as rapidly during heavy hydraulic or organic loading periods as those systems employing single compartment tanks. Multi-compartment tanks provide better protection against solids carry over into effluent piping during periods of surge flows or upset due to rapid digestion (Laak, 1980b; U. S. EPA, 1980b).

Laak (1980b) suggests, based on U. S. Public Health Service experiments (Weibel, Straun and Homan, 1949), that compartment interconnections in a multi-compartment septic tank should be inverted, vented U-fittings rather than horizontal slots cut in the compartment barrier. Otis (1982a) recommends interconnections be an open four inch port, elbow, or sanitary tee located below the scum level rather than a slot so that hydraulic oscillation between compartments is reduced. Effluent and inlet baffles will improve performance by reducing solids carry over and turbulence in subsequent compartments. Figure 2 shows a two compartment septic tank schematic, with interconnections that should prevent the carry over of grease and solids, suitable for for one family residences.

The U. S. Public Health Service (U. S. Dept. of Health, Education and Welfare, 1967), U. S. EPA (1980b) and Laak (1980b) recommend that the first compartment (where most sludge accumulation occurs) be 200 to 300 percent larger than the second compartment in a two compartment tank.

Increased retention of wastewater in a septic tank improves treatment efficiency (Laak, 1980b). Generally, a minimum detention period of 24 hours at average flow is recommended. Local and State regulations of septic tank design usually mandate a minimum tank volume based upon the estimated daily flow the tank will receive (often estimated from the number of bedrooms in a residence). Providing tank volume in excess of the minimum requirement will likely result in improved tank performance and decrease the required frequency of tank cleaning (Laak, 1980a). When designing a septic tank, approximately two-thirds of the tank volume should be reserved for the accumulation of grease and solids.

Septic tanks may provide substantial flow equalization (Otis, 1982a). The hydraulic pattern of septic tank effluent is a function of tank surface area and inlet/outlet configuration (U. S. EPA, 1978). As the surface area of the tank increases, flow equalization improves (Otis, 1982a). A 1000 gallon, single compartment septic tank tested at the University of Wisconsin reduced peak flows from three gallons per capita per hour (gpcph) influent to one gpcph effluent (U. S. EPA, 1978). Multiple compartment tanks will likely provide better flow equalization than single compartment tanks.

Septic tanks should be placed at least twelve inches below grade to prevent freezing in winter climates (Otis, 1982a).

Manholes must be provided over each septic tank compartment to facilitate cleaning. The U. S. EPA (1980b) recommends that smaller inspection ports be installed over each compartment to allow inspection without manhole cover removal. If the manhole cover is constructed to grade, a secure seal should be provided to prevent accidental entry or the escape of offensive gases (U. S. EPA, 1980b). When the manhole cover remains below grade, a record of its exact location should be kept with the home so that locating it for cleaning or inspection is easy.

Figure 2 shows a septic tank design, incorporating the design features just reviewed to optimize its performance. This particular septic tank is suitable for a three bedroom residence, but could easily be modified to serve other flows.

Operation of septic tanks is simple, but wastewater generators should exercise care to prevent materials that are not easily degraded (coffee grounds, cooking fats, bones, diapers, feminine hygiene products; Otis, 1982a) from entering the system. Ordinary amounts of bleach and detergents from washing should not harm system efficiency (U. S. EPA, 1980b). Similarly, brine waste from home water softening equipment, in normal quantities will not significantly detract from septic tank performance (U. S. EPA, 1978). Regarding septic tank start up, it is not necessary to add anything but wastewater to the septic tank (Otis, 1982a). The addition of enzymes or chemicals designed to improve septic tank performance have generally not been proven beneficial (and occasionally, proven detrimental) to tank performance (U. S. EPA, 1978). Chemical additions are generally not recommended (U. S. EPA, 1978).

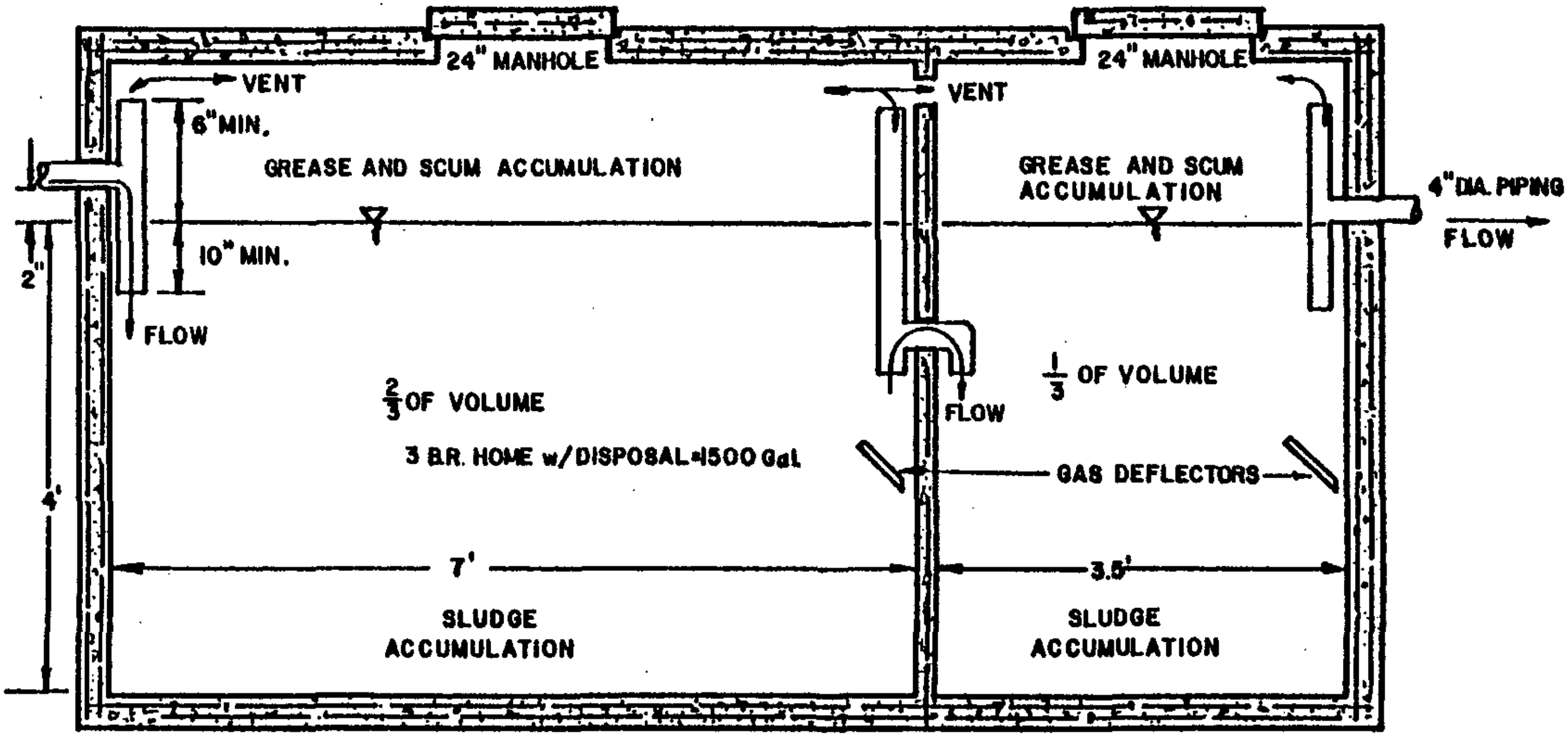


FIGURE 2: IMPROVED SEPTIC TANK DESIGN

SECTIONAL VIEW

NO SCALE

Sludge, wastewater and scum removed from septic tanks when cleaned is referred to as septage. Septage haulers may discharge their waste to land application sites, lagoons or wastewater treatment facilities. Generally, special handling facilities at treatment facilities are required to handle septage.

The frequency of septic tank cleaning (removal of septage) required depends on the rate of septage generation for that wastewater system and the size of the septic tank. For most residential homes, every three years appears to be sufficient. The U. S. EPA (1980f) reviewed Massachusetts and Florida studies relevant to this topic. Residential septic tanks in Wayland, Massachusetts, were cleaned, on average, every 3.2 years. Commercial, institutional and industrial systems were pumped annually. Florida residential systems serving a few elderly residents required pumping only once every 25 years. Tollefson and Kelly (1983) investigated required septic tank cleaning frequency of a sample of 350 homes in Manila, California. There, the average required septic tank cleanout frequency was 10.1 years. This frequency ranged from 2.4 to 37.5 years (Tollefson and Kelly, 1983). The U. S. EPA (1978) states that "generally it is good practice to pump the tank once every three years, depending on use." Otis (1982a) suggests an annual inspection of the septic tank, measuring sludge and scum depth to insure that they do not enter the discharge piping. He estimates a required cleaning frequency of two to five years, "depending on household habits" (Otis, 1982a). Large flow systems should be cleaned annually (Otis, 1982a). The U. S. EPA (1980b) suggests that inspections occur at least every two years, presumably cleaning as required, and that cleaning occur every three to five years if inspection programs are not carried out. The tank should be cleaned at least when the scum layer is within three inches of the bottom of the outlet device or the sludge level is within eight inches of the outlet device (U. S. EPA, 1980b).

Septage generation varies widely. It is a function of household habits and septic tank efficiency. Laak (1980a) indicates that accumulation of 60 to 85 gallons of septage per capita-year can be expected. Tollefson and Kelly (1983) report, based on a sample of Manila, California, residences, an average septage accumulation rate of 3.5 cubic feet per capita-year (26 gallons) but also indicate that septage generation varied widely.

When the septic tank has been pumped out, inspection of joints and walls for leaks or cracks may be made. Entering a septic tank is discouraged. When it is necessary to enter a septic tank, precautions against inhaling toxic gases that will be present must be made (U. S. EPA, 1980b; Otis, 1982a). Flotation of the tank (and subsequent structural damage) is possible after pumping the tank where high groundwater conditions exist. During construction, anchors can be placed to prevent this movement. It may be possible to simply delay the pumping where the high groundwater is seasonal.

It is not necessary to leave a quantity of septage in the tank to "seed" the tank after pumping (U. S. EPA, 1980b; Otis, 1982a). However, cleaning of the walls with detergents, chemicals or by scrubbing is of no aid to tank performance either; its practice is discouraged (U. S. EPA, 1980b; Otis, 1982a). Detergents and chemicals used for cleaning may cause sludge bulking and decrease sludge digestion (U. S. EPA, 1980b).

Massachusetts currently requires that the effective liquid volume of septic tanks be 150 percent of daily design flow or 200 percent of design flow where garbage grinders are installed. In each case, a minimum size of 1000 and 1500 gallons, respectively, is mandated. Septic tanks may not be installed where the seasonal high groundwater elevation is within one foot of the effluent invert. They also are required to be cleaned and inspected annually (Commonwealth of Mass, 1977).

CHAPTER 4

On-Site Soil Absorption of Septic Tank Effluent

A. Soil Absorption Systems

Disposal of residential wastewater is often to subsurface soil systems. Originally, pit privies were used for waste disposal. As rural electrification brought power to farms and isolated areas however, the use of indoor plumbing and pressurized water systems became commonplace (U. S. EPA, 1978). This resulted in increased quantities of wastewater and problems associated with its disposal. Since that time, on-site wastewater disposal systems such as the septic tank - soil absorption system have developed (U. S. EPA, 1978). Figure 3 shows a septic tank - soil absorption system schematic. Today, where suitable soils exist, septic tank - soil absorption systems are often considered the most reliable and least costly method of on-site wastewater management (Otis, 1982c). Approximately 25 percent of residential homes in the United States dispose of their wastewater to soil systems (U. S. Dept of Commerce, 1980). In Massachusetts, there are approximately 500,000 housing units (27 percent) disposing of waste to septic tank - soil absorption (ST-SA) systems (Veneman, 1982).

There are several soil absorption configurations currently in use. In most of these, a distribution pipe introduces septic tank effluent to a gravel (or similar) material. Flow through the gravel material distributes the effluent over a greater area. Storage of septic tank effluent is provided in the gravel pore spaces before absorption into the soil matrix. The distribution piping and gravel are most commonly constructed in trenches (see Figure 3) or beds but may also be placed as a pit, mound, fill, or artificially drained system (U. S. EPA, 1980a). (Mounds are described in detail later in this chapter.) The best configuration in any instance depends on site characteristics. Construction is often easiest and least expensive in a trench configuration. Another advantage of trenches is that their sidewalls act as infiltrative surfaces, decreasing the required size of the distribution network. A bed system is much wider than a trench system for it often has several distribution pipes. The bed bottom is its principal infiltrative surface (U. S. EPA, 1980b), usually necessitating greater excavation and distribution network requirements than a trench system.

Current ST-SA System Performance

Unfortunately, during the past several decades, septic tank - soil absorption systems have often been misapplied, resulting in high failure rates (Kriessl, 1982). Soils suitable to accept septic tank effluent are not always available. The U. S. EPA (1980b) estimates that only 32 percent of the total United States land area meets the traditional site criteria outlined in the 1967 Manual of Septic-Tank Practice (U. S. Dept. of Health, Education

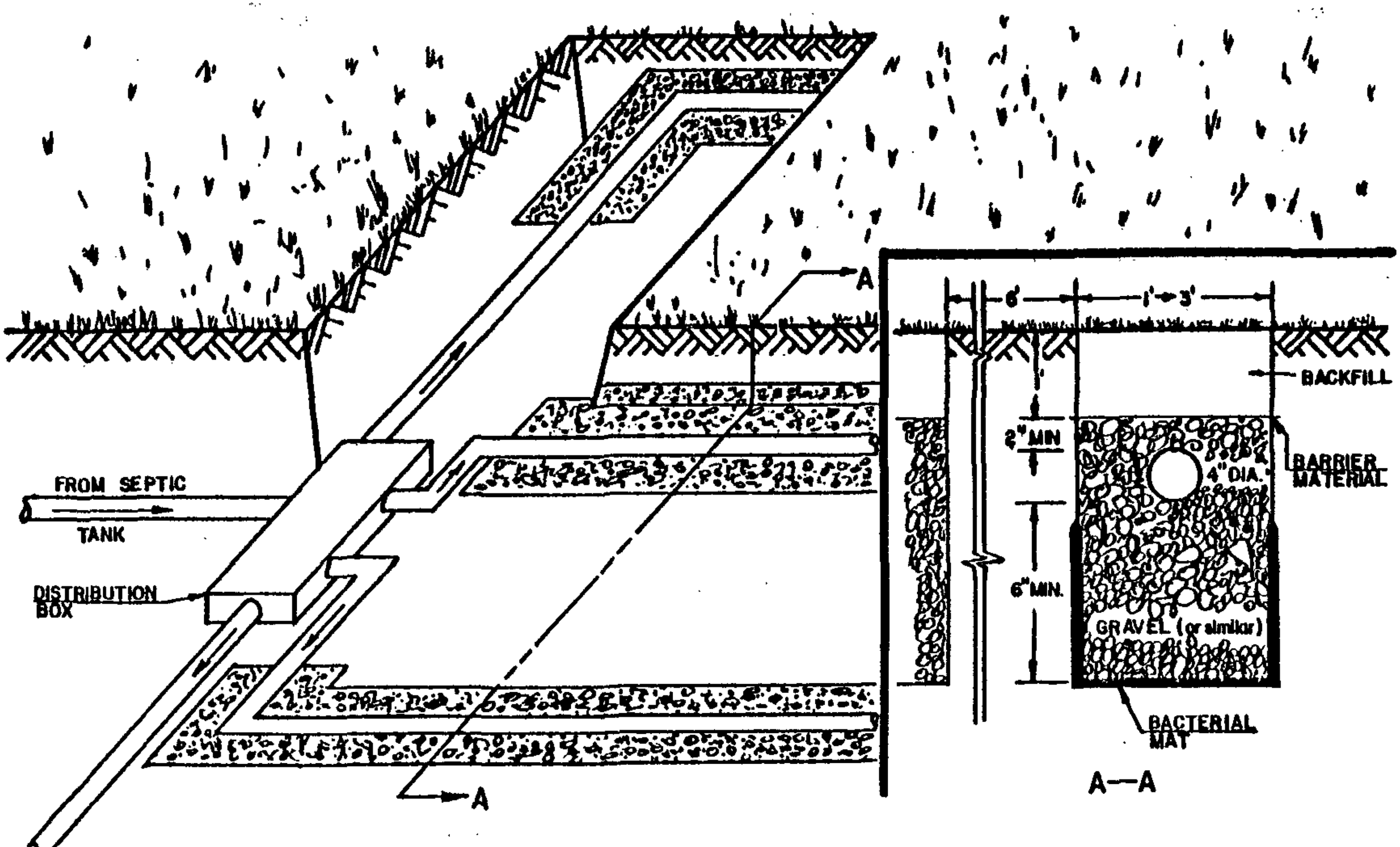


FIGURE 3: SOIL ABSORPTION SYSTEM
(TRENCH CONFIGURATION) NO SCALE

and Welfare, 1967). The soil hydraulic characteristics and depth to groundwater or impermeable layer are site properties that affect its ability to accept and renovate wastewater.

Even where suitable soils exist, methods suggested to assess that soil's ability to accept and renovate septic tank effluent are grossly inadequate. For example, soil structure, which, as discussed later, is paramount to that soils ability to support the microbial community necessary for wastewater renovation, is not addressed by existing Massachusetts subsurface disposal regulations. (Later in this chapter, existing site evaluation procedures are evaluated and improved procedures suggested.)

The Manual of Septic-Tank Practice (U. S. Dept. of Health, Education and Welfare, 1967) attempted to disseminate design criteria to public health officials and designers of on-site wastewater management systems. As these criteria became adopted into disposal regulations, reliability of systems improved. Saxton and Zeneski (1979) report on improved performance of ST-SA systems in Acton, Massachusetts after more stringent design and installation requirements were adopted in 1971. Hill and Frink (1980) also report on improved absorption system longevity after more thorough soil testing requirements and stringent design criteria were adopted in Glastonbury, Connecticut.

The number of properly performing ST-SA systems is difficult to accurately assess. A staff written article in Water and Sewage Works magazine estimates that less than 80 percent of these systems are performing properly (Water and Sewage Works, 1979). Veneman (1982) simply states that a large number of Massachusetts ST-SA systems do not operate properly.

Failure of ST-SA systems can be defined both hydraulically and by pollutant concentration reduction (treatment performance). Slonecker (1982) suggests that hydraulic failure can be evidenced by upward and lateral movement of septic tank effluent towards the ground surface. Surface discharge of septic tank effluent may create a public health hazard, and is often malodorous and unaesthetic. Treatment performance failure definitions include criteria such as organic, microbiological and nutrient removals. Poor treatment performance by subsurface systems has caused outbreaks of waterborne communicable diseases such as infectious hepatitis (Hepatitis A; Water and Sewage Works, 1979).

Septic tank - soil absorption systems have failed for a variety of reasons, often stemming from improper design and construction. Improper design may be due in part to difficulty in assessing the ability of a site to accept septic tank effluent. More specifically, high groundwater, shallow bedrock, inadequate soil permeability and inadequate sizing of the absorption system have been attributed to soil absorption system failure (Eshwege, 1980; Veneman, 1982). Other factors contributing to failure may be poor construction procedures, inadequate inspection procedures during construction by regulatory agencies, failure to follow design guidelines, improper system operation and maintenance (Eswege, 1980), and improper assessment of wastewater characteristics.

Septic tank - soil absorption system failure is often considered a function of time. Some believe that all ST-SA systems will fail eventually (Laak, Healy and Hardisty, 1974). Laak (1980a) however, states that properly designed, constructed and operated, ST-SA systems should function forever. He bases this on a concept of a long term acceptance rate (LTAR) of septic tank effluent to a soil. This concept is discussed later in this chapter. There is some controversy about this theory (Kristiansen, 1982), but in most soils, the half life of properly designed systems is more than 35 years (Hill and Frink, 1980). Several studies have attempted to predict ST-SA failure by statistically reviewing the installation and failure history of these systems within a town or region (Saxton and Zeneski, 1979; Hill and Frink, 1980; Dewalle, 1981). These studies report "survival curves" that generally show the greatest number of failures in the first few years. Slonecker (1982) attempts to predict ST-SA system failure by the use of aerial photography, searching for vegetative indications of improperly operating systems.

It is most important that the soil system be hydraulically sound (Laak, 1980a). Failure of a soil system to accept a quantity of wastewater results in either surface discharge of untreated septic tank effluent or backup of sewage into the home.

Surface discharge of septic tank effluent (hydraulic failure) usually indicates soil absorption field clogging. Clogging may result from: (1) compaction or smearing of soil surfaces during construction, (2) an improperly designed or operating septic tank not sufficiently removing solids, (3) excessive bacterial growth in the absorption field, (4) deterioration of the soil structure caused by ion exchange on clay particles, and (5) precipitation of insoluble metal sulfides during anaerobic conditions (Bishop and Logsdon, 1981). Laak (1970) found that insoluble metal sulfides are not present in sufficient quantity to be considered a significant component in absorption field clogging. Most commonly, improper construction, excessive bacterial growth and excessive solids loading are the causes of soil clogging (Bishop and Logsdon, 1981). Excessive bacterial growth may result from high concentrations of organic matter, a substrate for bacterial growth, in septic tank effluent. As a bacterial layer develops, slimy polysaccarides are excreted which further impede wastewater percolation. Excessive growth may prevent adequate soil absorption of septic tank effluent, causing hydraulic failure. Excessive solids in the septic tank effluent may clog pore spaces in the soil matrix, also reducing wastewater absorption.

Where rapidly permeable soils exist, percolation of septic tank effluent may occur so rapidly that little waste degradation is achieved. For example, a septic leachate detector system (*septic snooper*) was employed to detect septic leachate plumes along Lake Lashaway, located in North and East Brookfield, Massachusetts (Interdisciplinary Environmental Planning Company (IEP), 1980). Of approximately 200 cottages along or near the Lake Lashaway shoreline (Hardy, 1977), 49 leachate plumes were detected (IEP, 1980). At more than 10 locations, bacteriological investigation indicated that lake water exceeded Commonwealth Water Quality standards for fecal and coliform bacteria in class B waters (IEP, 1980). *Insufficient attenuation of septic tank* effluent in soil absorption systems is indicated, at least in part, as the cause of pollution in this instance (IEP, 1980; Noss, 1983). (Unfortunately, current Massachusetts subsurface disposal regulations ignore entirely the effect of rapidly permeable soils on treatment performance.)

The Clogging Mat

The clogging mat is a dark, slimy layer which forms at the infiltrative surface (DeVries, 1972; Kristiansen, 1982). The upper portion provides great hydraulic resistance and contains large amounts of organic material (Walker et al., 1973; Kristiansen, 1982). The lower portion contains metal sulfides (Kristiansen, 1982), of little hydraulic importance (Laak, 1980a). Kristiansen (1982) indicates that it is reasonable to assume that the makeup of the clogging material is mostly biodegradable accumulated suspended solids, bacterial cells and fragments of microorganisms. Polysaccarides and polyuronides, by-products of biological activity, are also found in the clogging layer and have been related to absorption field clogging (Kristiansen, 1982).

This clogging layer, the bacterial mat which reduces the transmittance of septic tank effluent to the soil, is most important in providing treatment of septic tank effluent. Similar to the operation of many wastewater treatment systems, bacteria present in the clogging layer, during replication and respiration, consume pollutants from the wastewater. This consumption purifies wastewater. The clogging layer also physically filters out solid material and microorganisms, further purifying septic tank *effluent*.

Bacterial replication increases the thickness or concentration of bacteria in the clogging layer. As the quantity of microorganisms increases beyond that needed to consume available substrates, the microorganisms begin to feed upon themselves, decreasing the thickness of the clogging layer. In a soil absorption system, the bacterial mat thickness varies from 0.5 to 5.0 centimeters, depending on the organic loading, solids loading and soil structure (Kristiansen, 1982). Organic and solids loading affect the amount of bacterial replication. Coarse soil structures, with their larger soil pore spaces, cannot structurally support a microbial biomass as well as finer structured soils. For this reason, the bacterial mat extends deeper into coarse soils. In extremely coarse soils, a homogeneous bacterial mat may not develop throughout the soil absorption system, allowing inadequately renovated septic tank effluent to percolate downward.

A suspected clogging mechanism is that previously suspended matter, accumulated in the clogging layer, is anaerobically degraded to polyuronides which aggregate soil and suspended solids particles (Kristiansen, 1982). Aggregation also occurs from bacterial excretion of a mass of polysaccarides and sugar molecules, sometimes referred to as a "glycocalyx" of fibers (Costerton, Geesey and Cheng, 1978). This glycocalyx may also serve as a food reservoir for bacteria (Costerton, Geesey and Cheng, 1978). As substrates become limited, microorganisms consume nutrients from the glycocalyx (Costerton, Geesey and Cheng, 1978). As the glycocalyx is degraded and microorganisms die due to substrate limitations, interparticle bonds break (Kristiansen, 1982), increasing the permeability of that region. It is theorized that as interparticle bonds are broken, remaining glycocalyx, polyuronides and smaller solids are flushed to deeper depths in the soil (Laak, 1980a). Here, due to pH shifts and endogenous respiration, organic and inorganic materials are dissolved and carried away (Laak, 1980a). In time, a sort of steady state of aggregation and separation of particles develops (Kristiansen, 1982). A buildup-breakthrough cycle of permeability, attributable to this clogging layer phenomenon, has been reported in several sources (Laak and Healy, 1977; Laak, 1980a; Kristiansen, 1982) and has led to the development of a long term acceptance rate (LTAR) concept (Laak, 1980a). The LTAR is the median hydraulic acceptance rate during the permeability changes, for a given hydraulic head. It is theorized, in short, that if septic tank effluent is applied to a soil at a rate less than its LTAR, failure of the absorption field will never occur.

Clogging layer permeability is affected by the performance of wastewater pretreatment processes (Laak, 1970). Based on information reported by Laak (1970), Laak, Healy and Hardisty (1974) propose a mathematical expression, useful for adjusting absorption field design area in all soils, depending on pretreatment unit effluent characteristics. The empirical expression is:

$$\text{Adjusted Area} = \{ \begin{matrix}\text{Septic Tank}\\ \text{Effluent Area}\end{matrix} \} \times [(BOD_5 + TSS)/250]^{1/3} \qquad (1)$$

where BOD_5 and TSS are expressed in mg/l. Methods for determining septic tank effluent area are presented later in this chapter, under the subheading "Design of Absorption Fields." The important point is that the permeability of the absorption system is a function of the applied fluid. Increased pretreatment of domestic wastewater reduces clogging at the infiltrative surface (Laak, 1970). It is important to system longevity to properly maintain pretreatment processes (such as septic tanks).

The clogging zone is a *highly reducing* environment and as such, only partial degradation of organic material can be expected (Kristiansen, 1982). Deeper below the crust however, unsaturated conditions, having higher redox conditions (aerobic) occur (Bouma, 1975; Smyth and Lowry, 1980; Kristiansen, 1982). Additional waste degradation will occur in this aerobic zone. Aerobic conditions are the result of greater permeability in the soil matrix (than the clogging layer), draining of fluid from large soil pores into smaller pores and aeration from the surrounding soil (Bouma, 1975; Smyth and Lowry, 1980; Kristiansen, 1982).

The effect of temperature on soil field clogging is not clear. As various information and conflicting conclusions are reported in the literature, further study is recommended (Kristiansen, 1982).

Unsaturated Soil Conditions

The hydraulic characteristics of unsaturated soil are very different than those of saturated soils. During saturated conditions, a large percentage of wastewater flows rapidly through larger soil pores (Smyth and Lowry, 1980). During unsaturated conditions, because of capillary action, water enters the smallest soil pores (which have the greatest capillary force; Otis, Bouma and Walker, 1974). Water moves into and through large pores only if the capacity of the smaller pores to conduct its movement is inadequate (Otis, Bouma and Walker, 1974). During unsaturated conditions, effluent moves through pores much more slowly than during saturated conditions and in a very irregular, tortuous path (Smyth and Lowry, 1980). Thus, unsaturated conditions increase the contact time between soil particles and septic tank effluent and presumably, improve wastewater purification through physical, chemical and biological mechanisms (Smyth and Lowry, 1980).

Bouma (1975) outlines acceptable hydraulic loading rates, designed to prevent hydraulic failure through the clogging zone and maintain unsaturated conditions below the bacterial mat, for a variety of soil types. For sandy soils, he suggests 5 cm/day (1.2 gal/sq. ft./day) maximum application rate. For silt loams and some silty clay loams, 5 cm/day dosed once daily, for sandy loams, 3 cm/day (0.72 gal/sq. ft./day); for silt loams and some silty clay loams he suggests 1 cm/day (0.25 gal/sq. ft./day).

Site Evaluation

Selection of a successful site for on-site wastewater disposal depends largely on soil quality at the chosen location, provided that proper design and construction procedures are followed (Veneman, 1982). A site that can support a biological mat, provide unsaturated conditions below the mat and not be prohibitively restrictive to transmittance of septic tank effluent is desirable. The ability of a soil system to accept and treat septic tank effluent is most often assessed by a percolation test. A percolation test is a type of falling head test, a measure of that soil's saturated permeability. In most communities, based upon the expected wastewater flow and the result of a percolation test, the soil absorption field is sized. Unfortunately, it is impossible to accurately correlate percolation rates to soil permeability (Laak, 1980a), flow through a biologically active soil treatment system and therefore, system performance.

A percolation test only measures the ability of a particular site to pass clear water. The percolation test was first devised in 1926 by Henry Ryon with the New York State Department of Public Works (Peterson, 1980; Laak, 1980a). With slight modification, it was endorsed by the U. S. Public Health Service in the 1967 Manual of Septic-Tank Practice (U. S. Dept. of Health, Education and Welfare, 1967) and has since become a national standard (Peterson, 1980). The procedure for performing a percolation test is outlined in the Manual of Septic-Tank Practice (U. S. Dept. of Health, Education and Welfare, 1967). In short, six separate test holes are dug where the absorption field is to be placed (Massachusetts subsurface disposal regulations require only one hole; Comm. of Mass., 1978). The bottom and sides of the holes are scratched with a knife to remove any smeared surfaces (of decreased permeability) and two inches of sand or gravel placed on the bottom of the hole (to protect the bottom surface while pouring test water into the hole). The soil is then "swollen" by keeping it in contact with water for four or more hours. Twenty four hours after the first water is added to the hole, the percolation rate, the rate that the water level drops inside the hole, is measured (U. S. Dept. of Health, Education and Welfare, 1967).

Peterson (1980) indicates that there may be quite variable results of percolation tests in similar soils, even when performed by professionals with previous percolation testing experience. Percolation test results in the same soil may vary by as much as 90 percent because of testing procedures, time of year of the test and interpretation of test results (Eshwege, 1980; U. S. EPA, 1980b). Percolation rates are significantly affected by: (1) depth to groundwater table or impermeable layer, (2) hydraulic head, (3) soil moisture, (4) shape and size of the test hole, (5) duration of the test, (6) capillary pressure, and (7) type of soil (Laak, 1980a). Sources of percolation test error are: (1) the use of power augers (which compact soil into the walls of the hole, reducing its permeability), (2) depth measuring errors, (3) improper accounting of the effects induced by the use of gravel backed perforated liners where percolation hole walls collapse, and (4) varying initial depth of water in the hole (Peterson, 1980).

Soil capillarity greatly influences water flow into soils (Healy and Laak, 1973). During a percolation test, this property may be responsible for a great deal of water absorption into the soil, especially if conducted during periods of low water table elevation and dry weather. Unfortunately, when an absorption field is operating near failure, its surrounding soil will be at or near saturation and of low capillarity (Healy and Laak, 1973). To reduce the influence of capillarity on percolation rate, the U. S. Public Health Service recommends that percolation test holes be saturated for at least 24 hours before the percolation rate is determined (U. S. Dept. of Health, Education and Welfare, 1967). Similarly, many local regulatory agencies require that percolation tests be performed during the spring. Hill and Frink (1980) attribute increased longevity of absorption systems in Glastonbury, Connecticut, in part to a spring testing requirement.

Soil absorption field size is most often empirically derived from percolation test results. The size is often based upon information supplied in the Manual of Septic-Tank Practice (U. S. Dept. of Health, Education and Welfare, 1967), which indicates, according to percolation test results, the square feet of absorption field required per household bedroom. Unfortunately, the relationship between soil percolation rate and absorption field performance has never been clearly established (Healy and Laak, 1973).

Researchers generally agree that the percolation test alone does not provide adequate information to properly design septic systems (Eschwege, 1980). Use of the percolation test assumes that the long-term ability of a soil to absorb septic tank effluent may be predicted by its short-term ability to conduct clear water (Peterson, 1980). The test cannot, as with any saturated permeability test, predict the rate of flow from a drainage field after a clogging layer (bacterial mat) develops (U. S. EPA, 1978). In spite of all its shortcomings, the percolation test can be a useful piece of information for soil absorption system design. Along with other information, the ability of a site to support a soil treatment process can be estimated (U. S. EPA, 1980b).

Such other information may include deep soil borings, useful for indicating the presence of impermeable layers, depth to groundwater, seasonal high groundwater (as indicated by soil mottling) and soil layering. Deep pit observation, to detect the presence of perched water tables, is suggested by Hill and Frink (1980). Description of site soils, especially texture, bulk density and structure, will also aid absorption system design (U. S. EPA, 1980b). Constructing soil tube samples and subjecting them to various loadings of septic tank effluent over an extended period could produce permeability data representative of conditions that might develop in that soil, but, for reasons of time and cost, seem generally impractical.

Other tests that, more reliably and consistently than the percolation test, measure saturated permeability have been described. Peterson (1980) describes a constant head apparatus that, by measuring the quantity of water removed from a reservoir, indicates saturated permeability. The State of California (1980) recommends a refined percolation test procedure, consisting of constant diameter and shape hole, a constant initial head and a float for more accurate head drop measurement. Healy and Laak (1973; 1974) describe and suggest the use of tube samples or a bailing pit (for use with high groundwater tables) for measuring saturated permeability. Neither test is significantly affected by capillarity and fairly good agreement between tube and pit permeability test results is reported (Healy and Laak, 1974). The tube sample test is rapid and simple (Healy and Laak, 1974). The pit permeability test is not as simple, it requires measuring groundwater flow into an excavated pit, but by measuring flow rate through a larger area of soil than a tube sample test, may be more accurate. Accuracy of the test is compromised somewhat by the depth required to perform the test. Soil permeability may gradually vary with depth. Veneman (1982) reviews, based on U. S. Soil Survey Staff Handbooks (U. S. Dept. of Agriculture, 1951; 1975), the applicability of Massachusetts soils for use in soil absorption systems. Ratings are based on several soil properties, including texture, structure, depth to groundwater or impermeable layer and slope. Management practices are suggested to overcome indicated limitations on any particular soil. The U. S. EPA (1980b) also stresses the importance of analyzing soil texture, structure and color (indicating drainage characteristics) in on-site wastewater disposal system design.

Design of Absorption Fields

Laak (1980a) recommends that a flow net analysis be used in the hydraulic design of subsurface absorption fields. The flow net analysis determines the hydraulic gradient of the absorption system to seasonal high groundwater. The hydraulic capacity of the site can be determined by assuming saturated conditions below the infiltrative surface and implementing the Darcy equation (which describes saturated flow through porous media; Freeze and Cherry, 1979). This procedure requires estimation of soil permeability (determined by field tests) and hydraulic head in addition to determining the hydraulic gradient. The design hydraulic loading rate for the absorption field must be less than this hydraulic capacity by a factor of safety. By knowing the expected daily wastewater quantity and the hydraulic capacity of the absorption site, the size of the absorption field can be determined.

Laak (1980a) then suggests that the absorption field also be sized based on an expression he presents empirically relating soil permeability to that soil's long term acceptance rate (flow through the clogging layer). The expression is:

$$\text{LTAR loading rate} = 5k - \{1.2/\text{Log } k\} \qquad (2)$$

Where k is permeability in ft/min and loading rate is in gallons/ ft^2 /day. It appears that, based on the source literature (Laak, 1980a), a reasonable safety factor has been incorporated into this expression. The absorption field is then sized based upon expected wastewater flow and the LTAR. The designer should choose the greater absorption system size of the two values, one based upon site hydraulics (flow net analysis) and the other based upon flow through the clogging layer (equation 2). Laak (1980a) indicates that within the permeability range found in most soils, the LTAR is somewhat insensitive. Therefore, permeability estimates more accurate than those determined by field tests are unnecessary for LTAR determination.

Smyth and Lowry (1980) suggest that absorption field area be sized according to phosphorus removal criteria (discussed later), indicating that adequate carbonaceous and microbiological waste purification will occur inherently.

The U. S. EPA (1980b) suggests that absorption systems be sized according to soil type and percolation rate. Suggested loadings vary from 5 cm/day for gravel and coarse sand to less than 1 cm/day for silty clay loams and clay loams having percolation rates from 61 to 120 minutes per inch.

Table 4 summarizes suggested hydraulic loading rates from several sources.

Distribution of Septic Tank Effluent

The use of pressurized distribution systems to evenly distribute septic tank effluent over the absorption field is encouraged by Otis, Bouma and Walker (1974). Pressure distribution systems can prevent localized overloading of absorption fields which could lead to inadequate wastewater purification. A small pump and piping network distribute septic tank effluent. The piping network and orifices must be carefully sized. Headlosses across the network should be great enough so that the network fills with septic tank effluent before much liquid is applied to the soil, ensuring essentially even distribution.

Laak (1980a) suggests that the gravel layer in a distribution system be sized to retain at least three days flow above the clogging mat so that peak flows may be attenuated.

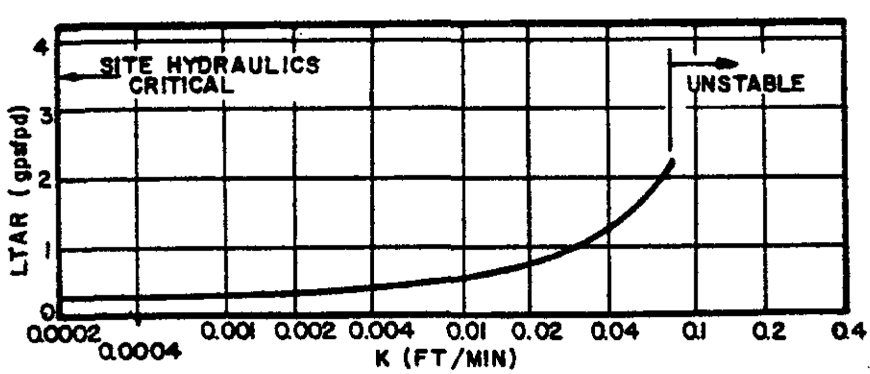
Construction Practices

The use of a "scraper-bucket" during construction has been recommended where smearing of absorptive surfaces (which may significantly decrease permeability through that region) is likely (Hansel and Machmeier, 1980). A scraper bucket is a conventional backhoe bucket modified by welding 1.5 inch long, 0.75 inch diameter rods, onto a removeable plate, spaced three inches on center. These protrusions will roughen trench sidewalls, preventing a smeared, impermeable surface from forming.

Table Four

Suggested Hydraulic Loading Rates for Sizing Soil Absorption Systems

Soil Type		Source
Rapidly Permeable:		
k greater than 0.02 ft/min:	Mound req'd.	(7)
PR greater than 0.1 min/inch:	Mound req'd.	(6)
Intermediate Permeability:		
Sands:	5 cm/day (1.2 gpsfpd)	(3)
Silt-Loams, some Silty-Clay Loams:	5.0 cm/day (1.2 gpsfpd)	(3)
Fine to Medium Sands:	3.4 cm/day (0.83 gpsfpd)	(1)
Sandy-Loams, Loams:	3.0 cm/day (0.74 gpsfpd)	(3)
Clay-Loams:	1.4 cm/day (0.33 gpsfpd)	(1)
Clays, some Clay-Loams:	0.6 cm/day (0.15 gpsfpd)	(3,8)
Low Permeability:		
PR less than 900 minutes/inch:	Build no system.	(5)
k less than 1×10^{-4} ft/min:	Hydraulic capacity of site governs size.	(2)
PR less than 120 minutes/inch:	Mound Required.	(6)
LTAR Graph: (vary loading with permeability)		(1,2,7)



Where gpsfpd = gallon/ft^2/day; k = permeability; and PR = percolation rate.

References:

(1) Healy and Laak, 1974
(2) Laak, Healy and Hardisty, 1974
(3) Bouma, 1975
(4) Kropf, Laak and Healy, 1977
(5) U. S. EPA, 1978
(6) Hansel and Machmeir, 1980
(7) Laak, 1980a
(8) U. S. EPA, 1980b
(9) Anderson, Machmeir and Hansel, 1982

(References used generally corroborate each other.)

Restricting traffic from the absorption field area, both before and after construction, is recommended to reduce soil compaction, which may decrease soil permeability (U. S. EPA, 1980b).

Absorption Field Rejuvenation

If a pretreatment unit fails and excessive solids are carried to the absorption field, hydraulic failure results and replacement or extension of the field may become necessary. Occasionally, failure will be the result of organic overloading. In this case, bacterial production is so great that permeability of the clogging layer is inadequate for the hydraulic loading. If the organic overloading is temporary, it is often advisable to dose the absorption field with hydrogen peroxide (Bishop and Logsdon, 1981; Andrews and Bishop, 1982). Hydrogen peroxide (H_2O_2), a strong oxidant, may oxidize materials clogging the soil (Andrews and Bishop, 1982). Oxidation of absorption field materials would best be achieved by introducing hydrogen peroxide to the system after the septic tank, perhaps to the distribution box. Within several hours, the absorptive capacity may be restored (Andrews and Bishop, 1982), but treatment performance will be decreased as the bacterial community is destroyed. Hydrogen peroxide dissociates to water and oxygen, innocuous end products (Bishop and Logsdon, 1981; Andrews and Bishop, 1982). The end products of oxidized clogging material are not adequately discussed to satisfactorily consider their environmental effects. However, a significant increase in absorption field effluent nitrate concentration is reported (Bishop and Logsdon, 1981). The increase is short term (Bishop and Logsdon, 1981) and should normalize after oxidation is complete and the bacterial community stabilizes. Short term environmental effects will be of site specific importance.

The suggested hydrogen peroxide dosage varies, depending on the extent of clogging, from 0.125 lb H_2O_2 / sq. ft. to 0.500 lb H_2O_2 / sq. ft. (Bishop and Logsdon, 1981). For an absorption field hydraulically sized for a family on five on mediocre soils (50 gpcd, 3 cm/day hydraulic loading rate) and this dosage range, the hydrogen peroxide material cost will be from 50 to 200 dollars (local delivery; based on telephone quotes: Astro Chemical, Springfield, MA and Hampden Color and Chemical, Springfield, MA; July, 1983). The cost of treatment is significant but certainly less expensive than absorption field replacement. In either case, the cost of failure should be sufficient impetus for the homeowner to maintain pretreatment facilities and exercise control over disposed materials.

A second method of absorption field rejuvenation is resting. One-year alternation of absorption beds has been suggested as a practical method of reducing biomass accumulation (Bouma, Converse and Magdoff, 1974; U. S. EPA, 1978; U. S. EPA, 1980b). Long-term resting desiccates the clogging mat, allowing aerobic decomposition. Such decomposition should increase permeability through that region. As this also decreases wastewater retention, it may be undesirable in rapidly permeable soils. Groundwater contamination due to insufficient treatment of septic tank effluent may occur as a result (U. S. EPA, 1980b). The cost of constructing a second absorption field may make one year alternation of beds undesirable, especially in light of the concept that properly sized, constructed and maintained, absorption fields should last forever (Laak, 1980a). It is probably more prudent and cost effective to conservatively design and build a single absorption field than to build two undersized alternating absorption fields. The U. S. EPA (1980b) suggests that since one-year resting may allow a greater hydraulic loading to an absorption field, the construction cost of such a system may be less than for a conventionally dosed system. This argument seems tenuous at best and unfortunately, no data is given to support their statement. One year alternation or resting of absorption beds seems unnecessary and impractical.

B. Design Example

This design example incorporates several ST-SA system design concepts discussed in this project report. Some new information is introduced here, in the form of design guidelines. This example is intended to demonstrate how a septic tank - soil absorption system can be designed based on a rational, engineering oriented, approach. The methodology used may seem at first somewhat lengthy and involved. However, with experience, the engineer would be able to design such a system very rapidly, probably at little additional cost over current design methods (and certainly providing a more sound and efficient system). We are intending to design a soil absorption system, utilizing a trench configuration, preceded by a two compartment septic tank.

For our example, we will assume that the Salomaki family desires to build a four bedroom, year-round residence overlooking Lake Pristine, a recreational resource and drinking water supply. There are no centralized sewerage facilities in the Lake Pristine region, therefore, an on-lot wastewater disposal system is necessary. We have been retained to design a system that will reliably purify and dispose of all wastewater generated at the Salomaki residence. We first decide, for the sake of example, to pay no attention to existing subsurface disposal regulations. Rather, our design will be based on engineering principles governing the implementation and successful operation of such a system.

Design Flow

It is desirable to first quantify the design flow (hydraulic loading). A maximum household population estimate of 2.5 capita per bedroom is reasonable. For this four bedroom house then, the maximum anticipated population is ten (10). From chapter two, we know that 45 gallons per capita-day is a good estimate of average wastewater generation. Multiplying, a maximum average flow of 450 gallons of sewage per day can be anticipated. A safety factor (multiplier) of 1.5 is appropriate for design of an on-lot disposal system, to prevent failure during peak flows. (The safety multiplier is yet reasonably small so that disposal system size does not become excessive. Recall that three days flow can be stored within the distribution network and that substantial flow equalization will be provided by the system itself. As with any engineering problem, the value of the safety factor should consider the cost of failure. In the case of an on-lot disposal system, failure would most likely not be catastrophic and would be preceded by warning signs such as dying vegetation or moist areas over soil absorption fields, allowing the owner an opportunity to reduce wastewater generation. A safety factor greater than 1.5, and at the most, 2.0, is difficult to justify.) The design flow then is:

$$4 \text{ br} \times 2.5 \text{ capita/br} \times 45 \text{ gal/capita-day} \times 1.5 = 675 \text{ gal/day} \quad (3)$$

Where br is the number of bedrooms and gal is gallons.

Next, an on-site investigation is conducted to determine if the site is hydraulically capable of disposing of this quantity of sewage. This investigation requires some excavation to determine hydro-geologic parameters. A general site schematic is shown in Figure D-1.

Site Description and Subsurface Investigation

The Salomaki property, in the region of the proposed on-lot treatment facility, slopes gently (2 to 5 percent grade) towards Lake Pristine (see Figure D-1). Because water elevations within drinking water wells along Lake Pristine exceed Lake Pristine's average water elevation, we suspect that groundwater, to some extent, feeds Lake Pristine. There are occasional ledge outcroppings near the site. Generally, the site is vegetated.

Deep holes are excavated at sites A, B, and C (see Figure D-1). Where possible, a depth of twelve feet below ground surface in the vicinity of the soil system is sufficiently deep to gather the information necessary for soil absorption system design. At the Salomaki property, excavation of only five to seven feet below ground surface was possible before refusal. Table D-1 presents a boring log of the subsurface investigation.

During deep hole excavation, the inconsistant nature of the depth to bedrock encourages the engineer to request further information about this parameter. Therefore, a dynamic sounding is performed at location D, providing information on the depth to bedrock only. Together with bedrock elevations at A, B, C and northwest of the site (exposed), we gather that the bedrock is sloping downward southeasterly. Further, because of apparent cleavages in the bedrock, it should be considered creviced - important in the later development of design criteria. We suspect

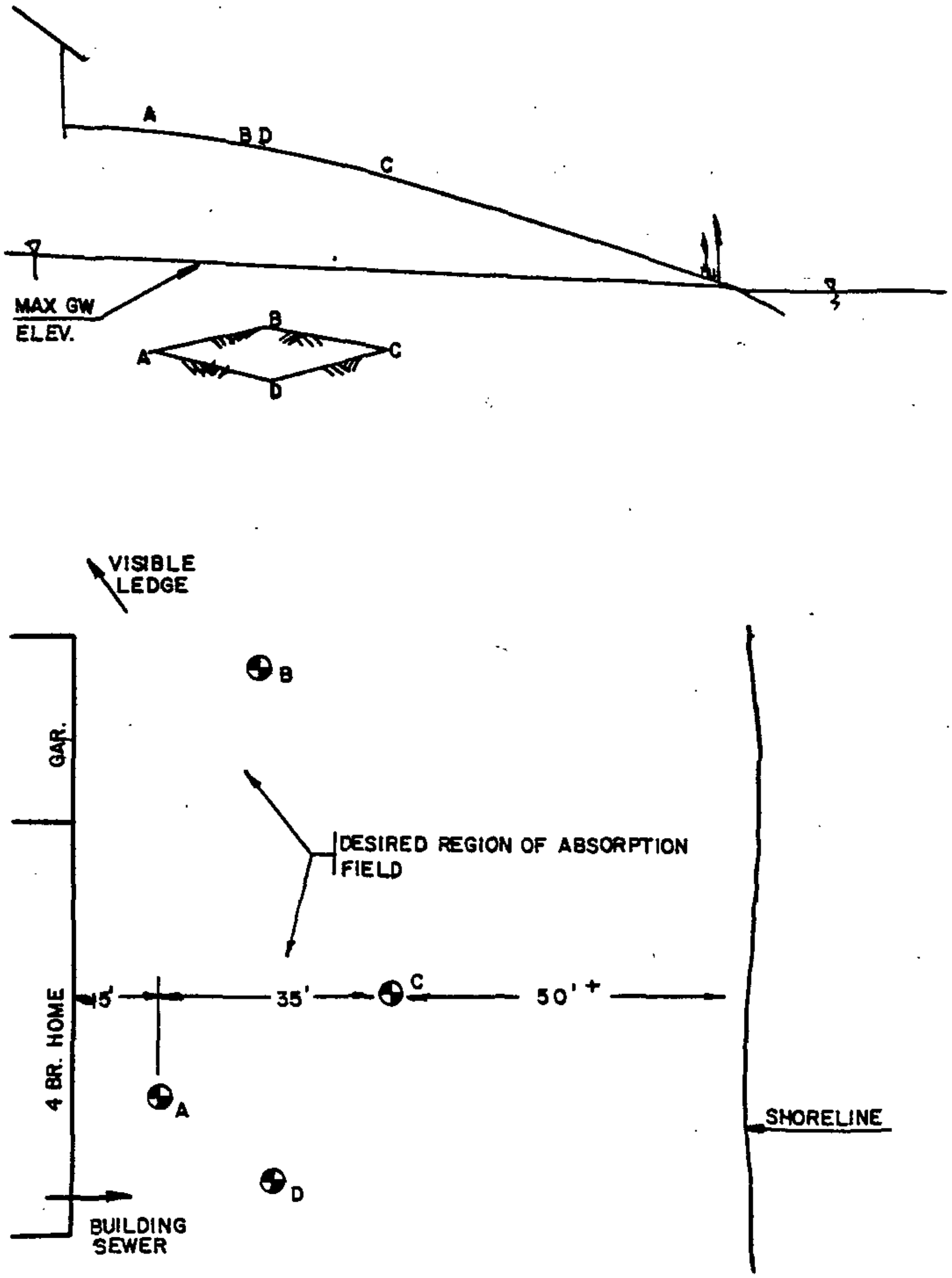


FIGURE D-1: GENERAL SITE SCHEMATIC
SECTIONAL VIEW – TOP
PLAN VIEW – BOTTOM

Table D-1

Soil Boring Log - Design Example

Elevations in Feet - Some local datum.
Date: Spring 1984. Subsequent to a long period of wet weather.

	Location			
	A	B	C	D*
Depth				
Surface:				
Description:	Turf vegetated with Scrub Pine and other small brush.			
Elevation:	104	103.5	103	103.5
A Horizon:				
Description:	Clayey-Loam, dark.			
Depth:	103	102.5	102.5	---
B Horizon:				
Description:	Brown, Sandy-Loam. Moisture approx. 3 to 5 percent.			
Max GW elev:	99.8	99.6	99.5	---
Description:	Continued Brown, Sandy-Loam, increasing moisture content.			
Refusal:				
Elevation:	96.7	98.0	96.7	96.0

* Dynamic Sounding only.

no unusual difficulty in installation or construction of the soil absorption field.

Hydraulic Analysis

For this evaluation we will assume several "worst case" conditions. Assuming saturated soil below the absorption trench, flow induced by capillary action is eliminated. Winter atmospheric conditions can be assumed, neglecting the effect of evapotranspiration on the water budget. We can minimize the available hydraulic gradient by assuming that the groundwater table is at its maximum elevation. Finally, for ease of analysis, we generally assume that site soils are homogeneous and isotropic (unless our site investigation indicated otherwise).

For our hydraulic analysis, it is important to measure the saturated soil permeability, k. A somewhat complicated (but fairly accurate) procedure is to remove an "undisturbed" soil sample and, using laboratory equipment, subject it to a head test. Field experiments that can estimate permeability are pit bailing tests and percolation tests. It is necessary to measure the soil/water interface area, change of head, quantity of water absorbed and length of time while performing these tests to determine k. Field tests are generally more desirable than laboratory experiments where non-homogeneous soils exist, because of their ability to measure fluid movement through a larger soil area. In the absence of field or laboratory tests, order of magnitude estimates can be made using U. S. Soil Conservation Service soil maps of the study area and/or the site description of the soil. Consulting reference material such as: Bouma, 1975; U. S. EPA, 1978; Sowers, 1979; and U. S. EPA, 1980b; permeability estimates can be made from the soil description.

The Connecticut Department of Environmental Protection has suggested a method to estimate permeability based on relating a change in water table elevation with an estimate of rainfall (Connecticut, 1983). The method is not suggested, for it relies on quantifying infiltration with the depth that the groundwater table has risen over an impermeable strata. In short, the methodology is too weak to support any permeability estimate. Other subsurface conditions could too easily affect the k estimate.

For the Salomaki property, we estimate saturated permeability using a pit bailing method (easy where shallow water tables exist) and a laboratory falling head test. The tests give reasonably close estimates of permeability and we conclude, therefore, that the brown, sandy-loam has permeability of approximately 80 cm/day (2.63 ft/day), an average to low value for a sandy-loam. For this example, we could assume that a high clay content, platey soil structure or fine texture exist in our sandy-loam - all characteristics that generally decrease soil permeability.

We must determine the available hydraulic gradient, i, to determine if our site can accept the estimated quantity of wastewater. There are two methods available to the engineer: (1) a flow net analysis and (2) an estimate based on groundwater elevation.

A flow net analysis, as suggested by Healy and Laak (1974), requires a scale drawing of the site subsurface conditions. It is important to know the depth to groundwater and impermeable strata as well as the location of any upstream or downstream impedences to flow. Healy and Laak (1974) suggest that in the absence of contradicting information, no effect on the groundwater table be assumed beyond 30 feet from the absorption trench. After construction of the flow net, the number of flow tubes divided by the number of equipotential drops derives the hydraulic gradient. A characteristic shape (mound) of saturated soil conditions below the absorption trench to the seasonal high groundwater table must be developed by the engineer. The effect of shortening the characteristic mound width is to increase the hydraulic gradient. A reasonably conservative design would use the maximum 30 foot width suggested by Healy and Laak (1974). The effect of overestimating the depth to impermeable strata is to overestimate the hydraulic gradient, certainly the engineer should utilize a depth no greater than the depth of subsurface investigation.

A hydraulic gradient estimate based on the existing gradient of the groundwater table is suggested by the Connecticut Department of Environmental Protection (Connecticut, 1983). The estimate may be useful where limited subsurface information is available or the designer chooses against flow net construction. In short, by knowing the difference in groundwater elevation at two test holes, a known distance apart, the hydraulic gradient can be estimated. Such calculations will very likely underestimate the hydraulic capacity of the site, particularly so if test holes are dug near the end of the dry season. Alternatively, the difference in seasonal groundwater elevations, as indicated by soil mottling, could be used (but would still underestimate i).

For the Salomaki property, a flow net is constructed (see Figure D-2). An absorption trench configuration must be assumed. The number of flow tubes is four (4) and the corresponding number of equipotential drops is thirty-four (34). Therefore, the hydraulic gradient (length/length - unitless), i, is $4/34 = 0.118$.

The next critical information is the area, A, through which wastewater will be introduced to the site. We have assumed a shallow absorption trench because of the shallow depth to groundwater (see Figure D-2), therefore, to provide storage capacity within the trench for three days flow we will assume a wide trench. Practically, 3.5 feet is the very maximum width that can be constructed with readily available construction equipment. (Some designers prefer to limit width to 3.0 feet.) An appropriate maximum trench length is 100 feet. Multiplying, the trench bottom area is 350 square feet. (For hydraulic analyses

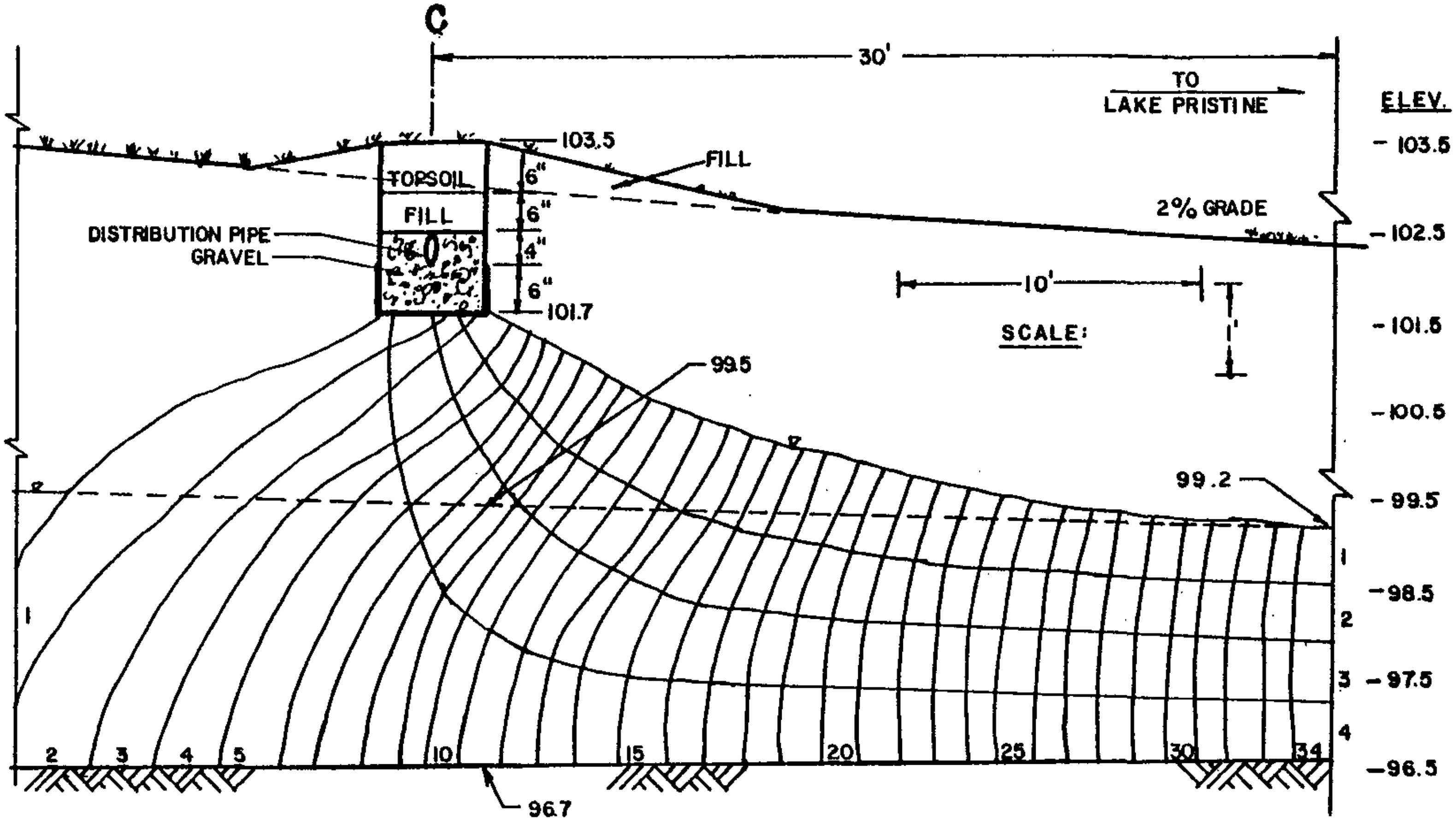


FIGURE D-2: FLOW NET ANALYSIS (scale drawing) sidewall exfiltration is customarily neglected - a sort of safety factor.)

Finally, we can apply Darcy's equation, an empirical expression representing laminar fluid flow through a porous media; in this case, water through soil. The equation is:

$$Q = kiA \qquad (4)$$

Where Q is flow from higher to lower head, k is permeability (or hydraulic conductivity), i is hydraulic gradient and A is area. At the Salomaki property:

$$Q = 2.63 \text{ ft/day} \times 4/34 \times 350 \text{ ft}^2 \qquad (5)$$

$$Q = 106 \text{ ft}^3\text{/day} = 810 \text{ gallons/day} \qquad (6)$$

As our anticipated wastewater flow to the site is 675 gallons per day, we conclude that under our assumed conditions, the site has the hydraulic capacity to remove the wastewater generated. An additional hydraulic load is infiltration from wet weather events. The remaining hydraulic capacity allows this site to remove 135 gallons of infiltration per day through the trench area, or approximately 0.5 inches per day, a small but not necessarily restrictive amount. During final design and construction, we will shape the absorption field area to divert runoff and precipitation away and limit infiltration by placing six inches of low permeability topsoil over the trenches.

Bacterial Mat Design

Having determined that our estimated absorption trench size can convey the Salomaki's wastewater to the groundwater, we must determine if this quantity of wastewater can safely and reliably be transmitted through the bacterial mat to the groundwater. In this analysis we are concerned with both hydraulic transmittance through the bacterial mat and wastewater renovation.

The hydraulic transmittance of the bacterial mat, in the long-run, LTAR concept, can be estimated by equation two (presented earlier; Laak, 1980a):

$$\text{LTAR loading rate} = 5k - \{1.2/\log k\} \qquad (2)$$

Where k is permeability in ft/minute and loading rate is in gallons per square feet per day. The literature also provides a graphic description of this relationship, shown in Table 4 (Healy and Laak, 1974). Substituting the permeability at the Salomaki site, 1.82×10^{-3} ft/minute (80 cm/day), into equation two yields a LTAR of 0.45 gallons per square foot per day (1.8 cm/day). Use of the graph produces a similar number.

Applying this LTAR to our expected wastewater flow rate of 675 gallons per day indicates that 1,500 square feet of absorption area are necessary for long-term operation of the system.

At this point the designer should check the characteristics of the wastewater that will be applied to the absorption field. If the wastewater had particularly high BOD or SS concentrations, as might occur in some industrial locations, the designer should increase the absorption area size to account for the increased thickness (decreasing permeability) of the bacterial mat. Equation one, presented earlier, describes this relationship, empirically derived by Laak, Healy and Hardisty (1974), based on work by Laak (1970). For example, were the sum of BOD_5 and SS 335 mg/l, a ten percent increase in absorption area size would be necessary. For the Salomaki property, we expect effluent from the septic tank to be similar to that of an average two-compartment septic tank. From Table 3, we know that the sum of BOD_5 and SS from a two compartment tank receiving residential wastewater is 141 mg/l. Therefore, utilizing equation one, we expect to be able to decrease our required absorption area by approximately 15 percent (as long as this does not exceed the hydraulic capacity of the site). The new absorptive surface area required for long term performance is 1,250 square feet.

Absorption Field Design

The design of the absorption field itself is constrained by several restrictions inherent to the development of an absorption trench cross-section. First, one foot of cover over the distribution pipe must be provided for insulation and protection from surface loads. If in continuous use, these pipes will not freeze, even where frost depths reach five feet (U. S. EPA, 1980b). Next, the pipe itself is four inches in diameter. A minimum gravel bedding depth, to support the pipe, provide storage of wastewater and to distribute flows, is six inches. Twelve inches or more is desirable. Finally, sufficient depth to creviced bedrock and groundwater must be provided to protect water quality.

Four feet is a suggested minimum depth from the bottom of the soil absorption trench to creviced bedrock (U. S. EPA, 1980b). Such a large distance is due to the uncertainty of fluid flow within creviced bedrock and therefore, the potential for contamination of a drinking water source, especially in rural areas where groundwater wells are common. Two feet of soil over the groundwater table is suggested to prevent groundwater contamination (U. S. EPA, 1980b). Although the literature indicates that essentially complete renovation of septic tank effluent can occur within one foot of trench bottom - provided that unsaturated soil conditions exist - two feet is perhaps a better, more protective without being excessively restrictive depth to groundwater limit.

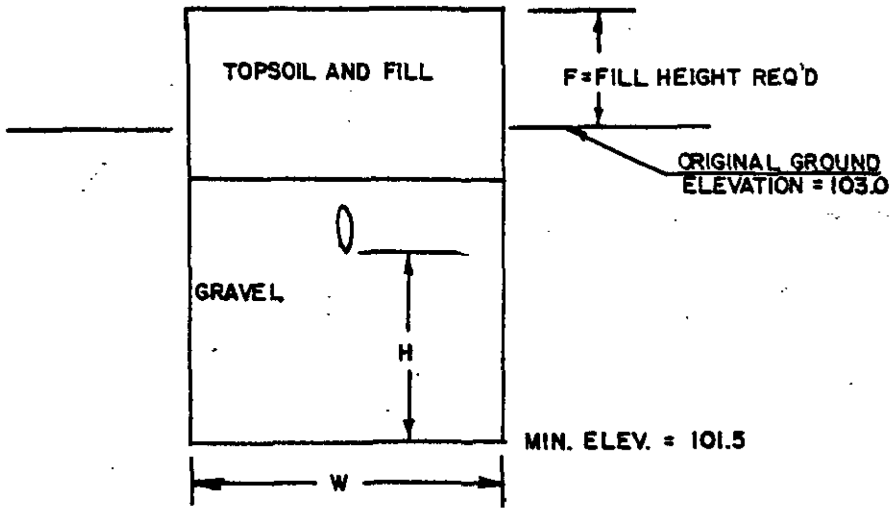
Our site is also restricted by the distance to a surface water body (Lake Pristine). Generally 50 feet from the edge of the absorption field to the shoreline is suggested to prevent contamination of a water body. Applying Darcy's Law to our site indicates that at least 165 days are necessary for fluid to travel fifty feet (at the i = 3/26 hydraulic gradient - a conservative gradient when considering the entire 50 foot distance), a safe value. Where rapidly permeable soils exist, the potential for nutrient and/or microbiological contamination of the water body exists. In such soils it may be necessary to move the absorption field further away from the waterbody.

The next task, having decided that our system is not located too close to Lake Pristine's shoreline, is to develop the trench configuration. Because of our shallow water table, and the trench restrictions discussed above, we must raise our trenches slightly. Development of the trench configuration (at this point concentrating primarily on its cross-section) is a trial and error procedure. We are constrained vertically by the minimal depths to groundwater and trench shape. Horizontally, we are limited to 3.5 feet by our construction practices. And finally, we must provide room for three days storage of septic tank effluent within the gravel or crushed stone distribution system.

The storage requirement necessitates determining the void volume of the gravel or crushed stone. Generally, the void volume of gravel is estimated between 20 and 40 percent (Sowers, 1979). For this example, we will assume 30 percent. The void volume of crushed stone would probably be similar; consultation with the crushing plant that the materials are obtained from would probably be the best approach to determine its void volume more accurately.

At the given wastewater generation rate, 90 cubic feet (675 gallons) per day, three days flow has volume of 270 cubic feet. Assuming 30 percent void volume, this requires 900 cubic feet of gravel within the absorption trench and below the distribution pipe invert. We must make an engineering judgement: Whether to make the trenches taller or to maintain shallow, wide trenches that require more linear feet of absorption trench. In this analysis, the trench sidewall area below the distribution pipe invert should be considered as an exfiltrative surface. Bouma (1975) suggests that for low permeability soils, only the trench bottom be considered as an exfiltrative surface, a sort of safety factor. For this sandy-loam the decision to consider sidewall exfiltration is appropriate. Figure D-3 demonstrates the various alternatives and their effect on system length.

After the trial and error procedure, and consultation with the Salomaki's to determine how great an increase in ground elevation is acceptable, the final cross-sectional segment shown in Figure D-3 is arrived at. It is not the most economical solution, but one that is most acceptable to the Salomaki's.



VOLUME = FT^3 AREA = FT^2 LENGTH = FT   • = CRITICAL PARAMETER

W	H	PER FOOT OF TRENCH LENGTH VOID VOLUME	PER FOOT OF TRENCH LENGTH ABSORPTION AREA	REQUIRED TRENCH LENGTH	F
2	2	4 •	6	225	1.8
2.5	3	8.75	8.5 •	147	2.8
3.5	3.5	12.25	10.5 •	119	3.3
3.5	0.5	1.75 •	4.5	514	0.3
3.5	1	3.5 •	5.5	257	0.8
3.5	0.75	2.63 •	5.0	342	0.6

CHOSEN CROSS SECTION = 3.5 X 1

FIGURE D-3: ALTERNATIVE CROSS SECTIONS

Next, the configuration of the trenches on the lot must be developed. During this location process it is important to avoid existing structures and area where vehicles might travel. Whenever possible, room should be left available for absorption system expansion should it ever become necessary. A minimum distance between trenches of 3.5 times the trench width is an acceptable separation distance. For the Salomaki property, a somewhat rectangular system, utilizing a distribution box to evenly distribute flow to all laterals is employed. Generally, 100 feet is the maximum length desirable for a distribution lateral. Shorter distances are more desirable. Figure D-4 shows the final system layout.

The distribution laterals themselves should be sloped slightly to aid their ability to distribute septic tank effluent. The septic tank will remove almost all solid materials, negating any need for a fast, "scouring" velocity within the distribution pipes. In most instances, a slope of 0.1 to 0.3 percent is sufficient.

At this point, a check should be made to see if any of the decisions made regarding absorption trench design adversely effect the site's hydraulic ability to accept all of the wastewater generated. In the initial hydraulic analysis, a trench 100 feet by 3.5 feet was assumed. As the final system design utilizes an area greater than this and distributes the hydraulic input over a greater area, we determine that our design revisions do not exceed the site's hydraulic capacity.

Septic Tank Design

The remaining component of the distribution system to be designed is the septic tank. Our design criteria will be to provide 24 hours flow retention, minimize upflow velocity and short-circuiting, prevent solids carry-over to the absorption field and provide for several years accumulation of solids and grease.

The average daily design flow at the Salomaki site is 675 gallons per day (90 ft^3). Therefore, the septic tank "clear space" should be this large or greater.

The accumulation of solids and grease can be estimated at approximately 62.5 gallons per capita per year (U. S. EPA, 1980f). Designing to provide for three years accumulation:

$$62.5 \text{ gal/cap/yr} \times 10 \text{ cap} \times 3 \text{ yr} = 1{,}875 \text{ gallons} \qquad (7)$$

indicates that 1,875 (250 ft^3) must be provided for accumulation of grease and solids. Therefore, the total volume to be provided below the effluent invert elevation is 2,550 gallons (340 ft^3).

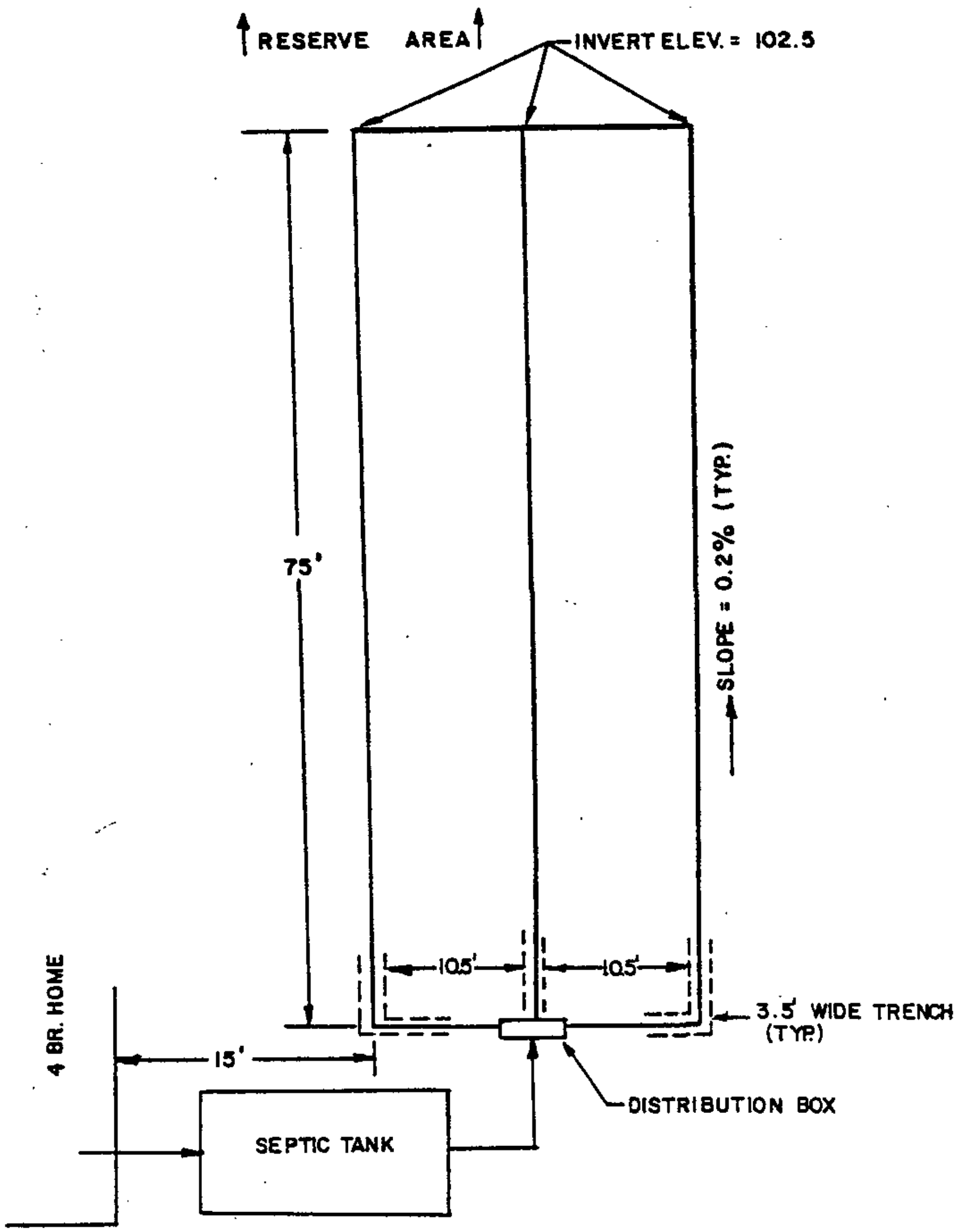


FIGURE D-4: FINAL ABSORPTION TRENCH CONFIGURATION

Our tank should conform to several "rules-of-thumb" that traditionally have been used to ensure that tank performance is satisfactory in several aspects. For example, for ease in cleaning, construction and to reduce upflow velocity, the tank depth should not exceed six feet. To prevent wastewater influent from disturbing solids and grease, its depth should be greater than four feet. Compartmentation of the tank should provide that the first compartment is twice the volume of the second. Finally, the surface area to depth "ratio," with surface area in square feet and depth in feet, should be greater than two in each chamber.

A trial and error process is then utilized, trading off length and width of the tank with height. After several tries, the final tank design, shown in Figure D-5, is arrived at. Its final construction should include manholes, baffles and gas deflectors and perhaps an inspection port as discussed in chapter three.

As a final precaution, when installing septic tanks in areas of shallow groundwater elevation, beware that unless properly anchored, the tank may float when empty (as might occur during installation or after cleaning), potentially causing structural failures. A concrete pad may provide sufficient anchorage when properly attached to the tank.

C. Wastewater Disposal Mounds

Wastewater disposal mounds are a type of soil absorption system, particularly suitable where high groundwater, an impermeable layer, excessively permeable or low permeability soils exist. Mounds were developed at the North Dakota Agricultural College in the late 1940's (California, 1980), and are occasionally cited as "NODAK" systems, in deference to their original design. Their monitoring revealed that, due to insufficient attenuation of septic tank effluent within the mound, inadequate treatment performance often occurred. NODAK systems have since been modified, more recently by Bouma et al. (1975), the U. S. EPA (1978; 1980b), California Water Resources Control Board (1980) and Otis (1982c). Properly designed and constructed, mounds should treat septic tank effluent satisfactorily with virtually no regular maintenance (U. S. EPA, 1980b).

Mound systems are essentially raised soil absorption fields. As such, the mechanisms and properties pertinent to their construction, operation, and maintenance are very similar to those pertinent to soil absorption systems in general, and described in the first portion of this chapter. Several mound configurations have been tested and their performance reported (U. S. EPA, 1978). Most currently suggested mound designs are slight modifications of the "Wisconsin Mound Design" described in a report prepared at the University of Wisconsin: Management of Small Waste Flows (U. S. EPA, 1978). A previous "Pennsylvanian" mound design

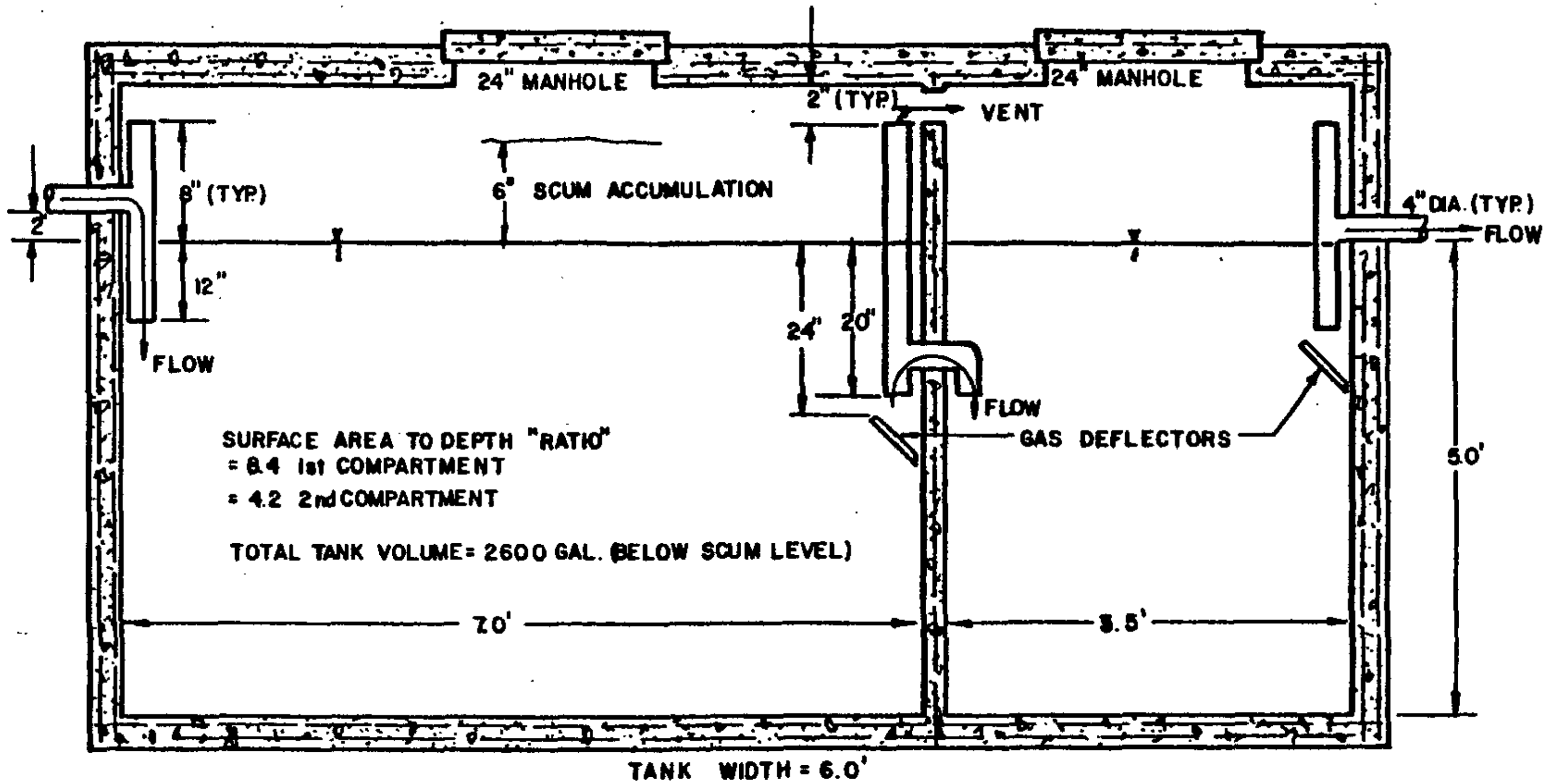


FIGURE D-5: FINAL SEPTIC TANK DESIGN (4 BR. HOUSE)

SECTIONAL VIEW - NO SCALE

suffered from inadequate hydraulic capacity (Mott, Fritton and Peterson, 1981) and has since been abandoned in favor of the "Wisconsin" mound (Otis, 1982d). Essentially, a sand fill is placed above a plowed existing surface. Gravel (or similar) material is placed over the sand fill. A distribution network of piping and gravel (or similar material) trenches or beds discharges septic tank effluent to the sand fill. The entire system is covered with a landscaped, less permeable soil. Figure 4 shows a mound system schematic.

Current Massachusetts subsurface disposal regulations (Comm. of Mass., 1978) do not permit the construction or use of any type of wastewater disposal mound. These regulations do permit construction of subsurface disposal systems in fill material, but the soil overlain by fill material must, by itself, be suitable for disposal field construction.

Thus many sites in Massachusetts are currently unsuitable, due only to existing (somewhat archeic) subsurface disposal regulations, for ST-SA system use. Construction of any subsurface disposal system in Massachusetts is prohibited in soils whose percolation rate is slower than thirty minutes per inch. Large disposal systems estimated to discharge more than 2000 gallons of septic tank effluent per day must be located on soils with percolation rates of at least twenty minutes per inch. Another regulation that restricts the use and construction of subsurface disposal systems in Massachusetts is that the maximum groundwater elevation must be at least (and for several disposal system designs, more than) five and one-half feet below ground surface (Comm. of Mass., 1978). Bouma et al. (1975) point out that soils with percolation rates slower than 60 minutes per inch often have seasonal water tables in spring or fall within two feet of the soil surface, due to perching of infiltrating water on top of slowly permeable subsoil horizons or due to lateral fluid movement through the topsoil. In Massachusetts, this implies that many building lots located on slowly permeable soils are currently unsuitable for development due to site percolation test and groundwater restrictions when sewerage or other on-site systems are unavailable or impractical.

Properly designed and constructed, wastewater disposal mounds can reliably and safely discharge septic tank effluent to soils with percolation rates as slow as 900 minutes per inch and groundwater elevation less than two feet from the soil surface. The U. S. EPA (1978) and Bouma et al. (1975) describe several mound systems installed at residential sites in Wisconsin. Three of these sites had soil percolation rates of 900 minutes per inch. Some seepage was experienced through the sides of two of these three mounds but it was felt that better distribution networks and plowing of the infiltrative surface, as suggested in current mound designs, would have prevented this (U. S. EPA, 1978). The U. S. EPA (1980b) recommends that at least twenty inches of unsaturated soil exist between the existing surface and maximum groundwater elevation. However, Simons and Magdoff (1979a) report

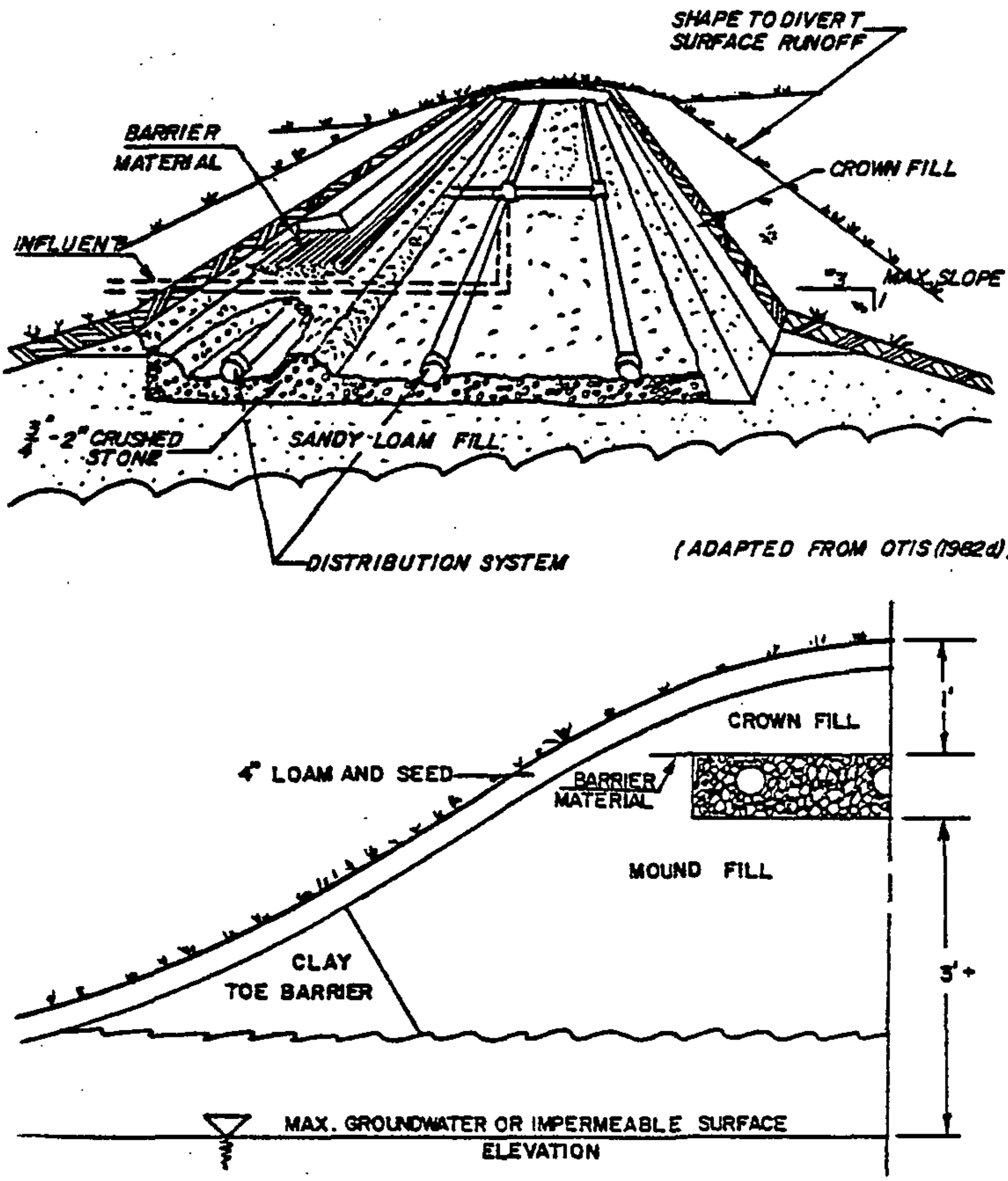


FIGURE 4: MOUND SYSTEM SCHEMATIC (TOP)
SECTIONAL VIEW (BOTTOM) NO SCALE

satisfactory performance of a wastewater disposal mound while seasonally perched groundwater came within two centimeters (one inch) of original ground surface.

The U. S. EPA (1978) describes a procedure to size mounds. If a medium gradation sand fill is used with a gravel bed distribution system, the bed should be sized at 5 cm/day. With a final mound height of 4.5 to 5.0 feet and sideslopes no steeper than 3:1, the basal area becomes much larger than is needed to absorb applied septic tank effluent based on the infiltrative capacity of the existing soil. If less permeable fill materials are used, lower hydraulic loading rates are required.

The sideslope requirement (to ensure stability) creates a large absorption area. Bouma et al. (1975) recommend 5:1 sideslopes. Couture (1978) illustrates that for permeability data presented by Bouma et al. (1975), at this slope, a five foot tall mound becomes approximately ten times wider than soil hydraulics require. More recent design guidelines (U. S. EPA, 1980b; Otis, 1982c; 1982d) suggest 3:1 sideslopes. This still requires a large basal area and a significant quantity of fill material.

Because of their size, mounds are expensive to construct and may be unaesthetic. Bouma et al. (1975) estimated (based on 5:1 sideslopes) 2500 to 3000 dollars construction cost per mound system. Properly landscaped however, a mound should not necessarily detract from a home's appearance. And if a pressure distribution network is employed, it may simply be a matter of extending the septic tank effluent transmission lines (restricted only by cost and headloss) to a more suitable mound location. The California Water Resources Control Board (1980), U. S. EPA (1980b), and Otis (1982d) illustrate several mound configurations adapting the mound concept to varying site requirements.

The depth of fill necessary to be placed over existing soil depends on the existing depth to groundwater, creviced bedrock or impermeable surface. Laak (1980a) and the U. S. EPA (1980b) illustrate that where a seasonally high groundwater table is of concern, absorption trenches could be constructed closer to the ground surface than normal, placing fill over the trenches for insulation only. Where groundwater is too close to the ground surface to allow this or where mounds are placed to overcome impermeable or excessively permeable soils, the depth of fill must be sufficient to provide renovation of septic tank effluent before reaching groundwater. A field study by Couture (1978) observed significant reductions in nutrient and organic pollutant parameters in the first six inches of fill below the distribution trench of a mound system. Fluctuations in COD removals below this depth were attributed to short circuiting and degradation of bacterial polysaccarides during anaerobic conditions. Anaerobic conditions were evidenced by significant nitrate reductions, attributed to denitrification processes. (Generally, field and laboratory studies do not report significant denitrification in mound systems). Experiments by Simons and Magdoff (1979b) using laboratory soil columns indicated that if unsaturated conditions are maintained in a sand fill, 30 centimeters (12 inches) of fill is sufficient for renovation processes to occur. The U. S. EPA (1980b) also indicates that 30 centimeters (12 inches) of fill is sufficient to provide renovation of septic tank effluent. Simons and Magdoff (1979b), Bouma et al. (1975) and the U. S. EPA (1978) recommend 60 centimeters (24 inches) sand fill in mound systems placed over low permeability soils. A seemingly more rational approach than these, presented by Otis (1982d), suggests that three feet (90 centimeters) of unsaturated soil, the combination of existing soil and fill material, exist between the bottom of the absorption trench and maximum groundwater table. Where mounds overlie permeable soils with shallow creviced bedrock, Otis (1982d) recommends a total of four feet of fill and existing soil because of the greater risk of contaminating groundwater used for water supply.

Sand is often suggested for use as fill material in wastewater disposal mounds (Bouma et al., 1975; U. S.EPA, 1978; Simons and Magdoff, 1979a; 1979b; Mott, Fritton and Peterson, 1981). Gravel was originally used in "NODAK" mounds but proved to be too permeable to provide satisfactory treatment of septic tank effluent (U. S. EPA, 1978), and should not be used. Other materials, such as clay-loams and silt-loams may be more suitable, especially where phosphorus retention within the mound is important. The phosphorus removal characteristics of these soils are described in the section "On-Site Phosphorus Removal." These materials have lower permeability than sand and therefore, must be loaded at lower hydraulic rates. Unfortunately, lower hydraulic loading rates increase disposal mound size and, hence, its construction cost. The U. S. EPA (1980b) suggests that, for economy, fill material be from a local source.

The "Wisconsin" mound design suggests 5 cm/day loading of the sand fill (U. S. EPA, 1978). Otis (1982c) suggests 5 cm/day for medium sand and sand/sandy loam mixtures and 2.5 cm/day for sandy-loam fill material.

Simons and Magdoff (1979b) performed column studies designed to simulate a wastewater disposal mound constructed over a low permeability soil. Septic tank effluent loading and depth of sand were varied. Columns loaded at less than 3.4 cm/day never failed. Based on their soil columns, they suggest 2 cm/day hydraulic loading for design but do not consider the increase in basal area a mound provides nor report if hydraulic failure in failed columns was due to low permeability soil or clogging at the gravel/fill interface.

Perhaps a more suitable method for determining a hydraulic application rate is that described in the previous section, "Soil Absorption Systems": Design an absorptive surface loading based on the LTAR of the fill material and, using a flow net analysis, be certain this loading is less than that soil's hydraulic capacity. For a mound system, it is also necessary to prevent overloading at the fill/soil interface. This requires comparing the permeability of the mound basal area with the flow this area must accept, including any precipitation or runoff inputs. In most cases it appears that, due to the large basal area formed by the sideslope requirement, failure at the fill/soil interface is unlikely. For a less involved design a loading rate based on the classification of the soil used for fill material can be chosen to size the gravel/fill absorptive surface area. These values can be found in Table 3 (in previous section).

Mounds should be shaped to conform to the contour of the site and to divert runoff (U. S. EPA, 1980b). In most instances, a rectangular bed with its long axis parallel to the slope contour is preferred to minimize the risk of seepage from the base of the mound (U. S. EPA, 1980b). In soils with percolation rates greater than 60 minutes per inch, the bed can be square if the water table is at least three feet from the original ground surface (U. S. EPA, 1978). Mounds should be oriented so that they are along convex and not concave slopes, again to better divert runoff and prevent seepage (U. S. EPA, 1980b).

Before and during construction, care should be taken to prevent compaction, which may decrease permeability of the existing soil. Mound construction should should occur only when the existing soil moisture content is below its plastic limit, so that smearing of infiltrative surfaces does not occur (Otis, 1982d). The first step, once the mound location has been chosen, is to plow the existing soil surface. Plowing helps ensure that the entire basal area may act as an infiltrative surface. It is suggested that soil be plowed to a depth of eight inches along the contour of the land, throwing soil upslope (Otis, 1982d). The use of disc plowing implements is discouraged as it may break soil into finer particles, further reducing soil permeability (California, 1980).

Immediately next, fill material is placed over the plowed surface, exercising care not to disturb or compact the plowed surface. Track mounted construction equipment is preferred over rubber tired equipment when working near and on the mound (Otis, 1982d). Rubber tired equipment is more likely to disturb the plowed surface during construction (Otis, 1982d). Mechanical compaction of the fill is not recommended, but as Couture (1979) attributes settlement of fill material for a six inch deficiency in actual mound height compared to design specifications, it may be desirable during construction to place fill material slightly higher than design specifications indicate.

The distribution network, gravel trenches or beds and conveyance piping, are placed next. Sufficient gravel pore space should exist below the piping to store several days flow to dampen the effect of peak flows.

A barrier, designed to prevent finer cover material from settling into and clogging the gravel pores, should be placed over the distribution network. The barrier may be a permeable filter fabric such as those used in roadway construction or straw or marsh hay as suggested by Bouma et al. (1975).

A low permeability clay toe barrier may be desirable to prevent seepage through this region during periods of high flow through the mound or high groundwater (lower hydraulic gradient). The toe barrier should extend below the existing soil surface to prevent flow along the toe barrier/soil interface. Clay material may also be placed over the distribution network barrier, to reduce infiltration into the disposal mound.

The entire mound should be covered with six inches of low permeability topsoil (Bouma et al., 1975) to reduce infiltration and support a vegetative cover. The cover should be shaped to divert runoff water away from the mound (U. S. EPA, 1980b). At least one foot total cover, topsoil and clay, over the distribution network is necessary to prevent freezing.

CHAPTER 5

Phosphorus Considerations

A. Current Adequacy of Treatment Performance

In most cases, the on-site wastewater disposal systems described in chapter four will provide sufficient wastewater renovation. Effluent from ST-SA systems is not completely innocuous, however. For example, nitrification occurring in absorption fields can induce potentially fatal methemoglobinemia in infants (Medovy, 1948; Bucklin and Myint, 1960) if drinking water concentrations exceed 10 mg/l NO_3-N. Since denitrification (a nitrate removal mechanism) is difficult to induce below absorption fields, engineers have relied on dilution to reduce groundwater nitrate concentrations to acceptable levels. Wastewater phosphorus also is not always removed to innocuous levels by ST-SA system treatment.

Phosphorus is of great concern, and correctly so, in many lakefront communities. Phosphorus concentrations, often critical to lake eutrophication, can significantly affect lake water quality. Water quality affects the desirability of the lake as a recreational and drinking water source, which in turn, affects the value of real property along these lakes (Bachman, 1980).

Phosphorus may be introduced to a lake waterbody from several sources. Through fertilization of agricultural lands, phosphorus may percolate to groundwater and be carried to a waterbody. Phosphorus may become associated with soil particles which, when eroded, may be carried to a waterbody by stormwater or rainfall (Wetzel, 1975). Upstream sources in general may transport runoff associated phosphorus from streets, fertilized lands and more developed areas to a receiving water. Phosphorus is also cycled within a lake, being released from sediments, incorporated into plant tissue and returned to the sediment when plant life ceases. Finally, and most importantly to this report, phosphorus can be introduced to a waterbody from inadequate or improperly operating wastewater treatment systems.

A significant quantity of phosphorus is present in rural domestic wastewater. Total phosphorus production from rural households is estimated by several sources at approximately 0.009 lb/cap/day (Siegrist et al., 1976; U. S. EPA, 1978; Laak, 1980b; U. S. EPA, 1980b). (Total phosphorus is the sum of many forms of phosphorus, some of which must be hydrolyzed to become available as a plant nutrient.) The major contribution of phosphorus to wastewater is the use of detergents with phosphate builders. The next most important contribution is blackwater (toilet wastes).

The relative importance of each of the above mentioned phosphorus loads to a waterbody is site specific. The accumulation of phosphorus in a waterbody depends on the hydraulic flow regime, the extent of sedimentation and the degree of biological productivity. Generally, the internal phosphorus loading is small (Lee, Rast and Jones, 1978). Phosphorus input from agricultural lands and upstream inputs depends on soil management practices and the characteristics of land and land use in the watershed. In most cases, as will be discussed in detail in this chapter, phosphorus is not significantly introduced to waterbodies from properly designed and operating ST-SA systems. Remember however, that only in recent years have sound design criteria for ST-SA systems developed and that a lack of permissible alternatives to ST-SA systems in the past has quite probably caused improper applications of ST-SA systems in Massachusetts lakefront communities. Hence, as described below, significant contributions of phosphorus to a water body, attributable to ST-SA systems, can occur.

B. Phosphorus Management

Eutrophication is the slow natural process of silt and nutrient accumulation in lakes. Eventually, the lake becomes completely filled in. Man's activities can increase the rate of eutrophication by several orders of magnitude (cultural eutrophication), to decades or years instead of geologic ages (Atlas and Bartha, 1981).

Eutrophic lakes characteristically have high levels of biological productivity and plant nutrients, often reflected by high densities of planktonic algae and possibly dense beds of aquatic plants (Bachman, 1980). They may have decreased water transparency, lower hypolimnetic dissolved oxygen concentrations and changes in fish species composition.

Eutrophication is caused by an abundance of plant nutrients. It is widely accepted that the nutrient most often controlling production in fresh water systems, and therefore trophic status, is phosphorus, owing in part to its lack of natural abundance in available forms (Wetzel, 1975; Dillon, 1976; Lee, Rast and Jones, 1978; Welch, 1980; Sheehan, 1982). Restricting the phosphorus supply is often an effective means of restoring or preserving the quality of a lake (Schroeder, 1979).

Phosphorus Forms

Phosphorus in domestic sewage can be broken down into four classes: orthophosphates, polyphosphates, metaphosphates and organic phosphates. Inorganic phosphorus forms comprise the largest portion of domestic sewage. Unfortunately, in a waterbody, many dissolved inorganic phosphorus forms are directly available for (generally undesirable) biological growth (Browman et al., 1979). Orthophosphate species are pH dependent (H_3PO_4, $pK_{a,1} = 2.1$, $H_2PO_4^-$, $pK_{a,2} = 7.2$, HPO_4^{-2}, $pK_{a,3} = 12.3$, PO_4^{-3}; Snoeyink and Jenkins, 1979). They characteristically have a tetrahedral structure, a phosphorus atom surrounded by oxygen atoms (Greenfield and Clift, 1975). Polyphosphates and metaphosphates can be grouped together as condensed phosphates. Their major difference is structural: metaphosphates have a ring structure made up of orthophosphate groups while polyphosphates form a chain of orthophosphate groups (Greenfield and Clift, 1975). Condensed phosphates must be hydrolyzed to orthophosphate species before becoming available for biological assimilation. Prolonged contact with microorganisms ensures the hydrolysis of condensed phosphates to orthophosphate (Snoeyink and Jenkins, 1979). Organic phosphorus compounds in sewage may be from microbial tissue, plant residues and metabolic by-products of living organisms (Loehr et al. 1979b). Organic phosphorus forms are many. Some important species are inositols, phospholipids, phosphoamides, nucleotides and sugar phosphates (Snoeyink and Jenkins, 1979). Inositols are the predominant organic phosphorus form (Smyth and Lowry, 1980). Organic phosphorus forms may be bacterially decomposed to orthophosphate (Clark, Viessman and Hammer, 1977). In a soil absorption system this would occur in the bacterial mat (U. S. EPA, 1977e).

Phosphorus Removal in Centralized Treatment Plants

Once in a waste stream, there are several options for phosphorus removal. Waste is often collected and removed to a central wastewater treatment facility. Here physical, biological, and chemical processes may remove phosphorus. Significant removal of phosphorus by conventional wastewater treatment schemes is unlikely. Properly designed and operated however, advanced wastewater treatment facilities can remove up to 90 percent of total phosphorus at reasonable cost (Switzenbaum et al., 1981). Residential on-site wastewater systems for phosphorus control often depend on soil to retain phosphorus or chemicals to precipitate a removable phosphorus compound.

At conventional wastewater treatment facilities, non-soluble phosphorus (approximately 10 percent of the total phosphorus load) may be settled from the wastewater during primary treatment (Metcalf and Eddy, 1979). A small amount of phosphorus will normally be consumed by bacterial growth requirements in secondary treatment processes. Bacterial phosphorus requirements are approximately 1/25th of their carbon requirement (in moles) while growing under nutrient-rich conditions (McCarty, 1975). When stressed and starved for phosphorus however, bacteria may develop a tendency to consume more phosphorus than their stoichiometric requirements, known as "luxury uptake." Significant phosphorus removals can be achieved by this process.

In advanced (or tertiary) wastewater treatment facilities, phosphorus is often chemically precipitated from wastewater. Precipitation is induced by adding aluminum, calcium or iron salts. While the exact chemical reactions are complex, they have been generally outlined in several sources (U. S. EPA, 1971; Metcalf and Eddy, 1979; Steel and McGhee, 1979; Snoeyink and Jenkins, 1980). Basically, cationic forms of aluminum, iron or calcium form an insoluble precipitate with orthophosphate. Condensed phosphates and organic phosphorus are removed by a combination of more complex reactions and sorption on floc particles (U. S. EPA, 1971). Competing reactions and kinetics may require the addition of mineral salts in excess of their suspected stoichiometric requirements. The characteristics of influent sewage significantly affect precipitation reactions. Influent wastewater pH is important to chemical treatment performance as it affects both orthophosphate species and solubility of precipitated compounds. Influent wastewater alkalinity is important as it is often consumed by precipitation reactions and therefore affects effluent pH. Low alkalinity wastewaters treated with alum (an aluminum salt) may require lime addition during treatment to offset pH suppression due to alkalinity consumption by both nitrification and precipitation reactions (Martel, DiGiano and Pariseau, 1977). In this case, other sources of aluminum may be more suitable. The U. S. EPA (1971) and Metcalf and Eddy (1979) outline the advantages and disadvantages of chemical precipitation at various points in a conventional activated sludge treatment system.

Chemical precipitation produces a significant quantity of chemical sludge. Martel, DiGiano and Pariseau (1977) report that sludge production tripled (by weight) when sodium aluminate was added to an extended aeration process. The addition of alum (and lime to control pH) in place of sodium aluminate resulted in sludge weight production increase of approximately 130 percent. Sludge production increases (in percent of weight) at conventional activated sludge plants are less.

Phosphate Detergent Bans

Reducing the phosphorus concentration of residential wastewaters may reduce the phosphorus loading to a waterbody. The phosphorus output from residences can most significantly and easily be reduced by the use of low phosphate detergents.

(The second major source of phosphorus in domestic wastewater is the blackwater contribution. Fecal and non-fecal mass contribution per day is approximately equivalent; 5.94×10^{-4}

lb/cap/day (Siegrist, Witt and Boyle, 1976). It appears that dietary changes, a significant cultural or sociological change, would be required to reduce this component.)

Phosphorus, in the form of pentasodium triphosphate (PSTP; $Na_5P_3O_{10}$) is often added to detergents to aid in cleaning. PSTP forms strongly bound soluble complexes with calcium and magnesium ions, softening the water. PSTP keeps dirt suspended, away from fabrics during the wash and prevents the deposition of insoluble calcium and magnesium salts (Gilbert and De Jong, 1978). PSTP has favorable toxicological, structural and cost characteristics (Gilbert and De Jong, 1978). Its major disadvantage is that when discharged to an aquatic environment, it may become available as a nutrient for undesirable aquatic primary productivity (Alexander, 1978).

No substitute has yet been found that is as effective, safe and inexpensive as PSTP for detergents (Gilbert and De Jong, 1978). Several compounds do exist that can provide detergent effects at reasonable costs. Gilbert and De Jong (1978) review several of these. Nitrilotriacetic acid (NTA) performance and cost is similar to PSTP but is a suspected carcinogen. Further, biodegradation of NTA may increase nitrate concentrations in the wastewater. A sodium carbonate-silicate mixture performs less efficiently than PSTP and may leave precipitated calcium and magnesium forms on fabric and washing equipment but has been used where PSTP and NTA were not permitted. Zeolites and organic compounds have also been evaluated. The most promising of these appears to be the organic carboxymethoxysuccinate (CMOS) due to its lack of short and long term toxicity, biodegradability and ability to perform under United States laundering practices. (European laundering practices favor much higher wash temperatures.)

Phosphate detergent bans may remove up to 75 percent of total phosphorus from the domestic wastewater. Pieczonka and Hobson (1974) found a 56 percent reduction in average total phosphorus at the Lackawanna, New York, sewage treatment plant after a phosphate detergent ban was enacted. Sawyer (1965) estimated that 50 to 75 percent of total phosphorus in a domestic waste stream is attributable to phosphates in detergents. The average estimate of Ligman, Hutzler and Boyle (1974) is 67 percent. Data from Siegrist, Witt and Boyle (1976) indicates that 70 percent of total phosphorus is attributable to detergents. Alexander (1978) estimates 71 to 75 percent. Alexander (1978) also describes the rationale for the U. S. EPA urging a phosphate detergent ban in the Great Lakes watershed. He points out that in practice, phosphate removal objectives at wastewater treatment plants are often not achieved, phosphate detergent bans may reduce chemical costs for phosphorus precipitation at the treatment plant, and that phosphate detergent bans elsewhere have been accepted by consumers. Pieczonka and Hobson (1974) found 70 percent chemical cost savings and suspected significant sludge handling cost savings after a phosphate detergent ban was enacted in Lackawanna, New York. In general, phosphate detergent bans seem an effective step to reduce domestic phosphorus output without placing much strain on the consumer.

Regarding the reliability of treatment plant performance referred to by Alexander (1978): Switzenbaum et al. (1980) reviewed responses from a questionnaire sent to 229 wastewater treatment plants with flows greater than one million gallons per day in the lower Great Lakes basin. Here, 80 percent of the responses indicated that phosphorus removal was being practiced; yet only 52 percent of treatment plants responding were discharging less than 1.0 mg/l total phosphorus. Treatment plants employing "truly tertiary processes" seemed to consistently achieve 1.0 mg/l effluent total phosphorus, although 0.5 mg/l effluent total phosphorus concentration was the treatment goal. Apparently, the critical factor in phosphorus removal performance is process design. Phosphorus removal to 1.0 mg/l can reliably be achieved without resorting to filtration when chemical precipitation followed by conservatively designed and operated clarification facilities is practiced (Switzenbaum et al., 1981).

C. On-Site Phosphorus Removal

On-site systems, similar to conventional centralized treatment schemes, may use chemical precipitation to achieve phosphorus removal. For example, package plants or septic tanks can be equipped to add precipitant to their influent. Practically however, these systems require a greater degree of operation and maintenance than most homeowners will be willing to provide, both for chemical addition and sludge removal.

Brandes (1977) describes the use of alum for phosphorus precipitation in a blackwater septic tank. Alum was automatically dosed to the conveyance piping in the home after each toilet flush. Greater than 95 percent total phosphorus removal was achieved when properly dosed. Improved BOD_5, SS, fecal and total coliform, iron, sodium, potassium and chloride removals within the septic tank are also reported. Sludge production increased by a factor of 2.35 (by weight). Dampening the effect this increase would have on septic tank pump-out frequency was an increase in sludge density. This study indicates very low chemical costs for operation of this system (4.43 dollars per capita-year).

On-site systems that discharge their waste to a soil absorption field may more reliably, and with less labor, remove phosphorus from the waste stream. Soils may have a great capacity to retain phosphorus and, as previously discussed, where suitable soils exist, subsurface treatment is probably the most reliable and cost efficient method of wastewater disposal (Otis, 1982a).

It is unlikely that the phosphorus loading to a waterbody from a properly operating ST-SA system would be significant. Soils generally are extremely efficient at removing phosphorus from applied wastewaters (Gilliom and Patmont, 1983). Only where ST-SA systems are improperly implemented or in soils with little sorption capacity (Sikora and Corey, 1976) would the pollution potential of phosphorus from septic tank effluent be considerable.

Gilliom and Patmont (1983) performed groundwater monitoring at Pine Lake, Washington, and report that old septic tank-soil absorption systems (1940-1950 construction) located in saturated soils may not efficiently remove phosphorus and therefore, introduce phosphorus to a waterbody. Generally, 99 percent removal of septic tank effluent phosphorus in properly designed and operating systems occurred (Gilliom and Patmont, 1983). Absorption fields in their study were constructed on an acidic permeable soil (Alderwood) underlain by a less permeable glacial till.

A literature search and four year groundwater monitoring program at an active subsurface absorption system in sandy soil in Burnett County, Washington, was performed to study phosphorus transport (U. S. EPA, 1977e). The groundwater monitoring program indicated that downstream of the absorption field, no phosphorus contamination had occurred. The literature review concluded that: (1) soil minerology was more important than soil particle size to phosphorus removal, (2) usually, within short distances of effluent application, greater than 95 percent total phosphorus removal occurs in soil, and (3) septic tank wastewater disposal systems generally do not contribute significant quantities of phosphorus to surface waters.

Phosphorus is present in soils in both organic and inorganic forms. Their relative distribution varies widely and depends on soil type (Keeney and Wildung, 1977). Most phosphorus in soils is associated with the solid phase, hence the concentration of phosphorus in the soil solution rarely exceeds one mg/l (Keeney and Wildung, 1977).

Phosphorus Retention Mechanisms

Within the soil matrix there are five mechanisms of soluble phosphorus retention: biological uptake, physical adsorption, anion exchange, chemical adsorption (chemisorption) and chemical precipitation (Smyth and Lowry, 1980). Of these, chemisorption and chemical precipitation are the most significant. Biological phosphorus removal within the soil matrix results from flora and fauna activity. During the growing season, as evidenced by application of secondary effluent to a soil filter bed in Northern Minnesota (Nichols and Boelter, 1979), vegetation may remove 22 to 45 percent of total phosphorus. Physical adsorption occurs as a result of van der Waals forces, hence it characteristically has low bonding energies (Weber, 1972). Phosphate anions may only be temporarily removed from an aqueous system by physical adsorption (Smyth and Lowry, 1980). Anion exchange, a form of exchange adsorption, is also not a significant phosphorus removal mechanism (Smyth and Lowry, 1980). As the net ionic charge on colloidal soil particles is overwhelmingly negative (Loehr et al., 1979a), the attraction of phosphorus forms (predominantly anionic) to the soil matrix by this mechanism is unlikely. Only in organic soils can anion exchange be a significant phosphorus removal mechanism. Chemisorption is a very significant phosphorus removal mechanism, especially at total phosphorus concentrations less than 5 mg/l (Sikora and Corey, 1976). Chemisorption exhibits high energies of adsorption, forming chemical bonds with the adsorbent (Weber, 1972). Chemisorption is similar to chemical precipitation but does not require that ions be released from the soil mineral to form the chemical bond as precipitation does (Smyth and Lowry, 1980). Chemical precipitation, the formation of relatively insoluble products from constituents that previously were in solution (Loehr et al., 1979a), is also a significant phosphorus retention mechanism. Precipitation reactions however, are much slower than adsorption reactions (Griffin and Jurinak, 1974; Sikora and Corey, 1976).

Soil Adsorption and Precipitation of Phosphorus

Fiskill et al. (1979) studied phosphate sorption kinetics on acid, sandy soil. Adsorption sites were associated with clay particles and iron and aluminum oxides. The movement of soluble phosphorus is described as a cromatographic process with mass transfer at any point being controlled by diffusional transport, sorption kinetics, or both. Batch samples indicated that adsorption over a seven day period was a non-linear, time dependent function. The rapid and then gradual removal of phosphorus from solution by the batch sample gave credence to a two-site sorption model where both rapid and slow reversible adsorption processes occurred. An important conclusion of their study is that the extent of phosphorus sorption from a flowing soil solution depends on the pore velocity of fluid. This infers that in order to optimize phosphorus retention, low hydraulic loadings should be practiced.

Griffin and Jurinak (1974) studied adsorption-desorption and precipitation reactions of phosphorous with calcite, a naturally occurring soil mineral, and developed a slightly different model. Adsorption of phosphorus was broken into two components: A rapid second order component occurring during the first ten minutes of contact and a slower first order component representing the surface rearrangement of phosphate ion clusters into calcium-phosphate heteronuclei. Adsorption was followed by calcium-phosphate crystal growth. The type of calcium-phosphate compound nucleated depended on the calcium to phosphorous ratio. Desorption of phosphorous consisted of two first order components. The first component, the dissolution of phosphorus mineral from the calcite surface, was found to significantly detract from the rapid adsorption process.

Novak and Petschauer (1979) studied orthophosphate adsorption kinetics onto Muskegon dune sand. Batch adsorption experiments showed rapid phosphorus removal followed by a slower reaction. Interaction with calcium minerals was suspected, because of the mineral composition of this sand and the time period of the rapid adsorption process. Calcium crystal growth took place from several days to two weeks. A three step model is described, based on three adsorption rate limiting mechanisms: interparticle mass transfer, intraparticle mass transfer and Langmuir type adsorption-desorption. An important concept that Novak and Petschauer (1979) use to describe soil column breakthrough characteristics is that as calcium phosphate minerals are formed on the particle surface, more vacant adsorption sites are provided so that more orthophosphate can be removed from solution. This may explain why soils generally show a greater capacity to remove phosphorus than is demonstrated by simple batch experiments alone.

Van Riemsdijk, Beek and DeHaan (1979) also describe a rapid adsorption process followed by a "long-term reaction" period for phosphorus reaction with aluminum hydroxide ($Al(OH)_3$). The long term reactions are surface reactions which may result in the ultimate formation of stable phosphate compounds. Column experiments, performed at pH 8, showed little phosphorus retention by quartz sand alone, but when aluminum hydroxide was added, greater than 97 percent total phosphorus removal was achieved. Chemical fractionation and scanning electron microscope observation showed that calcium-phosphate formation was not important.

In most soils, a similar process of rapid phosphorus adsorption followed by precipitate formation occurs, involving iron, aluminum and clay minerals as well as calcium, depending on pH and soil composition. The adsorption of phosphorus onto metal oxides may take minutes to days, the precipitation days to weeks (Beek and Van Riemsdjik, 1982). At acid pH, these metal oxides are commonly aluminum and iron. Aluminum appears to be of greater importance than iron in phosphorus adsorption. Vijayachandran and Harter (1974) review this topic across a range of soil types and suggest that past correlations between iron concentration and phosphorus adsorption are of localized significance only. In their study, the extractable aluminum concentrations from two particular procedures (pH 4.8 NH_4OAc and HCl-NaOH) correlated well with phosphorus adsorption over a range of soils. Kardos and Hook (1976) also stress the importance of metal oxides (such as the sesquioxides Fe_2O_3 and Al_2O_3) in phosphorus retention by soils. Phosphorus adsorbs onto exposed aluminum atoms on the edge surfaces of clay minerals depending on the number of reactive sites per edge face area, dimensions of the clay platelets and stoichiometry of the adsorption (Beek and Van Riemsdjik, 1982).

The importance of clay minerals to phosphorus adsorption is also described by Willman, Peterson and Fritton (1981). Soil columns of sand and sand-clay mixtures (zero to 12 percent clay) were evaluated in terms of their ability to renovate septic tank effluent. Sand only columns showed decreasing phosphorus removal capability over the 23 week study period. All columns with clay removed virtually all phosphorus. Probably due to the somewhat high phosphorus concentration in the applied septic tank effluent (approximately 20 mg/l total phosphorus), precipitation is cited as the predominant phosphorus retention mechanism, secondary to adsorption. It is again indicated that aluminum and iron, associated with the clay material, are very important to precipitation and adsorption reactions in acid conditions. The formation of calcium phosphates is indicated as the retention mechanism under alkaline conditions.

Magdoff and Keeney (1975) describe septic tank effluent phosphorus retention by sand, a silt loam and a calcerous sandy loam under anaerobic, 8 cm/day hydraulic loading. Phosphorus concentrations were greater, both before and after the experiment, in silt loam than sand. Retention on sand and silt loam was attributed to adsorption, and subsequent precipitation of calcium phosphate. Considerable calcium-bound phosphorus was found on the calcerous sandy loam. Approximately 50 percent total phosphorus removal is reported.

Anderson et al. (1981) describe the removal of phosphorus from secondary effluent applied to a soil-turf filter. Phosphorus removal improved as loading rates decreased. Sandy soils removed less phosphorus than mixed soils at the same application rates. This difference decreased with time. Decreased phosphorus removal efficiency was attributed to high loading rates and exhaustion of soil precipitants. Adsorption is not cited as a phosphorus removal mechanism.

Over long term applications, soils may retain a significant ability to retain phosphorus. Kardos and Hook (1976) review four land application sites receiving various sewage sludge loadings for nine to eleven years. All four sites (three on Hublersburg clay-loam and one on Morrison sandy-loam) showed sustained ability to remove phosphorus. Soils where crop uptake occurred showed better phosphorus removal but in no case did more than three percent of applied effluent phosphorus pass through 120 cm of unsaturated soil. The clay loam performed better than sandy-loam. Kao and Blancher (1973) report the ability of a Mexico silt-loam to adsorb phosphorus content had not decreased, although the total phosphorus content had doubled, after 82 years of phosphate fertilization. Various crops were grown on the soil during this period.

Adsorption reactions are significantly affected by pH. At pH values below seven, the oxide surfaces of soil particles are likely to be positively charged, enhancing chemisorption of anionic phosphorus forms (Bolt, 1976), most likely onto iron and aluminum surfaces (Sikora and Corey, 1976). Generally, phosphorus adsorption onto calcium surfaces occurs under alkaline conditions (Sikora and Corey, 1976).

The use of dolomite or calcite chips to remove phosphorus from wastewater was studied using soil columns by Sikora, Bent, Corey and Keeney (Sikora et al., 1976). Here, calcite chips or dolomite were placed below the clogging mat in an induced anaerobic environment. Anaerobyosis was induced by methanol addition to the dolomite or calcite. Denitrification, using methanol as a carbon source, was also intended to occur in this region. Calcite proved superior to dolomite for phosphorus removal, attributed to the presence of magnesium carbonates in the calcite. Excellent phosphorus removal was seen during the first month of operation but rapidly became insignificant. The decrease in phosphorus removal was attributed to organic anions in the effluent competing for sorption sites and microbial growth physically blocking sites. The use of calcite or dolomite for phosphorus removal in an aerobic environment below a clogging layer has not been evaluated.

The importance of organic material in soils to phosphorus retention has also been studied. The ability of organic soils to retain phosphorus varies widely (Nichols and Boelter, 1982). Stuanes (1982), reviewing phosphorous sorption in soils indicates that organic matter in soils may help sorption by sorbing phosphate or hinder it by blocking sorption sites on inorganic particles. Smyth and Lowry (1980) also point out this negative aspect. Vijayachandran and Harter (1975) review studies that found organic matter of importance in phosphorus removal. These studies attributed phosphorous removal to the presence of organically chelated iron and aluminum (Vijayachandran and Harter, 1975). Reneau and Pettry (1976) found significant NH_4F extractable phosphorus (signifying aluminum-phosphorus compounds; Peterson and Corey, 1966) near the site of septic tank effluent discharge to an organic coastal plain soil (Varina) and attributed this in part to anion exchange with organic material in the soil. It appears that the availability of aluminum is more important than the presence of organic matter to phosphorus removal.

The use of peat soils (high organic content) to remove phosphorus has been studied. Tilstra, Malueg and Larson (1972) review several studies of phosphorus adsorption by peat soils and conduct an analysis of a peat soil proposed as a phosphorus sink for Detroit Lakes, Minnesota, wastewater. Here laboratory data indicated that when the peat material was kept aerobic, excellent (95 to 99 percent removal) phosphorous fixation occurred. Field lysimeter performance in this study dropped during a four month trial (August to December) from 92 to 76 percent phosphorus removal. Phosphorus removal in the peat layer of a peat-sand filter was attributed to the high aluminum, iron and mineral content of the peat (Nichols and Boelter, 1982). Osborne (1975) reported almost complete total phosphorus removal in a peat filter treating secondary effluent and suggested that a grass crop was responsible for much of the phosphorus removal. Rock et al. (1982) studied the use of peat soil in an absorption bed receiving septic tank effluent. Approximately fifty percent phosphorous removal occurred over 3.5 years. The subsequent addition of a grass crop to the bed surface did not significantly increase phosphorous removal (Rock, 1983). If aerobic conditions are maintained, peat material is useful for phosphorus removal.

The long term effects of treating septic tank effluent with a highly organic soil such as peat are not clear. Rock et al. (1982) report a deterioration of peat cell opening size after treating such waste and indicate that under anaerobic conditions, peat may be utilized as a carbon source for denitrification, accelerating decomposition of the peat bed.

The hydraulic application rate is very important to phosphorus retention by soil. Hydraulic loadings that maintain unsaturated, aerobic conditions are desirable. During unsaturated conditions, because of capillary forces and the formation of air spaces in the middle of pores, fluid is forced in a very irregular, more tortuous path through the soil matrix than during saturated conditions (Brutsaert, Hedstrom and McNeice, 1980; Smyth and Lowry, 1980). As the degree of soil saturation decreases, phosphorus retention increases due to increased contact time, viscosity of fluid and tortuousity of the flow path (Brutsaert, Hedstrom and McNeice, 1980). During saturated flow, capillary forces are minimal (Brutsaert, Hedstrom and McNeice, 1980) and a large percentage of the fluid flows rapidly through the largest soil pores (Smyth and Lowry, 1980). By increasing the contact period between effluent and the soil particle surface, adsorption and precipitation are more likely. Maintaining positive redox conditions (aerobic) is also important to retaining adsorbed and precipitated phosphorus. Under reducing conditions (anaerobic), much of iron associated phosphorus in the soil is released to the soil solution, establishing a new equilibrium with aluminum and calcium bound phosphorus (Sikora and Corey, 1976).

In summary, phosphorus retention by soil is a function of many variables. Minerology of the soil, particularly the presence of available iron and aluminum in acidic soils and calcium in alkaline soils, is important. Coarse soils, with less surface area for adsorption (Gilliom and Patmont, 1983), remove phosphorus less efficiently than finer grained soils. Most importantly, the hydraulic application rate should be low enough to maintain unsaturated, aerobic conditions.

Table 5 summarizes site and soil qualities important to on-site phosphorus retention.

Table Five

Site and Soil Properties Important to Phosphorus Retention

1). Unsaturated Soil:
Maintain Aerobyosis.
Preferable Flow Characteristics.

2). High Sesquioxide Content:
Provide Aluminum and Iron Oxides Necessary for Adsorption and Precipitation Reactions.

3). Calcium Minerals:
Necessary for Adsorption and Precipitation in High pH Soils.

4). Small Grain Size:
Provide Reactive Sites.
Induce Capillary Retention of Fluid.

5). Contact Time:
Allow Reactions to Occur.

6). Clay minerals:
Can Provide Both Grain Size and Sesquioxide Requirements.

7). Organic Materials:
Important Only in Their Ability to Provide Aluminum and Iron.

CHAPTER 6

Alternative Collection Systems

A. Rationale

The past three chapters have described on-site treatment systems that are very dependent on site soil and hydrogeologic characteristics. And while this report indicates that many more sites than are currently deemed suitable for absorption field construction can accept and treat wastewater, there still will be situations where construction of on-site soil systems is impractical. It becomes necessary in such situations to convey wastewater (sewerage) to a more suitable disposal site.

Conventional sewerage systems rely on gravity and, occasionally, pumping stations to convey sewage to a treatment facility. Since gravity flow will most likely be towards the shoreline at a lakefront community (U. S. EPA, 1977d), conventional sewerage technology would require that the collection main be placed close to the shoreline. Construction of sewer mains in and along a lake shoreline would be difficult (due to high groundwater elevation) and potentially harmful to the local environment. Geological characteristics such as the presence of boulders or shallow depth to ledge would further impede construction, increasing the cost of a collection system.

At rural lakefront communities, conventional sewerage may not be practical. Because of low housing densities and difficult terrain, such a system may impose an excessive financial burden on homeowners. Where it is desirable to remove sewage from the property, a system that can overcome the difficulties inherent to lakefront locations at reasonable cost is needed.

This chapter describes three systems that are viable alternatives to conventional sewerage. In fact, these systems may be more cost effective than conventional sewerage systems in both large and small flow applications. The first two, pressure and vacuum sewerage systems rely on an artificially increased pressure differential to convey sewage. The third, small diameter gravity sewers, relies on a pretreatment step to remove the minimum flow velocity requirement constraining conventional sewers. Significant construction cost savings are possible with all of these systems.

B. Pressure Sewerage Systems

A pressure sewer system simply conveys sewage as a result of an artificially increased energy grade line. The increase in energy is provided by a pump, imparting energy either by spinning a fluid mass (centrifugal pump), or imparting force directly to the fluid (pneumatic ejector or positive displacement pump). In these systems, each home, or cluster of homes, is equipped with a pumping facility. Sewage is transported up gradient to a more suitable location, perhaps to a gravity flow main or treatment location. The major advantage of pressure sewer systems is that they are not restricted by line and grade as conventional sewerage systems are.

There are three distinct types of pressure sewer systems. One employs a pneumatic ejector to raise raw sewage or septic tank effluent to a gravity sewer or treatment location (Clift, 1968; U. S. EPA, 1977d). A second system, known as the grinder pump (GP) system, grinds raw sewage to a slurry, then pressurizes it for conveyance. Only the third system requires wastewater pretreatment before pressurization. A septic tank or similar apparatus removes solid material and grease from wastewater before pumping. This system is referred to as the septic tank effluent pumping (STEP) system.

Pneumatic Ejectors

Clift (1968) reviews the construction and three years of operation of a pneumatic ejector system serving 42 homes in Radcliff, Kentucky. One half or one horsepower motors moved raw sewage at 15 gpm against 20 or 35 feet total dynamic head. Three inch house laterals and a four inch main discharged the sewage to a gravity sewer. Here, mechanical and electrical failures were often attributed to corrosion and were directly proportional to the dynamic head the pump was required to overcome. Although no pretreatment of sewage occurred before pumping, clogging of pump or discharge piping apparently was not a problem in these applications. A critical restriction was the low head capabilities of these pneumatic ejector pumps. Currently, the CLOW Corporation (Florence, KY), Ecodyne Corporation and Franklin Research Company manufacture pneumatic ejector pumps (U. S. EPA, 1977d; Benjes and Foster, 1976). A cycle of vacuum and compressed air impart a force on the fluid, forcing it along the conveyance piping. Pneumatic ejectors are also used in package pump and treatment plant applications (CLOW, 1983a).

Grinder Pumps

A similar system, in that it pressurizes essentially raw sewage, is the grinder pump system. This system is probably the most common pressure sewer system in practice. Certainly more information is available for the GP system than any other low pressure sewer system (U. S. EPA, 1977d).

Several demonstration projects of GP systems have been reviewed (Carcich, Farrell and Hetling, 1972; Gray, 1975; Williams, 1975; U. S. EPA, 1977d; Milnes and Smith, 1978; McDowell, Beekman and Goldman, 1979), all of which show generally acceptable operational and maintenance characteristics. A potential problem, noted at two GP systems, in Pheonixville, Pennsylvania, and Albany, New York, was the accumulation of grease and fibrous materials along pipe walls, reducing cross sectional areas by as much as 40 percent (U. S. EPA, 1977d). Further study of this problem is warranted. Manufacturer GP information is available from several firms (U. S. EPA, 1977d); locally from the Environment One Corporation (Schenectady, New York) which manufactures and markets a series of grinder pumps suitable for residential and cluster applications (Environment One, 1973; 1978).

The characteristics of GP sewage conveyed by pressure collection systems will probably exhibit slightly higher BOD, SS and nutrient concentrations than municipal sewage, owing to a lack of infiltration/inflow into pressurized systems and grinding. An Albany, New York, GP system had average wastewater characteristics of 330 mg/l BOD5, 855 mg/l COD, 310 mg/l TSS, 80 mg/l TKN, 15.9 mg/l TP and 81 mg/l grease. (The reader should consult chapter two or the appendix of this report for information regarding wastewater pollutant parameters.) Also noted at Albany was that grinding may produce sewage with generally finer solids (U. S. EPA, 1977d). The effect this may have on primary sedimentation processes is not clear.

Grinder pump systems employ motors of one half to one horsepower to drive the grinder and pumping units, usually constructed as an integral unit. The grinder impeller should be constructed of hardened, corrosion resistant material. Farrell (1972) states that a one horsepower Environment One grinder pump is capable of grinding foreign objects occasionally found in sewage such as wood, plastic, and rubber to a fine slurry.

The pumping unit is often of progressing cavity design (called semi-positive displacement by the Environment One Corporation). Progressing cavity pumps are often used for transporting sewage sludges for they offer high head capabilities (50 psig) without clogging (Benjes and Foster, 1976). Their head-flow characteristic curve is steep, exhibiting very little change in flow as the total dynamic head load changes (Farrell, 1972). This may be desirable in situations where dynamic head loads vary greatly during pump operation.

Grinder pump installations are normally constructed of corrosion resistant materials and valved to prevent backflow of sewage to the home (Environment One, 1973). Small (1.25 inch) diameter piping often serves as the house lateral to the collection main. The collection main is also small, usually less than four inches in diameter, depending on the number of homes served. The use of small diameter piping to serve a given hydraulic load increases system dynamic head load and requires more rapid flow velocities than a larger diameter pipeline would require. Rapid flow velocities will scour and keep clean pipe walls. To prevent clogging, gravity sewers that receive ground sewage must also be designed to maintain scouring velocities.

Little information is available on the long term performance of GP systems. The U. S. EPA (1977d), reviewing several GP installations, indicates that start up problems with sensing devices and electrical components should be expected. A U. S. EPA sponsored GP installation at Grandview Lake, Indiana, tested three types of commercial GP units during 1974. Design problems such as excessive pressure, loss of pressure, excessive wear, valve failure, air in pressure lines, and overloaded units were reported in all three types of GP units causing frequent and often difficult service calls. Reportedly, two of the three pumps were modified by their manufacturer shortly after this experience (U. S. EPA, 1977d). A private consulting firm proposing a GP sewer system to serve 27 homes at Lake Thompson, Massachusetts, estimates grinder pump core replacement every 10 years and pump stator replacement every three years (Tighe and Bond, 1979).

Power consumption of GP units was studied at Pheonixville, Pennsylvania, and Albany, New York, projects. Approximately 0.8 watt-hours per gallon of sewage can be expected (U. S. EPA, 1977d). For a family of four, generating 65 gpcd near Amherst, Massachusetts (power cost = 0.088 dollars per kilowatt hour; including 0.03 dollar fuel adjustment charge; Bean, 1983), the resulting annual power cost is $6.68.

<u>STEP Pressure Sewers</u>

The STEP system, although not as extensively researched as GP systems, may provide a viable means of sewage conveyance at less cost than GP systems. The initial cost of a STEP pressure sewer system is increased by its pretreatment requirement. A baffled, 1000 gallon, single compartment septic tank will cost approximately 250 to 300 dollars (based on telephone quotes, July, 1983; River Rd. Excavating, Hadley, MA and Northfield Concrete, Northfield, MA). A two compartment septic tank would most likely be slightly more expensive. The STEP system also requires a wet well after the septic tank for the pump unit. The cost of a STEP pumping unit however, is significantly less than grinder pumps. Submersible sump pumps, modified with non-corrosive impellers are often used in STEP systems (U. S. EPA, 1977d). The U. S. EPA (1977d) estimates that 200 dollars will purchase a submersible sump pump suitable for residential applications. In comparison, a basic Environment One grinder pump (model GP210), suitable for basement installation, including 60 gallon tank and on/off sensing device costs approximately $1900 (based on telephone quote from distributor of Env. One products: F. R. Mahoney Associates, Hingham, MA; October, 1983). The Lake Thompson analysis (Tighe and Bond, 1979) estimated $2500 for a similar item including placement outside the home. Figure 5 shows a STEP system schematic.

General Pressure Sewer Design Information

In all types of pressure sewage system design, the potential effects of exfiltration of sewage should be considered. Exfiltration may result from maintaining a sewer main at higher pressure than its surrounding soil.

Access to pump units should be made available by installation either in a home's basement or a manhole constructed outside the home. If constructed, the manhole should be placed close to the home to avoid power line voltage drops and to decrease the cost of the gravity sewer (conveying flow from the home to manhole). High water alarms should be conspicuously placed in the home, so that wastewater generation can be stopped in the event of a power or pump failure.

It may be desirable to provide a backup disposal process if the reservoir capacity of the GP is small. In a power failure event, homeowners receiving municipal water would most likely still be capable of generating a significant quantity of wastewater. A homeowner with an electric water pump would probably only generate wastewater, in a power failure event, comparable to the amount of water remaining in the homeowner's piping and hot water reservoir. Where a failed septic tank - soil absorption system (ST-SAS) is being replaced with a GP system, an overflow connection to the failed ST-SAS can provide temporary wastewater disposal. A septic tank alone may provide sufficient wastewater storage while GP system repairs are made.

Design of pressure sewer systems in rural areas should conform to the available power supply. While in most cases this will not present a problem, it is conceivable that voltage drops occurring along power supply lines may be significant enough to require a transformer before the pumping unit. Operation of a motor at less than its rated voltage causes overheating and decreased motor life. Also of concern is the type of pump motor. In locations where flow or hydraulic head require that the pump motor be several horsepower (or more) the engineer should be certain that the motor is capable of operating on single phase power (normally provided to residential units). As motor horsepower rating increases, the probability that the motor requires two or three phase power increases. Two and three phase motors operate with less vibration and may be less expensive than single phase motors. If necessary, a single phase motor can be used to drive a two or three phase power generator, in turn driving the pump motor. The reverse problem might occur at an industrial location served by two or three phase power (commonly at 208 volts). In this case the two or three phase power can be split to provide single phase power but a transformer would be required to increase line voltage to the 240 volts required by some pumping units (Solomon, 1983). In every case it seems that if power is delivered to a home, it is possible to operate a motor of some type so that grinding and pumping may occur. The added

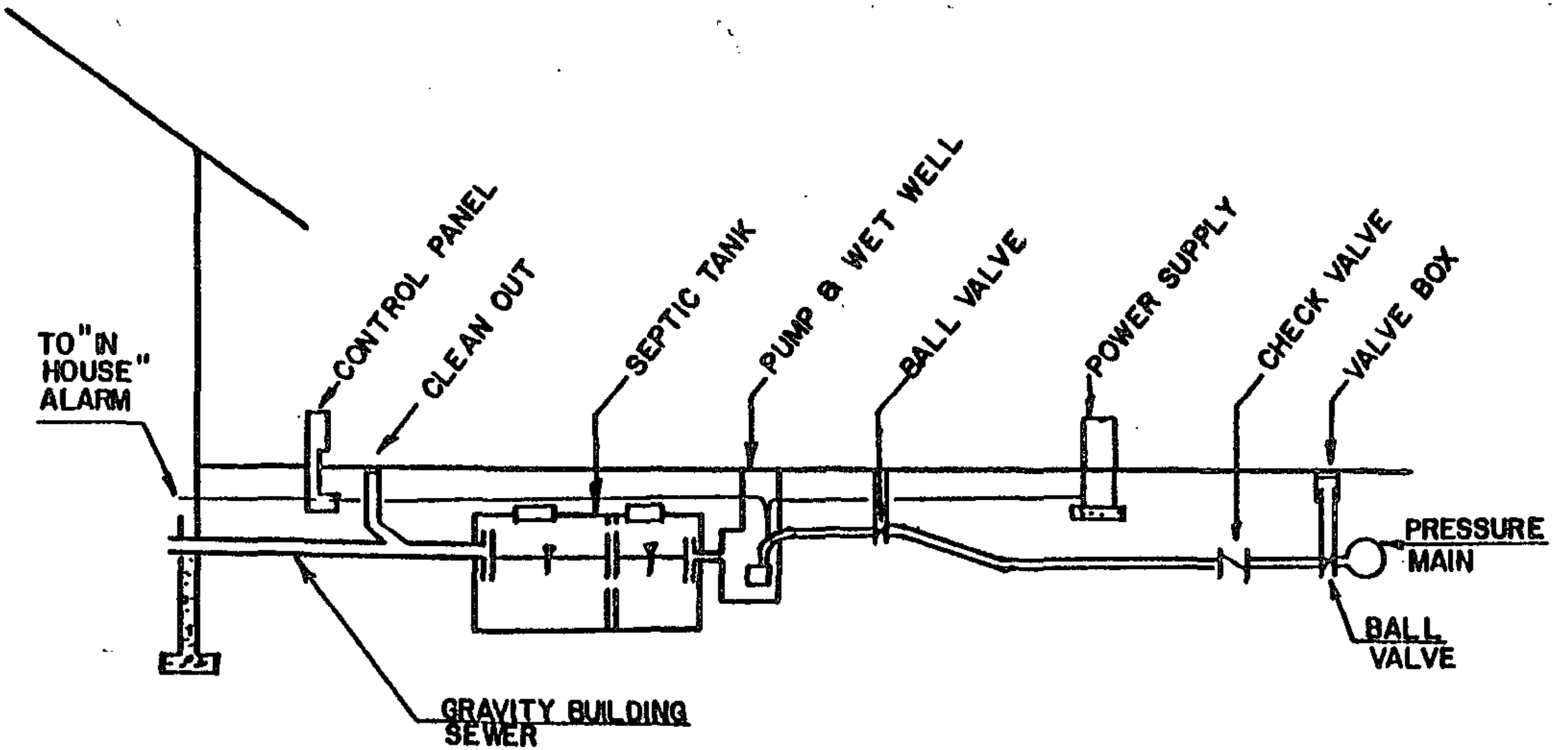


FIGURE 5 : SCHEMATIC OF "STEP" PRESSURE SEWER ON-LOT FACILITIES INSTALLATION. NO SCALE

(Adapted from Tollefson and Kelly, 1983) pipe, SDR-21 or schedule 40, may be desirable in some applications. During construction, pipe installation should be monitored as improper construction techniques have led to leaks and reduced pipe strength (Williams, 1975; U. S. EPA, 1977d).

The actual design of the collection system (pipe sizing, dynamic head estimates) is beyond the scope of this project. General concerns during design should be to prevent backflow to any home, ensure adequate pump capacity - even when several pumps in a branch are operating, and provide reliable operation. Similar to water distribution systems, thrust blocks must be used at changes in flow direction. The U. S. EPA (1977d) generally reviews collection system design. Tollefson and Kelly (1983) provide general information on the use of a computer model (identifying nodes, pipes and demands) to design pipe networks. Flanigan and Cadmik (1979) review some basic head loss equations (Darcy-Weisbach and Hazen-Williams), the effect of pressure system appurtenances on flow and describes a simple case of multiple pump operation. Further information would be available in hydraulics texts and from pump manufacturers.

Administrative concerns in a community pressure sewer system would be to make available emergency service and perhaps backup pumping units for the system. In some applications, a hybrid pressure-gravity collection network may be the most economical design, although any criteria used for allocating the operational and construction costs of such a collection system would be subject to debate.

C. Vacuum Sewerage Systems

For reasons similar to pressure sewer systems, vacuum sewer systems may also be a viable alternative to conventional sewerage systems in rural lakefront communities. Both vacuum and pressure sewer systems rely on an artificially induced pressure differential to move sewage. In pressure sewer systems a pump imparts a force "behind" the sewage to move it to areas of lower pressure, namely along the sewer main. In vacuum sewer systems a vacuum pump lowers the pressure in the sewer main, inducing a mixture of sewage and air to travel along it. Figure 6 shows a general vacuum sewerage system schematic.

Vacuum systems are mechanically more simple and in some cases less expensive to install and operate than pressure collection systems (King, 1981). Pressure collection systems require that each home, or cluster of homes, own and maintain a pumping unit. Vacuum systems rely on a central pumping station to create vacuum in collection pipes. Each home, or cluster of homes, in a vacuum system must have a wet well and interface valve (separating the vacuum system from the sewage at atmospheric pressure) to periodically introduce air and sewage into the collection system.

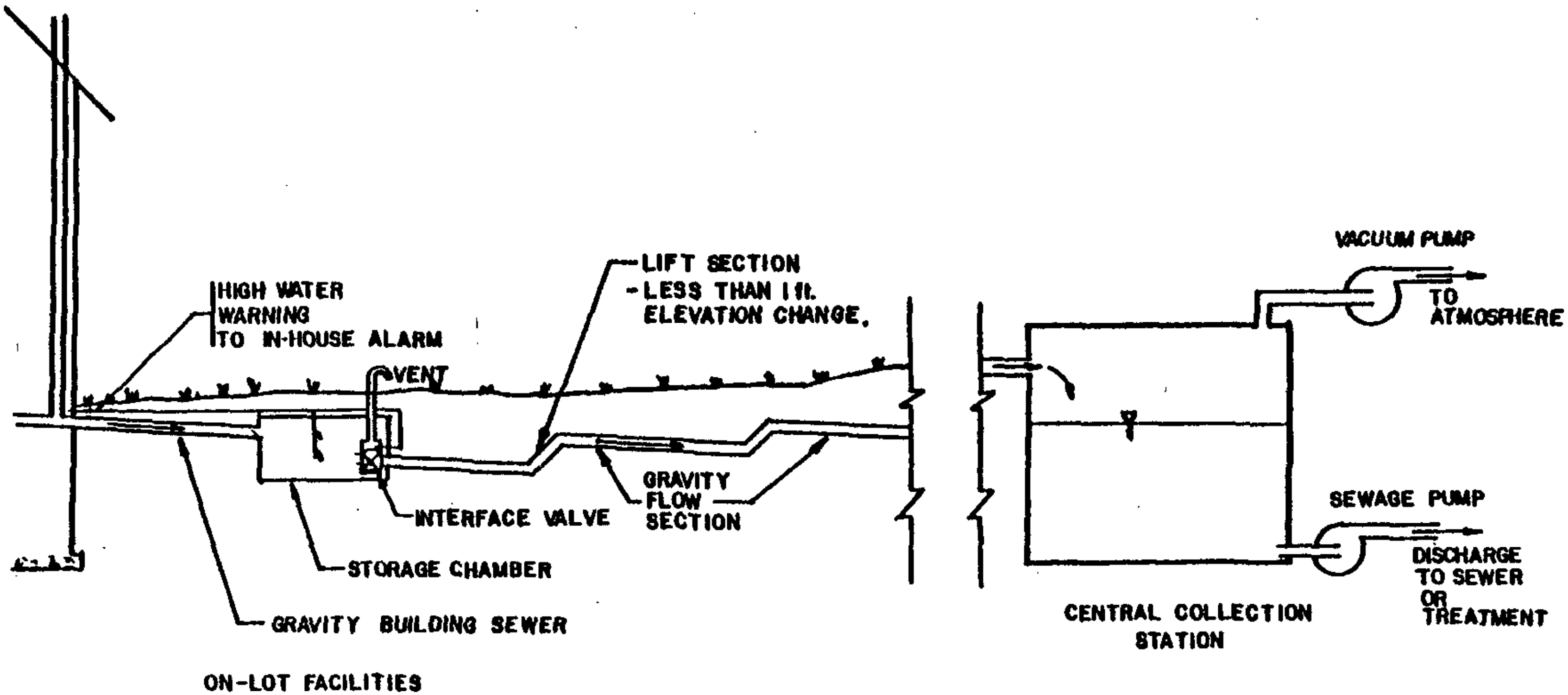


FIGURE 6: VACUUM SEWERAGE SYSTEM SCHEMATIC

NO SCALE

Vacuum wastewater collection systems are currently used aboard large ships, at military bases, and at several residential locations in the United States. Previous vacuum sewer systems in the Bahamas and some United States locations have performed poorly due to hydraulic overloading, improper assessment of vacuum lift requirements, and solids deposition within the collection mains (Skillman, 1979). Currently, there is a lack of good information on system design criteria, performance of existing systems, and the behavior of sewage in vacuum collection systems. Historically, designers have compensated for this lack of knowledge by "overdesigning" systems, transporting small amounts of sewage by using large quantities of air (Skillman, 1979).

Vacuum collection systems are particularly attractive where groundwater contamination, due to sewer system exfiltration, is of concern. If collection mains remain in suction, exfiltration of sewage should not occur. Infiltration of groundwater however, is encouraged by maintaining negative pressures in the collection system.

Construction of vacuum sewer systems is relatively simple: manholes are not required and generally, the system can be routed around any obstacles that may be discovered during installation (Foreman, 1982). The depth of pipe need only be sufficient to prevent damage from overpassing vehicles and freezing in cold climates.

Application of vacuum collection systems is restricted by the vapor pressure of the fluid being transported. Vapor pressure is the pressure of a vapor in equilibrium with a solid or liquid at a given temperature (Sears, Zemansky and Young, 1976). When fluids are exposed to local pressure at or below the vapor pressure of that fluid (as might be induced by suction), boiling of the fluid occurs (Vennard and Street, 1976). When fluid turns to a vapor in this manner, energy requirements for transmittance increase and solid materials become separated. Transmittance is no longer practical. Because the predominant fluid in sewage is water, for now we can assume that sewage behaves like water with respect to suction limits. (The actual collection system design, particularly transport velocity, will consider the effect of impurities present in sewage.)

Vapor pressure of water increases with temperature. At 32 degrees Farenheit, the vapor pressure of water is 0.09 pounds per square inch (psi). At 212 degrees Farenheit, 14.7 psi (which happens to be the average normal atmospheric pressure at sea level) is the vapor pressure. In other words, if the local atmospheric pressure is less than 14.7 psi (29.93 inches of mercury), water at 212 degrees Farenheit will vaporize. At 70 degrees Farenheit, probably the highest temperature that would be expected for domestic sewage, the vapor pressure is 0.36 psi (Clark, Viessman and Hammer, 1977). Assuming that atmospheric pressure at the collection system location never drops below 13.75 psi (28 inches of mercury), we are left with (at 70 degrees

Farenheit) 13.39 psi that our suction system can induce without vaporizing the fluid. This corresponds to 30.9 feet of water at 70 degrees Farenheit (water density = 62.30 lb/ft^3; Clark, Viessman and Hammer, 1977). From this 30.9 feet, frictional headlosses and a reasonable safety factor must be subtracted to determine the practical static head the collection system may overcome. Frictional headloss will vary with fluid velocity, type of pipe, fittings and length of travel. A reasonable safety factor is four to five feet. In practice, this leaves approximately 20 feet of static head that the system may overcome. This is the reason then, why vacuum sewer systems are generally only efficient on flat or gently rolling areas (Johnson, 1978). In order to raise sewage over this practical limit, subsequent vacuum stations and wet wells at atmospheric pressure must be constructed.

The three main components of a vacuum sewerage system are the interface valve, the collection main and the central collection station (figure 6 shows a vacuum sewerage system schematic). When sufficient sewage and vacuum are present, the interface valve opens, allowing a predetermined quantity of air and sewage to enter the main. Atmospheric air expands as it enters the system, driving wastewater forward (Hassett and Starnes, 1981). Approximately 80 percent of the expansion will be towards the collection station (Hassett and Starnes, 1981). Introduction of air also increases the power requirements required to create and maintain a vacuum in the collection system (Skillman, 1979). Design and operation of a sewage collection system without air is impractical because of air leakage into the system, gases that evolve when a fluid is exposed to a vacuum (Skillman, 1979) and the aid that air provides in maintaining scouring velocities in the pipeline.

Collection mains are usually three to six inch diameter, PVC (polyvinyl chloride) or ABS (acrylonitrile butadiene styrene) piping (Foreman, 1982). Plastic pipe materials are often preferable because of their weight, available fittings, and speed of assembly. Vacuum collection mains are constructed in a sawtooth profile so that reliable transportation of the sewage occurs.

The sawtooth profile results from the behavior of sewage in a vacuum pipeline. Skillman (1979) analyzed flow through a vacuum system constructed of clear PVC piping. First, the interface valve opens, allowing a slug of sewage followed by atmospheric air to enter the main. Due to expanding air, friction along the pipe wall, and the inability of the fluid to support significant shear forces, the slug rapidly disintegrates. The slug becomes a swirling annular flow (fluid along the pipe wall and gas in the center of the pipe) and then disintegrates further to a mist. Slug deformation allows air to flow around and through the slug. During deformation, wastewater velocity decreases and mist particles begin to settle. The mist droplets collect at the bottom of the pipeline and travel downslope (via gravity flow) to the next lift in elevation.

These lifts should change elevation at most twelve inches and generally, should be constructed at least every 500 feet to minimize excavation costs (Foreman, 1982). Elevation lifts should be constructed of 45 degree bends connected by a piece of sewer main. At the lifts, wastewater collects and the momentum of wastewater and air, introduced from subsequent openings of interface valves, carries the previously disintegrated slugs over the lifts (Hassett and Starnes, 1981).

Previously, the operational concept of wastewater in a vacuum collection system was that wastewater collecting at the lifts reformed a slug which would be lifted the next instance an interface valve opened or a sufficient pressure differential (before and after the slug) developed. These lifts would have been constructed of 90 degree bends connected by a piece of sewer main. By this sequence, wastewater would eventually travel to the collection station. The current concept indicates that air flows above the liquid throughout the the pipeline, maintaining a high vacuum condition throughout (Hassett and Starnes, 1981).

The collection main profile should be constructed to maintain gravity wastewater flow velocity at greater than 2.0 and less than 10 feet per second. The minimum velocity requirement has traditionally been used to ensure that solid materials remain suspended in the wastewater. The maximum velocity requirement prevents structural damage to the pipe from scouring. Skillman (1979) recommends a minimum flow velocity of 3.5 feet per second. This has been cited as a sufficient velocity to ensure that entrained or trapped gases will not collect above the fluid, forming in effect, an air blocked pipeline (Skillman, 1979). This would require however, full pipe flow, which is unlikely in a system that purposely introduces air and is designed to have air flowing above or through the liquid throughout the pipeline.

The central collection station consists of a vacuum reservoir, vacuum pump and wastewater discharge pump. The vacuum reservoir collects wastewater, connects to both pumps, and reduces the frequency of vacuum pump cycling. The vacuum pump essentially gathers low pressure air, compresses it to atmospheric pressure, and discharges it to the atmosphere (Skillman, 1979). The wastewater discharge pump removes accumulated wastewater and discharges it to a treatment facility. It is conceivable that gravity flow could deliver wastewater from the collection station to the treatment facility, negating the need for a wastewater discharge pump.

A potential problem of vacuum collection systems lies in the effect of collection piping leaks on central collection station and collection piping performance. As air or groundwater leaks into collection mains, the pressure inside the main increases, causing the central collection station to operate more often than expected. Also, as air or fluid leaks into the main, the pressure differential within the main decreases, thereby decreasing wastewater flow velocities (Skillman, 1979). This may result in deposition of solids and eventual clogging of the main.

Maintenance of vacuum sewerage systems consists of daily checks on vacuum and sewage pumps and weekly checks on standby power and alarm systems (Foreman, 1982). Foreman (1982) suggests that every six years each interface valve be overhauled and adjusted for proper operation.

Regarding the cost to construct a vacuum sewerage system, Hassett and Starnes (1981) estimate that the vacuum valve assembly and holding tank costs 1,427 dollars installed, based on bid prices (August, 1979) for a vacuum collection system employing approximately 1,000 of these units (located in Queen Anne's County, Maryland).

The cost of operating a central collection station has not been reported but can be estimated from the power requirements of motors employed and their frequency of operation in a collection station. The air to liquid ratio represents a major influence on the operational energy requirements (Skillman, 1979; Hassett and Starnes, 1981). Air to liquid ratios from 1:1 to 4:1 are typical in current system design (Hassett and Starnes, 1981). Skillman (1979) reports a linear increase of power required to transport wastewater with increases in the air to liquid ratio.

There are currently several companies that manufacture and sell vacuum wastewater collection systems. They are: Envirovac Division Dometic Inc., Jered Brown Inc., Mansfield Inc., Vacu-Tech Inc. and Airvac Division of Burton Mechanical Contractors Inc. (Foreman, 1982). These manufacturers will provide design criteria in addition to that which is currently available in the literature. As mentioned previously, owing to a general lack of knowledge about these systems, system design is currently very conservative. As more research is completed on vacuum sewerage systems, their applicability and usage are likely to increase.

D. Small Diameter Gravity Sewers

The last alternative sewerage system considered here, small diameter gravity sewers (SDGS), provides an alternative to conventional gravity sewers without requiring an outside energy source to artificially increase the pressure differential between the generator and the disposal site. Gravity provides the energy necessary to transport sewage. As such, a net negative gradient must exist between generator and disposal site. As described below, their advantages over conventional gravity sewers involve construction cost savings due to both materials and methods, and their ability to be constructed close to the ground surface, even in terrain of varying topography.

Conventional gravity sewerage system design requires that wastewater flow velocity be maintained at more than two feet per second (scouring velocity) to provide sufficient turbulence in wastewater so that solid materials remain suspended and greasy materials do not accumulate along the flow path. By preventing deposition of solids and accumulation of grease, clogging of the sewer main is (hopefully) avoided. Conventional design also requires that flow velocities not exceed 10 feet per second, as speeds in excess of this may cause structural damage to the pipe due to potentially abrasive action of solid materials in wastewater at these speeds.

Small diameter gravity sewer design does not require a minimum or maximum flow velocity (Otis, 1982b). By removing solid materials and grease before wastewater enters the conveyance system, concern of clogging or structural damage is essentially unnecessary.

It is necessary that each home or cluster of homes have a pretreatment facility for SDGS implementation.

Removal of solids and grease may be provided by filters, an Imhoff tank, or most commonly, a septic tank. Chapter three of this report reviewed septic tank design, performance and operation. From this chapter, the reader may recall some characteristics of septic tank effluent (presented in Table 3) and general information about septic tank design. In short, a septic tank's primary purpose is sedimentation and as such, it should be designed to prevent short circuiting, turbulent flow and provide storage for accumulated materials. Figure 2 shows a two compartment septic tank suitable for serving a three bedroom residence. Secondary to settling performance is anaerobic digestion. Anaerobic digestion degrades the carbonaceous component of wastewater and also, "markedly changes the characteristics of solid materials" in wastewater (Ludwig, 1978). Certainly septic tanks do not remove all solid materials from wastewater but the small, discrete, non-gelatinous, solid materials present in septic tank effluent are much less likely to induce clogging than the gummy-gelatinous solids found in raw sewage (Nottingham and Ludwig, 1948; Ludwig, 1950; Ludwig, 1978).

Another advantage of wastewater pretreatment before discharge to sewers is the flow equalization that the pretreatment process may provide. Attenuation of peak flows allows implementation of sewer mains of smaller diameter than conventional systems. In fact, Simmons et al. (1982) suggest that septic tanks used in SDGS systems be modified specifically to attenuate peak flows.

Small diameter gravity sewers have been operating successfully in Australia since 1961 (Otis, 1982b) and in the United States since 1975 (Simmons et al., 1982). Unpublished information obtained from a Springfield, Massachusetts consulting firm that is familiar with small flow technology indicates that twenty two small diameter gravity sewer systems were either under construction or in design as of October, 1982 in New York State (Ward, 1983).

There are two variations of SDGS systems. A more progressive design, known as the variable grade sewer (VGS) design, has been in use (quite successfully) at Mt. Andrew, Alabama since 1975. Sewer mains in this system are designed along the system's hydraulic grade line, allowing pipe sections to be laid at negative, flat, and positive slopes. A more conservative design is that practiced in Australia, and several locations in the United States. This system requires a minimum flow velocity (although not as fast as conventional sewer design) and larger diameter pipes than the VGS design. To maintain a minimum flow velocity, sewer mains must always be lain at a minimum negative slope, often requiring greater depth of construction.

Cost savings over conventional sewer systems can be achieved with both SDGS variations. However, because VGS systems can reliably transport sewage and be constructed at lower cost than the more conservative design, VGS systems are preferable.

Small diameter gravity sewers may be particularly suitable at lakefront communities. Because limited excavation is required to place SDGS lines, it may be practical to locate collection lines along the lake shoreline. In fact, it may be possible to set SDGS lines in the lake bottom. If these locations are not practical, placement along the lake's perimeter road (should one exist) will still most likely be less expensive than conventional gravity sewers.

SDGS Materials and Construction

The main impetus for implementing SDGS systems is cost savings. System cost is increased by its pretreatment requirement. As mentioned previously in this chapter (see STEP pressure sewers), a single compartment, 1,000 gallon septic tank will cost approximately 250 to 300 dollars. A more efficient and reliable two compartment tank will likely cost more. (A designer may be able to take advantage of existing septic tanks, further increasing cost savings, where SDGS systems are proposed to replace failing ST-SA systems.) A gravity sewer conveys sewage from the building to the septic tank or other pretreatment facility.

Simmons et al. (1982) recommend that septic tanks be modified to attenuate peak flows. This attenuation is accomplished by providing surge storage in the second compartment of the basin which drains into the effluent sewer through a 3/16 inch diameter hole in the base of a two inch diameter standpipe. Overflow relief is also provided. Figure 7 shows their recommended design. The septic tanks used in their study (somewhat similar to their recommended design) were inspected after 18 months of operation. No clogging of clarifier tubes was noticed, but treatment performance provided by these tanks was no better, and occasionally worse, than conventional single compartment septic tanks. The poor performance was attributed to too small hydraulic capacity in the first chamber. Despite this poor pretreatment performance, the VGS system has performed successfully, at least through its five years of reported operation (Simmons et al., 1982; Simmons and Newman, 1982). The improved tank design utilizes the second compartment to store surge flows and a small orafice in the effluent piping to limit the rate of septic tank effluent flow into the sewer main.

Significant material cost savings can be realized after the septic tank. Pipe diameters become much less than the four inch house laterals and eight inch minimum diameter sewer main lines employed in conventional sewers. (These diameters are often specified to conform to readily available cleaning equipment and provide ventilation above flowing wastewater -- not necessarily to meet hydraulic requirements.) Otis (1982b) reports that small diameter sewer mains should be sized to accommodate peak flows while flowing full. However, based upon reported reliable Australian experience and the availability of low cost cleaning equipment (not hydraulic criteria), Otis (1982b) recommends four inch minimum diameter piping. Simmons et al. (1982) and Simmons and Newman (1982) report on five years of successful operation of a system employing two and three inch diameter mains serving 31 homes in Mt. Andrew, Alabama. Both reports recommend a minimum 1.5 inch diameter house lateral and two inch diameter main.

Sewer appurtenances become more simple in SDGS systems, also providing significant material savings. Manholes, installed in conventional sewers at least every 350 feet and at all changes in flow direction, to provide access for cleaning and maintenance are unnecessary in SDGS sytems. "Clean-outs," a simple extension of the sewer main to the ground surface, are provided instead. Figure 8 shows a clean-out schematic. Otis (1982b) recommends that clean-outs be placed at every intersection of four or more lines, at intervals of 750 feet where minimum gradients occur, and at intersections of two lines at depth greater than 7.5 feet. Clean-outs allow small sewer rods to be pushed through any clogs that develop. Besides cost savings, clean-outs are suggested in place of manholes because manholes can be a source of undesirable grit, debris and inflow into sewer lines (Otis, 1982b).

With the VGS design, it may be necessary to provide mainline vents before and after constantly filled (full flow) sections. These vents will maintain atmospheric pressure in open channel flow regions and hence, prevent gas buildup which may preclude sewage flow. These vents may simply be extensions of the sewer main, open to the atmosphere and raised above the hydraulic grade line. In some cases, ventilation through house roof vents will be sufficient.

In some cases, the designer may find it prudent to place back flow prevention devices along house laterals. This would prevent

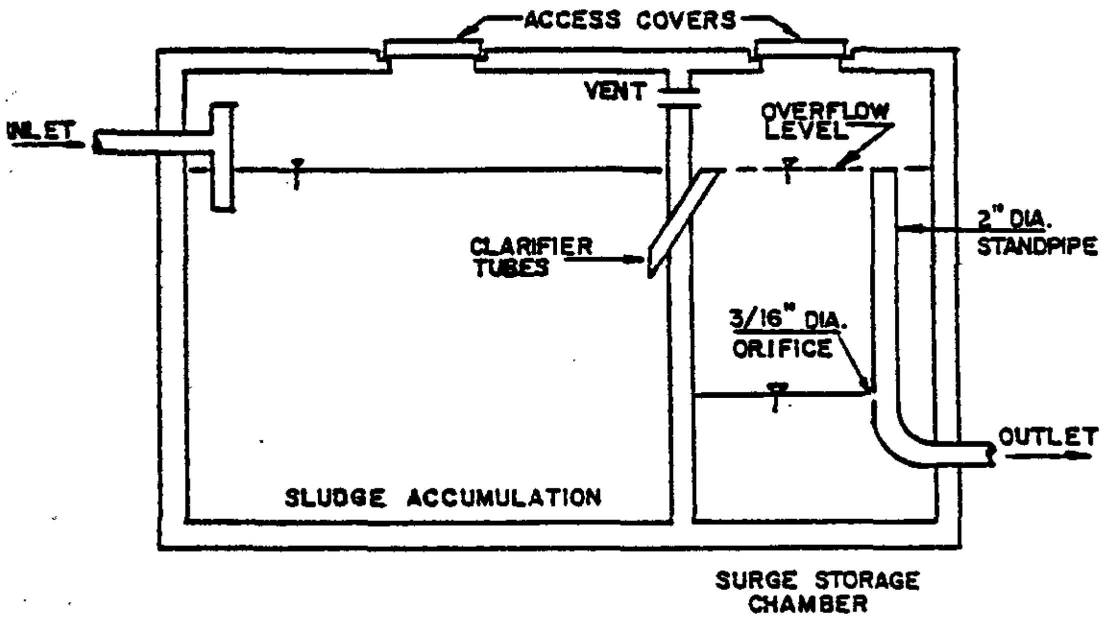


FIGURE 7 : SEPTIC TANK DESIGNED TO ATTENUATE PEAK FLOWS

(SIMMONS et al., 1982)

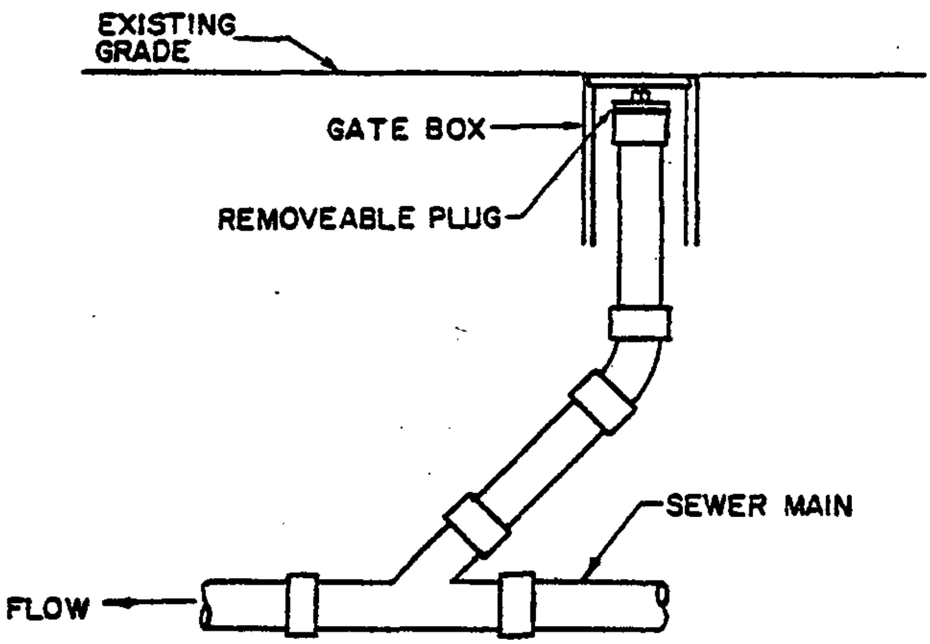


FIGURE 8 : "CLEAN OUT" FOR SMALL DIAMETER GRAVITY SEWERS

sewage from backing up into a septic tank from the main line. Generally, a properly designed system should not require backflow prévention devices. The designer can adjust the hydraulic grade line by choosing pipe sizes and depth of excavation so that backflow would not occur. Where necessary, backflow devices that minimize obstruction to the flow path while open are desirable (Simmons and Newman, 1982).

Construction of SDGS systems is much easier and, hence, less expensive than conventional sewerage systems. Specifically, trench width is less for smaller diameter pipelines and trench depth is often less for SDGS systems since a minimum pipe slope (to maintain a minimum velocity) is not required. The SDGS main need only be placed deep enough to prevent freezing and wheel load damage. Small diameter pipe is lighter and easier to handle than eight inch (or larger) diameter conventional system pipe, allowing the use of longer pipe lengths. This speeds construction. Sewer system infiltration and inflow (I/I) should also be reduced as the number of pipe joints (sources of I/I) is reduced. The line and grade of the SDGS main is less critical than conventional sewer mains (especially with VGS designs), saving alignment costs and also accelerating construction. However, when sewer mains are not placed at exact locations and are plastic, a metal wire (toning wire) should be placed directly over the pipe to make its subsequent location easier (U. S. EPA, 1980b). This should not significantly increase SDGS system cost.

Construction of SDGS appurtenances are also easier than conventional appurtenance construction (such as clean-outs in place of manholes), again translating to cost savings.

In the future, as discussed previously in this chapter (see STEP pressure sewers), advanced pipe laying equipment and pipeline materials may also further speed construction and, therefore, further reduce SDGS construction costs.

Field Performance

Two United States SDGS systems have been reported. A SDGS system employing a minimum flow velocity requirement and serving 79 homes, 6 businesses and 1 school in Westboro, Wisconsin, is reported by Fey (1978) and the Small Scale Waste Management Project (SSWMP) (1981). A variable grade SDGS design, serving 31 homes in Mt. Andrew, Alabama, is reported by Simmons et al. (1982) and Simmons and Newman (1982).

The Westboro, Wisconsin, project was implemented to replace soil absorption systems which were failing due to unsuitable soils. This project was really a hybrid system -- low lying areas of Westboro were served by STEP pressure sewers discharging to gravity sewers. The hybrid-SDGS system alternative allowed 13 more homes to be served than a conventional sewerage facility would have. Twelve percent system construction cost savings (collection and soil absorption field treatment) over conventional wastewater management technology (conventional collection and stabilization pond treatment) are reported (SSWMP, 1981). Cost savings attributable to collection alone cannot be developed with the limited information presented. It is reported however, that because of the manhole and minimum slope requirement, substantial cost savings compared to conventional sewers were not realized (SSWMP, 1981). A post-construction review speculated that substantial savings would occur with a modified design (SSWMP, 1981). More specifically, waiving the minimum velocity requirement (1.5 feet per second at one-half full flow), utilizing smaller diameter pipe (four inch minimum diameter main), replacing manholes with clean-outs, and requiring fewer existing septic tanks to be replaced (all but nine were replaced) are all modifications that would induce more substantial cost savings than those actually realized.

Some of the problems experienced in Westboro resulted from poor wastewater flow estimates. In the project's final design, flow estimates sixty percent greater than realized (40 gpcpd average) were employed. Poor flow estimates unnecessarily increase construction and operational costs for both wastewater conveyance and treatment. These costs, of course, are borne by the user.

Odors and the corrosive nature of septic tank effluent induced problems and complaints in Westboro (SSWMP, 1981). Ferrous materials in pumping stations along the SDGS main line were particularly vulnerable to corrosion (SSWMP, 1981). Therefore, the SSWMP (1981) suggests that all lift station components be constructed of non-ferrous metals. By minimizing agitation of septic tank effluent in the lift stations, odor problems were reduced (SSWMP, 1981).

Another operational problem reported at Westboro is an increase in wastewater suspended solids during conveyance (SSWMP, 1981). Apparently, part of the problem is sloughing of biological slime in sewers. The growth of significant biological slime in the pipeline was surprising; septic tank effluent lines are normally clean, even after years of operation (Fey, 1978). The slime growth here was probably a result of using excessively large diameter piping. The piping scheme in Westboro (four inch diameter pipe at 0.67 percent negative slope) could serve approximately 1800 persons (six times the existing load) at peak flows of one gallon per capita per hour (gpcph) -- while flowing half full (SSWMP, 1981). This provides a great amount of surface area upon which biological growth can occur. When a peak flow does occur and biological growth is sufficient, sloughing results. The use of smaller pipe might provide more frequent scouring of the pipe sidewall and less area for growth so that significant biological growth would not occur. Simmons et al. (1982) suggest that peak flow estimates of 0.4 gpcph where flow equalizing septic tanks are employed and 0.6 gpcph (plus a safety factor of ten gpcph to system total) where traditional septic tanks are employed be used for sizing SDGS mainlines. The manholes also contributed to solids problems in Westboro since they allowed debris to enter sewer mains. Generally however, the Westboro project operated very well (SSWMP, 1981).

The Mt. Andrew, Alabama, project, which also was a hybrid STEP/SDGS project, also performed satisfactorily and required little maintenance. Problems reported were insufficient septic tank performance (insufficient BOD and SS reductions because its design was essentially too small -- 500 gallons for a two bedroom home) and two instances of residential pump failure. Despite the poor pretreatment performance in this project, no problems in wastewater conveyance in the variable grade sewers have occurred. This is considered to indicate, by Simmons et al. (1982), that the VGS system is reliable.

After 18 months of operation, mainline pipe sections in low points were removed and inspected. These lines were coated with a thin greyish residue, of little hydraulic significance, but no heavy solids were noticed (Simmons et al., 1982). This again is significant in light of the poor pretreatment performance. No sloughing of a biological slime is reported in either report describing this project (Simmons et al., 1982; Simmons and Newman, 1982).

Cost Information

The cost to place VGS lines is estimated (1982 dollars) at two dollars per linear foot (Simmons et al., 1982). Inexpensive materials and the ability to lay sewer lines along the existing grade, independent of elevation, account for the substantial cost savings over conventional sewers.

A letter to P. E. and H. Engineers of Lexington, Kentucky, from W. F. Esmond of the New York State Department of Environmental Conservation (July, 1982; Ward, 1983) summarized construction bid information (actual bids and engineers' estimates) from 13 projects in New York State. (Small diameter gravity sewer systems here are similar in design to the Westboro, Wisconsin, project; Dauchy, 1983.) This information (Ward, 1983) indicates that when a significant portion of the project involves laying of small diameter sewers (four inch minimum required diameter in New York state), construction costs on the order of ten dollars per foot are reasonable. It is not clear how often, on the average, pumping stations are constructed in these systems. These same estimates indicate that eight inch diameter sewer installation costs are approximately 25 to 50 percent more than the SDGS option.

As mentioned previously, accurate cost information for the SDGS collection system in Westboro, Wisconsin, is not available.

Summary

Small diameter gravity sewers, preceded by septic tanks, can reliably, and often cost effectively, transport sewage. While a net negative gradient between user and discharge location is required, rolling topography in-between should be of little concern. Small diameter gravity sewer pipes can be constructed, within reason, to follow existing topography. (The variable grade concept has been proven but, like any other technology, it cannot be abused (Simmons and Newman, 1982).) The result can be substantial excavation cost savings.

The backbone of small diameter gravity sewer systems is pretreatment of wastewater to remove solids and grease. As with any septic tank or similar pretreatment device, the accumulation of solid material requires occasional pumping by a septage hauler. Failure to regularly clean such facilities may lead to clogging of the sewer line.

An enforceable, supervised program to periodically inspect and clean pretreatment facilities should be a part of all community collection systems. Such a program was developed in Westboro, Wisconsin (SSWMP, 1981). A community sewerage district, a local government agency which enabled Westboro to obtain easements onto private property for cleaning and inspection of all septic tanks, was formed. The Town of Westboro now hires a contractor to clean one-third of all septic tanks annually. (Non-residential septic tanks should most likely be cleaned more frequently.) Because the septic tank pumpings are regularly scheduled and not emergency calls, significant cost savings per pumped tank are realized (SSWMP, 1981). In Westboro, where residential lift stations are required to lift sewage to the sewer main grade, the homeowner is responsible for the operation and maintenance of that station (SSWMP, 1981).

In a SDGS system, it is not necessary to maintain a scouring velocity. The successful performance of the Mt. Andrew, Alabama, sewer system, which experiences periods of very low flow and was constructed with negative, flat, and positive slope pipe sections, supports this conclusion.

CHAPTER 7

Package Plants

This report has already discussed on-site treatment systems dependent on a soil matrix for purification. Where suitable soils do not exist, or creation of such conditions is prohibitively expensive, "higher technology" systems, generally independent of soil matrices, are required before habitation of that region is allowed. Higher technology systems generally are more complex, energy and labor intensive, and require more maintenance and greater operator knowledge than soil dependent systems.

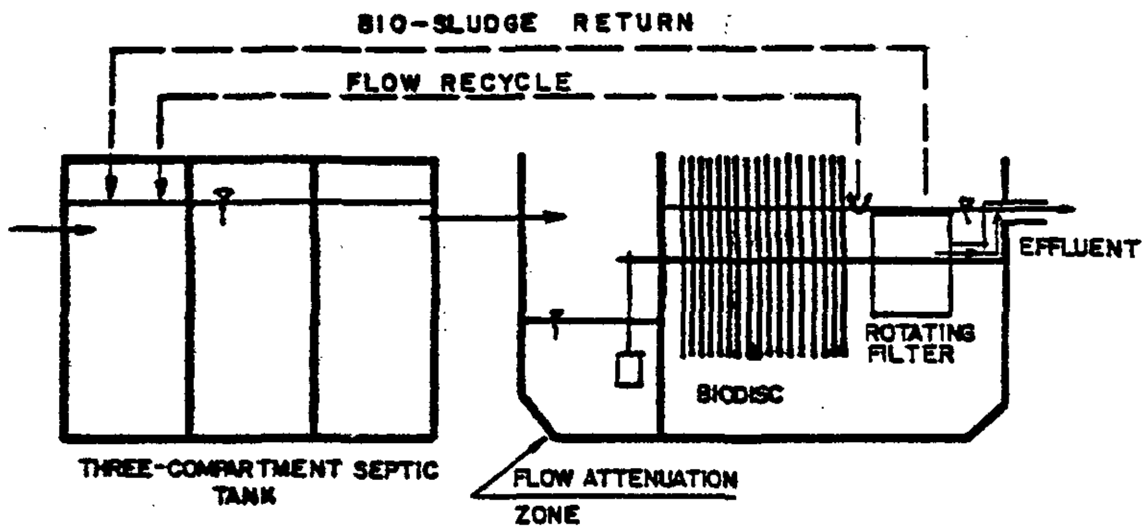
A conventional approach to wastewater treatment has been to collect wastewater throughout a large region and provide a biological wastewater treatment process at its terminus. Problematic to this approach are: (1) the environmental effects of discharging a large quantity of treatment effluent as a point source, (2) the production of residues (Laak, 1980a), (3) the cost to construct such systems, and (4) the cost to manage, operate and maintain these systems.

In chapter six, collection systems that convey sewage, with potentially significant cost savings to the user, were discussed. Such systems can be adapted to the needs of regions requiring small flow technology, particularly their characteristic financial restrictions. A wastewater treatment facility at the collection system terminus should be no exception.

Package plants, generally, are wastewater treatment systems that may meet small flow technology goals. To a degree, they are "scaled-down" versions of large wastewater treatment facilities. They are not, as large treatment plants are, custom built. Herein lies their biggest advantages. Package plants are produced in an assembly line manner, reducing their construction cost. They are known as package plants because they are usually prefabricated and delivered to a site ready to be connected to influent sewer, power supply and effluent discharge.

A. Package Plant Technology

Harr (1982) suggests that there are basically two types of package plants: Treatment plants developed especially for on-site wastewater treatment and treatment plants developed for large flows and scaled down to serve small flows. Examples of the former are septic tanks, and a Mecana type package plant, shown in Figure 9 (Harr, 1982). Examples of the latter are the extended aeration processes shown in Figure 10. Treatment plants scaled down to serve small flow needs should be modified to accept a



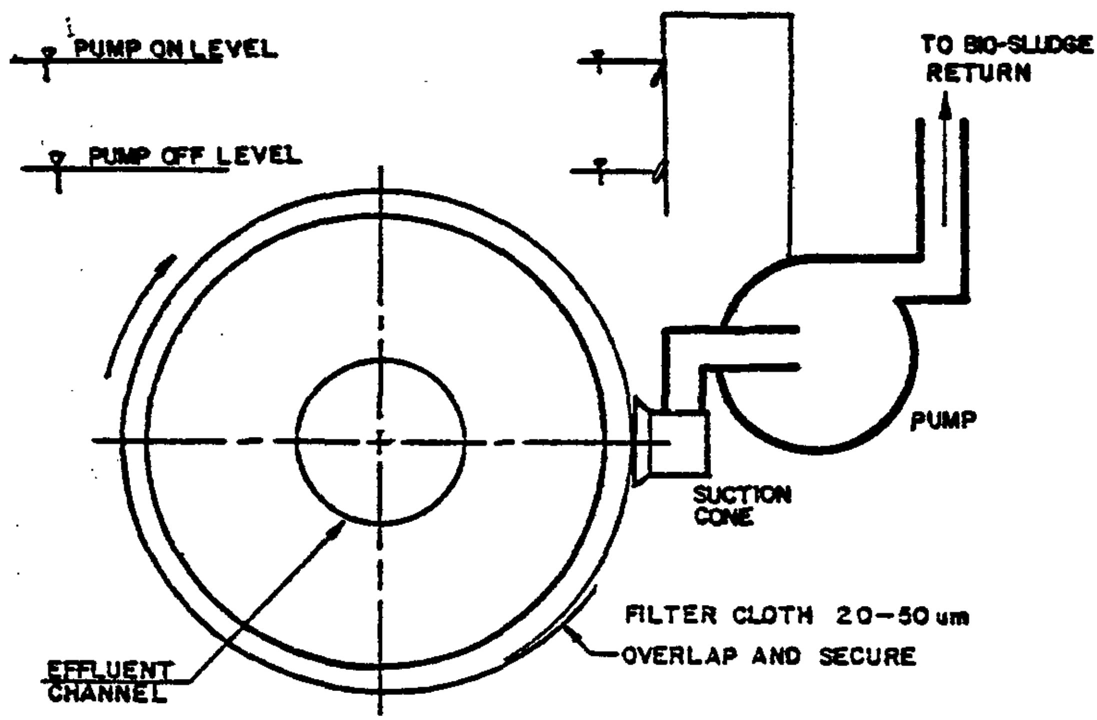


FIGURE 9: MECANA PACKAGE PLANT
PROCESS SCHEMATIC (TOP)
CLOSE-UP OF FILTER (BOTTOM)
(HARR, 1982)

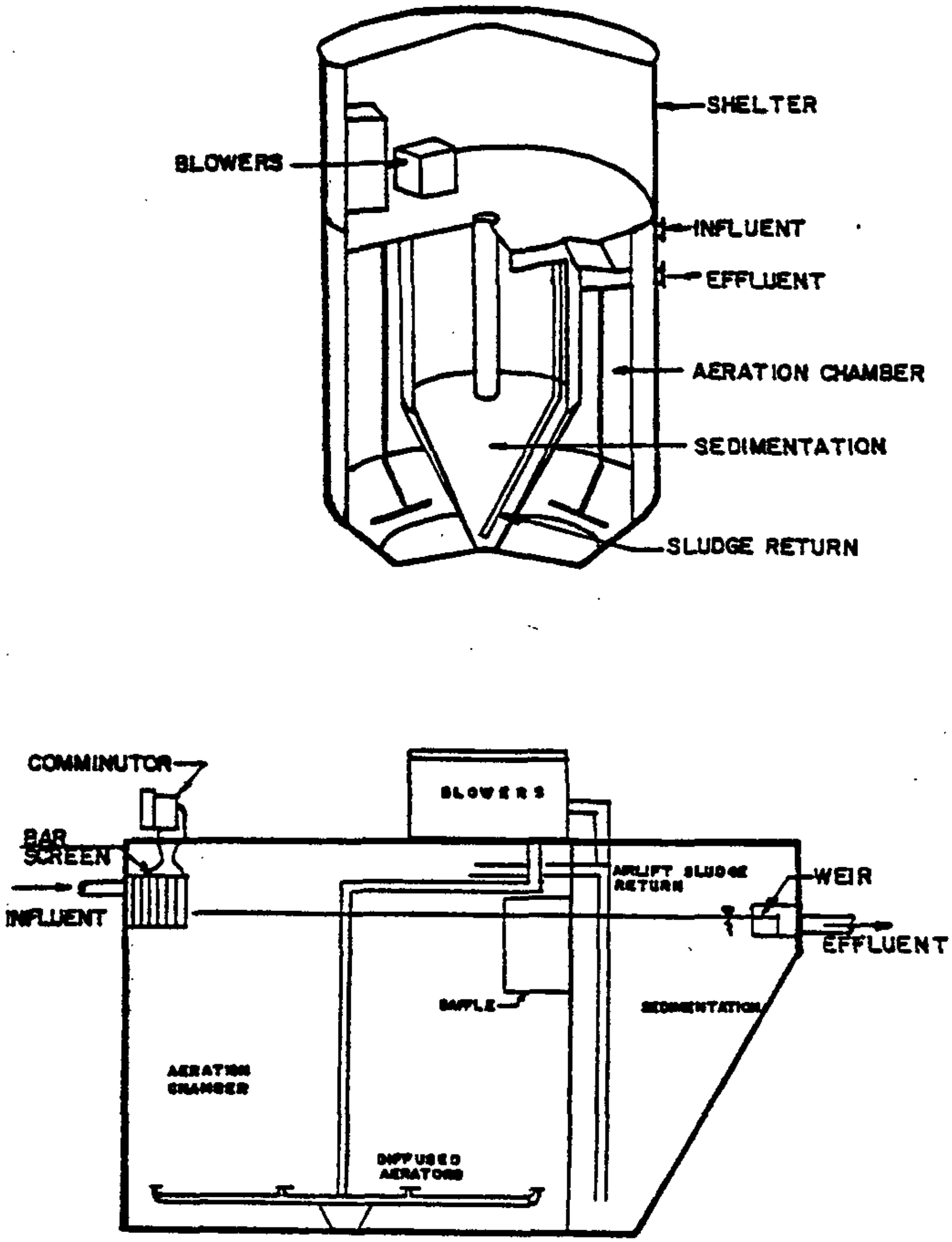


FIGURE 10: EXTENDED AERATION PROCESSES
ALCLEAN PLANT (HARR, 1982) TOP
CLOW CORPORATION (CLOW, 1983a) BOTTOM

slightly stronger wastewater (higher BOD, SS and grease and oil concentrations) with greater diurnal variation than large municipal plants will experience. Chapter two of this report reviews the characteristics of rural domestic wastewater.

Currently, there exists a general lack of information regarding package plant performance, reliability and cost. Literature on biological wastewater treatment however, is certainly in abundance and from this, projections on package plant performance can be made.

The two most common package biological systems are extended aeration activated sludge systems and fixed film processes. Both provide, when properly designed and operated, adequate treatment and reliability.

B. Extended Aeration

Extended aeration activated sludge is an aerobic biological process which oxidizes degradable soluble organic and inorganic materials to their oxidation end products (such as CO_2, H_2O, SO_4^{-2}, N_2, NO_2^-, NO_3^-, and a microbial biomass; U. S. EPA, 1980b). Extended aeration processes operate in the endogenous respiration phase of most of the mixed group of microorganisms significant to wastewater treatment. Long mean cell residence times (MCRT's) (usually between 20 and 30 days), long aeration periods and relatively low organic loadings are responsible for this (Metcalf and Eddy, 1979). An advantage of operation during the endogenous respiration phase is that residue production decreases. In fact, it was initially thought that extended aeration processes would stabilize domestic wastewater without requiring sludge wasting. In theory, if mixed liquor suspended solids remained in a range of 5,000 to 7,000 mg/l and hydraulic retention at 24 hours, sludge wasting would not be required. The sludge production rate would be low enough so that solids discharged over the effluent weir would prevent the accumulation of solids within the system (Grady and Lim, 1980). Presumably, effluent pollutant concentrations would still be low enough to meet discharge criteria. In practice, as the residence time of microbial cells in the system increases, net cell synthesis (and hence, the need for sludge wasting) decreases, but never reaches zero due to the presence of a certain amount of nondegradable solid synthesized by microbes (U. S. EPA, 1980b).

A disadvantage of biological treatment during the endogenous phase of growth is that the settling characteristics of the population are poorer than systems operating at MCRT's in the range of three to nine days. As MCRT's increase beyond 15 days, a deterioration in the settling characteristics of the mixed liquor is seen, the result of small floc particles, called pin floc. As a microbial population develops, polysaccarides are excreted. At MCRT's below 15 days, this biopolymer acts to congregate bacteria and form settleable biological floc particles. At long MCRT's, excessive biopolymer production may be responsible for restabilizing bacteria (Grady and Lim, 1980). Another mechanism may be that during endogenous respiration, these biopolymers are consumed by bacteria, breaking up the floc particles. The exact mechanism is not clear (Grady and Lim, 1980). Another operational disadvantage of extended aeration is that the compressibility characteristics of extended aeration sludge are worse than those of activated sludge systems operating at MCRT's of three to nine days.

The performance of extended aeration plants at removing soluble BOD should be very good. Figure 11, adapted from Grady and Lim (1980), indicates that at high MCRT's, very low effluent substrate concentrations result. Figure 12, also adapted from Grady and Lim (1980), shows that at high MCRT's, cell production decreases and oxygen requirements increase. The increased oxygen requirement is due to cell decay.

What these points about extended aeration processes should indicate to the designer are the importance of conservatively designed clarification facilities and sufficient aeration capacity to ensure adequate treatment performance. Clarification is perhaps the most important process in any activated sludge processes and for extended aeration, the design of these facilities becomes even more critical.

Martel, Digiano and Pariseau (1979) report that chemical precipitation using aluminum salts, intended for phosphorus removal, improved overall treatment performance of an extended aeration package plant as well as achieving significant phosphorus removal. Particular improvements in BOD_5, SS and turbidity were noted. Improved performance was attributed in part to chemical coagulation of colloidal organic particles. Aeration in activated sludge systems is obviously important but for extended aeration processes treating rural domestic wastewater it should not constrain the system so much that advanced aeration processes (eg. pure O_2) are required. Diffused aeration, providing both aeration and mixing, is common in package plants. No studies have reported septic conditions as a result of insufficient aeration capacity (only mechanical failures; Guo, Thirumurthi and Jank, 1981).

Figure 13 shows two variations of extended aeration processes, a batch system and a flow through system. The batch system offers simplicity in construction; by closing off influent lines and aeration, the batch reactor acts as a sedimentation basin. Clogging of the aeration diffusers during sedimentation is of concern. Also, space must be made available for influent

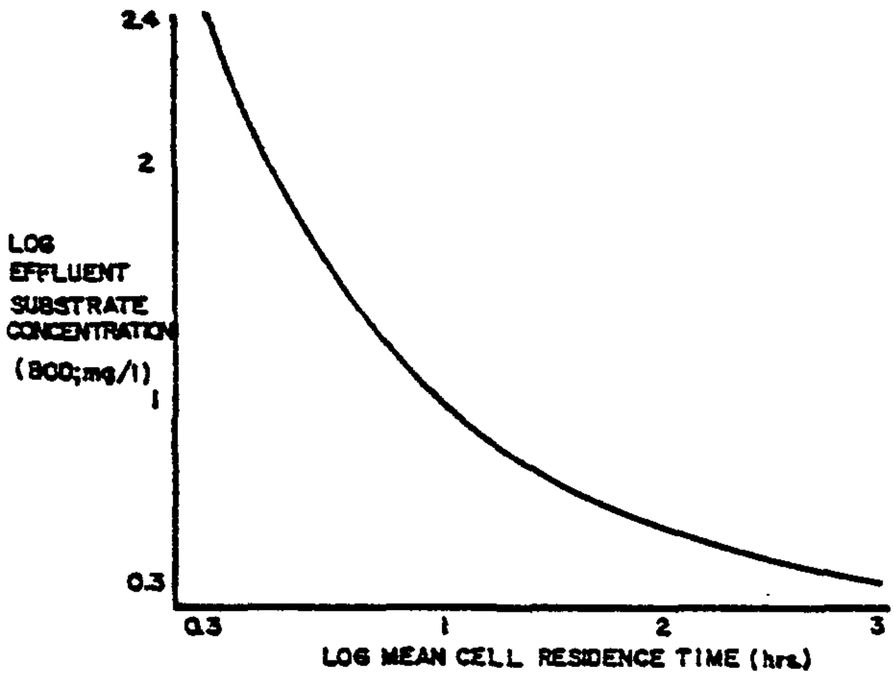


FIGURE 11: MCRT-EFFLUENT SUBSTRATE CONC. RELATIONSHIP ADAPTED FROM GRADY AND LIM (1980)

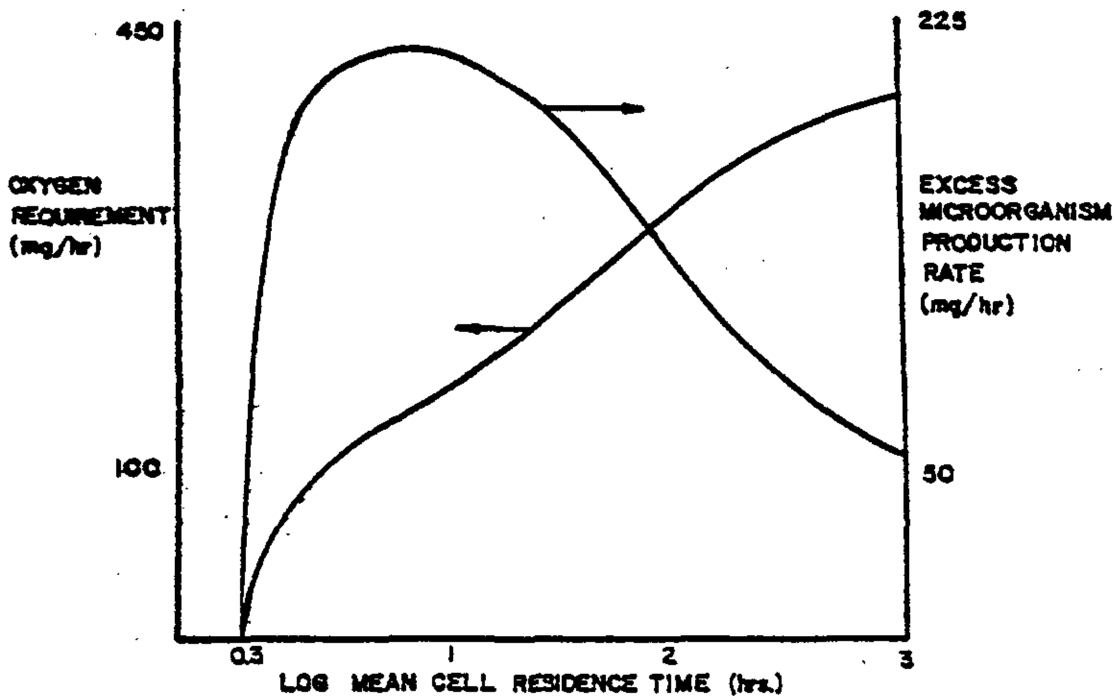


FIGURE 12: MCRT-CELL PRODUCTION AND OXYGEN REQUIREMENT RELATIONSHIP
ADAPTED FROM GRADY AND LIM (1980)

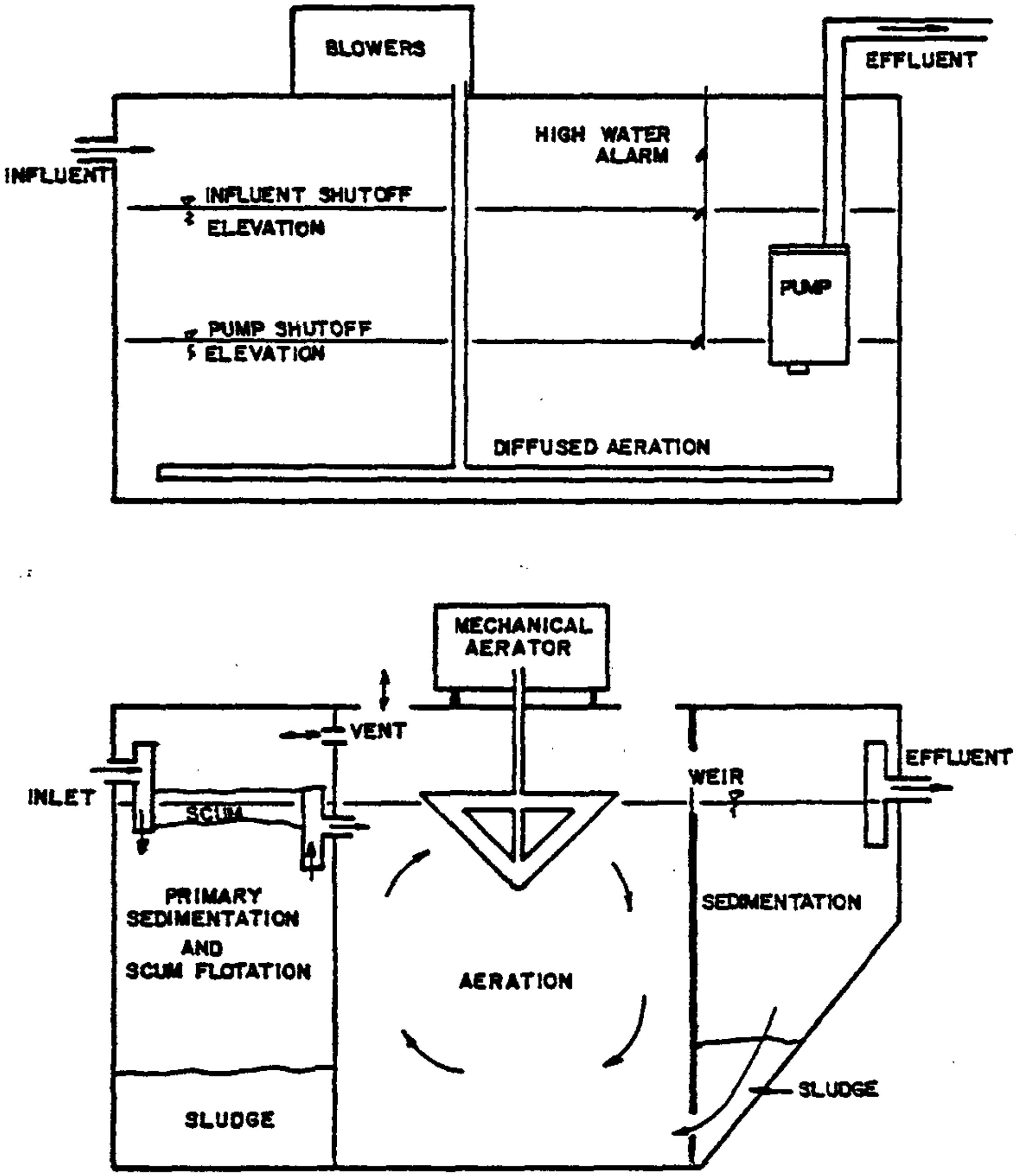


FIGURE 13: BATCH EXTENDED AERATION (TOP)
FLOW-THROUGH EXT. AERATION (BOTTOM)
(U. S. EPA, 1980b) NO SCALE

holding while its entrance has been shut off from the tank. Batch processes however, provide better soluble BOD removal than continuous flow processes. The installation of sequential batch reactor systems in rural areas has been suggested by Irvine, Miller and Bhamrah (1979). The diurnal flow variation of rural sewage may make such systems practical, settling sewage during low flow periods so that the required influent holding volume is low.

Extended aeration processes have several operational characteristics that may make their use in rural areas undesirable. As does any activated sludge process, it requires a great deal of attention to ensure adequate, reliable treatment. It is a delicate process that reacts to changes in flow and waste concentration (U. S. EPA, 1980b). Rich (1980) points out several weak points of activated sludge systems in general. These are: (1) a minimum resistance to shock loading, (2) a great sensitivity to intermittent operation, (3) a high degree of required operational skill, (4) high capital costs, and (5) high operational and maintenance costs.

Guo, Thirumurthi and Jank (1981) compared field performance of twenty extended aeration package plants to performance of 22 extended aeration processes under somewhat ideal conditions reported by the U. S. National Science Foundation (NSF; U. S. NSF, 1966). The NSF study reported average BOD_5 and SS effluent concentrations of 15 and 20 mg/l. Field performance data of the extended aeration plants studied by Guo, Thirumurthi and Jank (1981) indicated that of the 20 plants, only four produced effluent of comparable quality to the NSF report. The majority of plants did not meet treatment performance objectives (Guo, Thirumurthi and Jank, 1981). Poor performance was attributed to many problems including equipment failures and improper process design. The major cause of poor performance was determined to be a lack of proper maintenance due to insufficient manpower and operator knowledge (Guo, Thirumurthi and Jank, 1981).

C. Fixed Film Processes

Other variations of biological wastewater treatment commercially available in prefabricated form depend on microbial growth attached to an inert media (fixed film). Fixed film systems are able to concentrate a large microbial mass into a small space, allowing adequate treatment within a short hydraulic retention time and hence, compact system size. There are basically two fixed film systems: biodiscs and biofilters. Most package plants marketed today are biodiscs or downflow filters such as trickling filters or sand filters. Fixed film systems that seem feasible but currently are not commercially available as package plants are fluidized/expanded beds and anaerobic packed beds.

Biodiscs, also known as rotating biological contactors (RBC's), rotate through the wastewater, bringing the attached microbial growth in contact with their food source. Figure 14 shows an RBC package plant schematic. The discs are partially (40% of area) submerged in the wastewater. As the disc rotates, oxygen is transferred to the wastewater, maintaining aerobic conditions at the surface of the biofilm/wastewater interface. Additional air may be introduced to the bulk liquid but generally is not necessary (O'Shaughnessy, 1983). Fluid shear forces, due to the rotation of the biodisc, act to remove microbial growth from the inert surface. In this manner, a steady-state mass of bacteria may develop. Sheared microbial growth must be removed (most commonly by sedimentation) from the wastewater before disposal.

Biofilters are available in many configurations. Trickling filters, packed towers and upflow filters can be thought of as biofilters. Figures 15 and 16 show several biofilter schematics. A distinction of biofilters from biodiscs is that during biofilter operation, wastewater is transported to the attached microbial growth rather than moving the biological growth to the wastewater.

Overall operation of fixed film processes, similar to extended aeration processes, may be considered in the endogenous growth phase (Clark, Viessman and Hammer, 1977), the result of long MCRT's. The ability of microorganisms to remain fixed until hydraulic shear sloughs excessive bacteria off provides these MCRT's.

An aerobic/anaerobic process is responsible for renovating wastewater in most fixed film processes. An exception are processes that are intentionally only anaerobic. If air is drafted through the fixed media, as is common in RBC's and trickling filters, aerobic conditions will occur at the outer edge of the microbial film. As wastewater moves deeper into the film, oxygen is consumed by microorganisms and anaerobic conditions develop. Adsorption of colloidal material from the wastewater to the biofilm is also responsible for some wastewater purification.

Grady and Lim (1980) present a model that includes mass transfer limitations to describe wastewater renovation in fixed film processes. A stagnant liquid film is present between the bulk liquid and biofilm. It is thought that, due to mass transfer limitations, the concentration of microbial substrate decreases through this stagnant layer to the microbial film. Thus the concentration of substrate that microorganisms are exposed to is less than that in the bulk liquid. This decreases substrate removal rates and increases the area of biological attached growth required to achieve pollutant reductions.

Recycling of wastewater dilutes influent pollutant concentrations and generally reduces fixed film process reaction

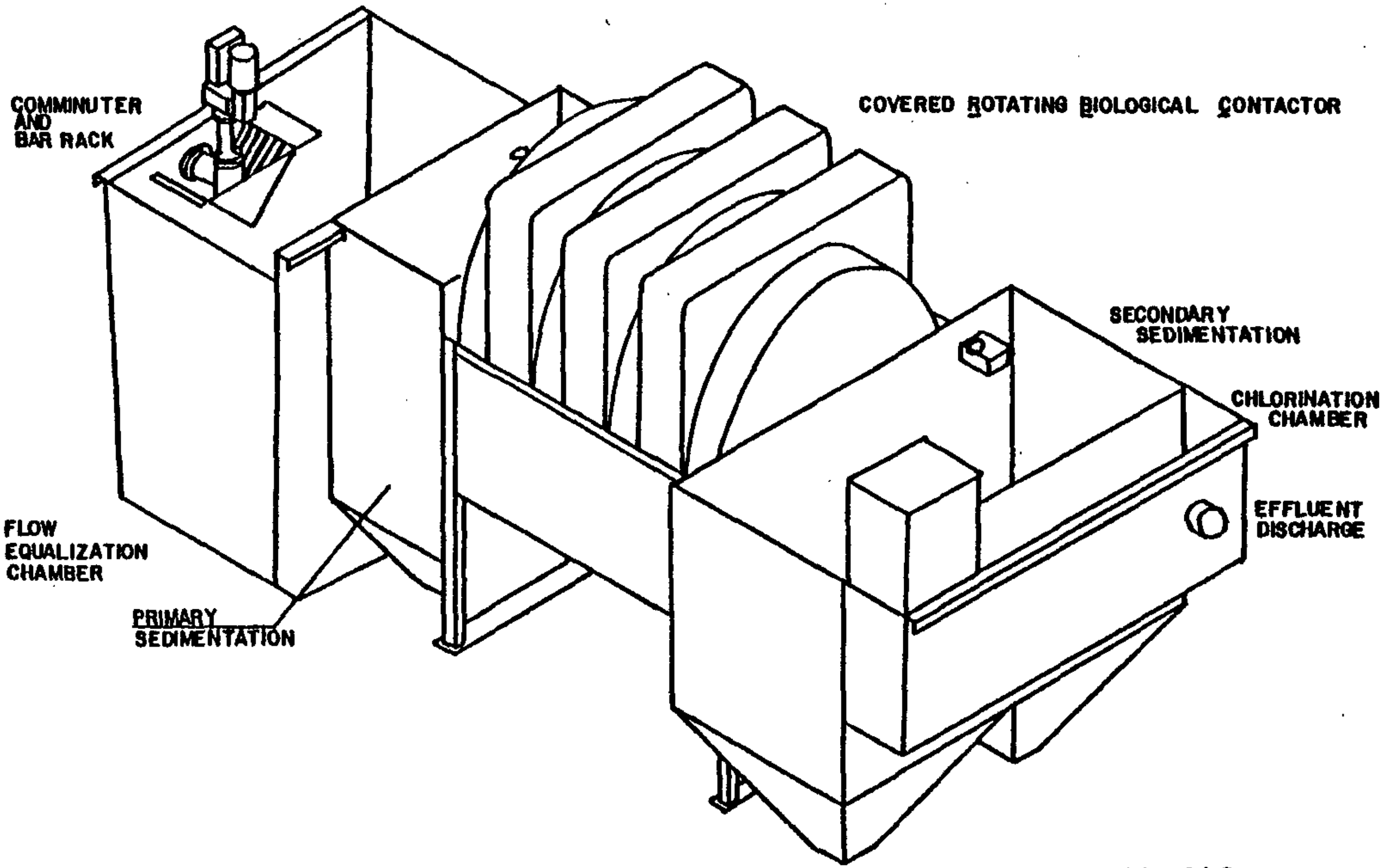


FIGURE 14: FIXED FILM (RBC) PACKAGE PLANT SCHEMATIC

(CLOW, 1983d) NO SCALE

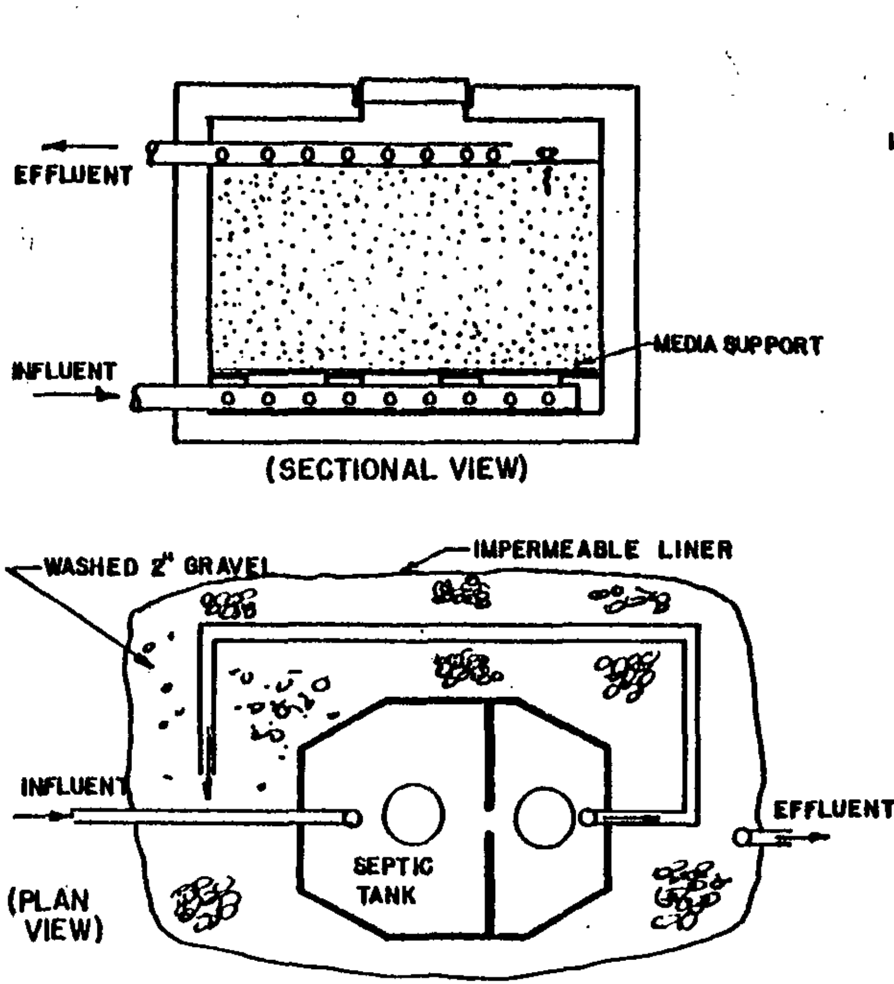


FIGURE 15: ANAEROBIC BIOFILTER SCHEMATICS

NO SCALE UPFLOW FILTER (KENNEDY, 1982) TOP
WRAP-AROUND FILTER (KENNEDY, 1982) BOTTOM

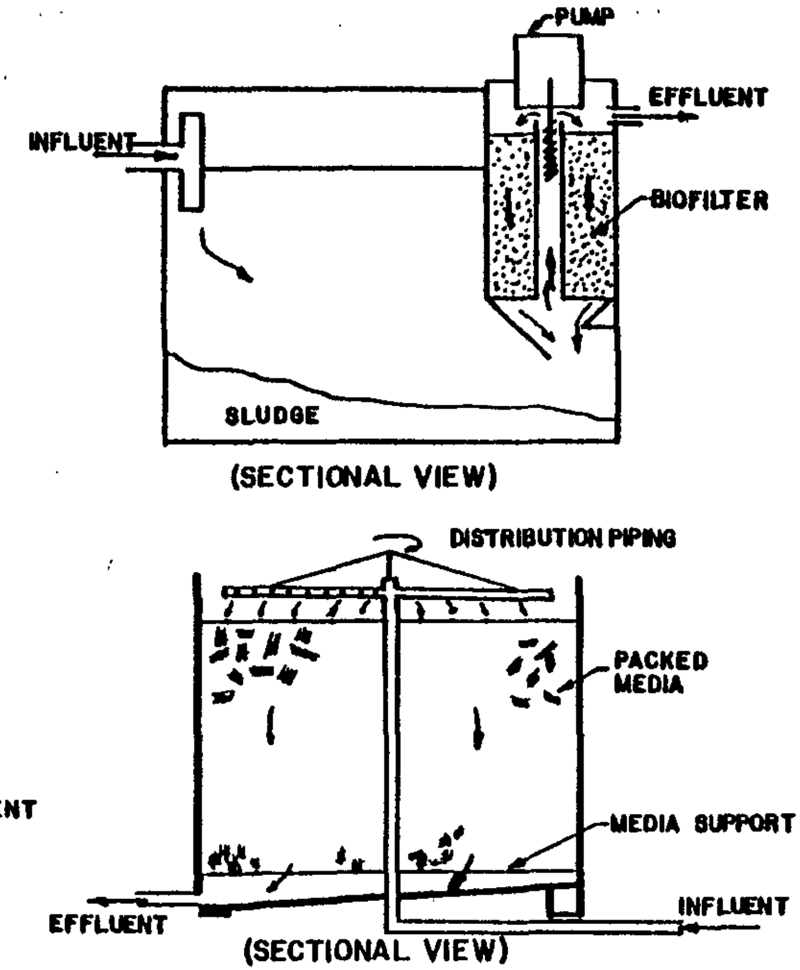


FIGURE 16: AEROBIC BIOFILTER SCHEMATICS

NO SCALE DOWNFLOW FILTER (U.S. EPA, 1980b) TOP
TRICKLING FILTER (U.S. EPA, 1980b) BOTTOM

rates. Although often desirable, the ability to recycle is usually not provided in package plants. Recycled operation can have several advantages over non-recycled operation. For example, rural domestic wastewater diurnal flow patterns normally show little flow during night hours. Recycling would continue to provide substrate to the attached growth, keep biological surfaces wet and provide fluid shear so that excessive biological growth does not begin to clog pores. Recycling also provides toxicant dilution within the treatment plant, dampening its effect on the treatment process and may help control nuisance organisms such as filter flies.

Fixed film processes in general are less susceptible to shock than suspended growth systems. While a hydraulic overload can flush a suspended growth systems' biological community out of the plant, the attached microorganisms in fixed film processes are much more likely to remain. Although unlikely, the entire fixed mass could be removed if fluid shear were sufficient. What is more likely is that only a portion of the mass would be removed during surge flows. Similarly, during toxicant loadings, the microbial mass in a fixed film process has a greater probability of tolerating a toxicant loading than the biological community in suspended growth systems.

The U. S. EPA (1980b) and Harr (1982) review fixed film package plant performance. They both point out the importance of primary treatment to reliable fixed film process operation. Debris not removed before fixed film processes may clog the filter or disc, making biological surfaces unavailable. While there is little long-term field experience with fixed film systems, the simplicity these systems offer should make them attractive alternatives to extended aeration plants. Flow to these systems can be fixed by pumping system design and sludge wasting can be controlled by a timer setting (U. S. EPA, 1980b). Their processes are less labor intensive than suspended growth systems; 8 to 12 semi-skilled man-hours per year plus analytical requirements (eg. permit conformance testing) can provide adequate performance (U. S. EPA, 1980b). Properly designed, they should produce effluent of equivalent quality as extended aeration facilities (U. S. EPA, 1980b). The U. S. EPA warns against excessive organic loading and indicates that should anaerobic conditions develop, poor performance and foul odors will result. During operation, visual inspection of biological surfaces can indicate the type of biochemical process taking place. O'Shaughnessy (1983) indicated that green surfaces indicate carbonaceous BOD removal while brown surfaces indicate nitrification processes. If sufficient inert media is present for biological growth, essentially complete nitrification can be expected (U. S. EPA, 1980b).

The Mecana treatment system, shown in Figure 9, has worked well in Switzerland (Harr, 1982). Primary sedimentation is provided by a three compartment septic tank. The septic tank, buffer zone and disc dosing method (bucket-by-bucket lift) all create an evenly loaded system. Clarification is provided by a rotating filter. Filter cloth replacement is necessary at least once a year. Sludge removal is provided by the suction device that travels along the filter cloth and is activated by head loss through the filter. The other biodisc plant reviewed by Harr (1982) is the Parca Norrahammer plant, shown in Figure 17. This plant is also reliable but has suffered from disc clogging. Harr does not indicate what the disc separation distance is. Harr also reviewed two biological filter plants, the Upo-Vesimies plant and the Emendo package plant (Figures 18 and 19). The Upo-Vesimies plant utilizes PVC for its inert media. Harr indicates that several mechanical problems have occurred. Over 1800 of these units have been delivered in Europe. The Emendo plant biological filter apparantly has a high rate loading and therefore poor (60%) BOD_7 (BOD measurement after seven days of incubated digestion) reduction. Phosphorus removal is very good however (90%), the result of chemical precipitation (Harr, 1982). Sludge production increases are expected.

As a matter of interest, Harr also describes a chemical treatment plant, shown in Figure 20. This plant is available for one to five households. It provides BOD and phosphorus removals similar to the Emendo plant. The Wallax plant requires no electricity however. Sludge is removed four times per year.

D. Summary

Package plants can provide a cost effective method of wastewater purification in situations requiring small flow technology. These plants may provide very good purification of wastewater if operated and designed correctly. Unfortunately, in the past, inadequacies in operator training, maintenance and process design have led to less than optimal performance.

Two types of package plant processes, extended aeration and fixed films are compared. Based upon their simplicity, lower operational costs and stability, fixed films processes should generally be preferred.

Immediate further research needs are in the areas of field operational performance and cost, so that reliable, low maintenance systems can be developed. Accurate comparisons of the feasibility of small flow treatment processes to larger conventional wastewater treatment systems are also impossible at this time, owing specifically to the lack of accurate capital and operational cost information.

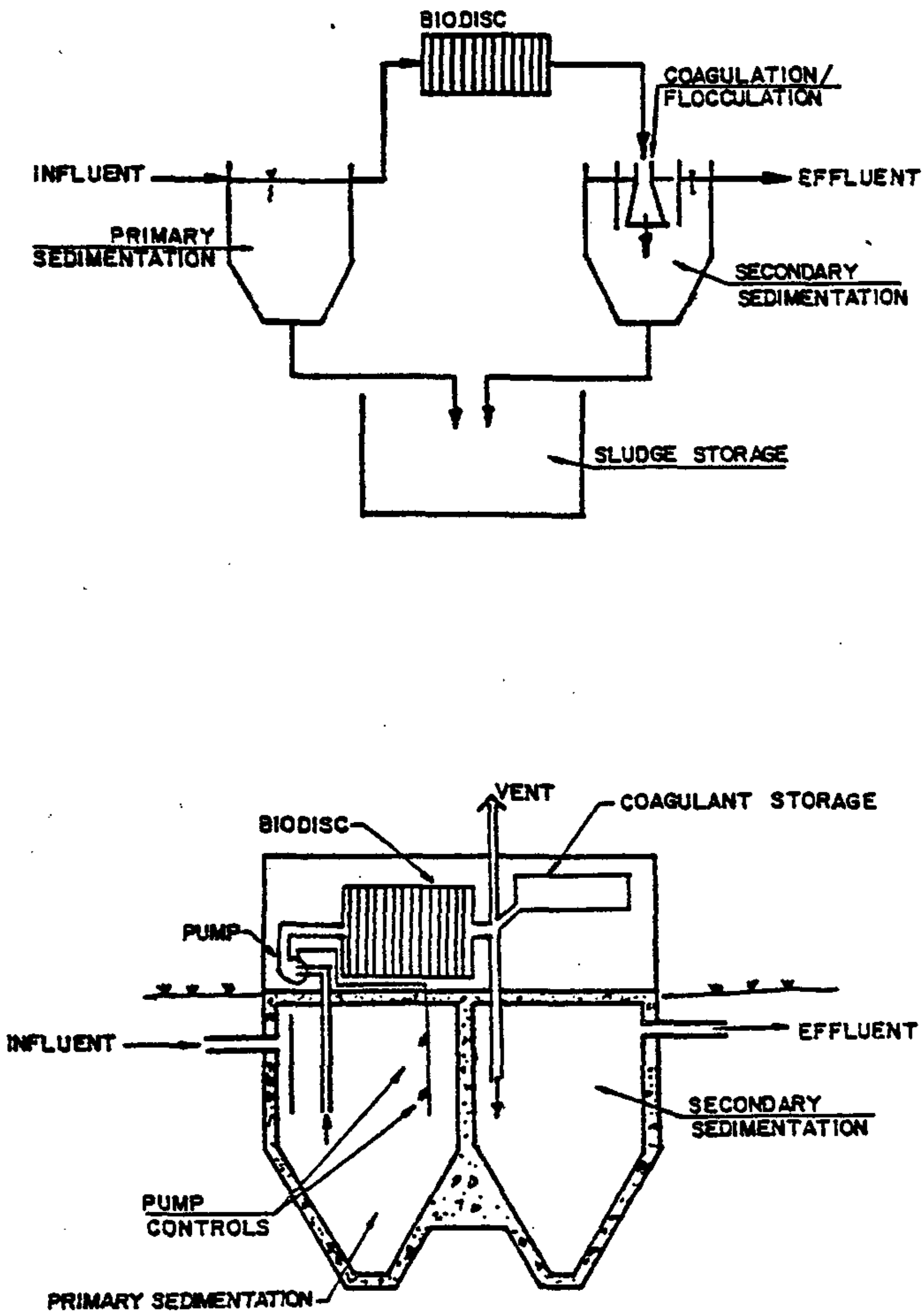


FIGURE 17 : PARCA NORRAHAMMAR PKGE. PLANT
PROCESS SCHEMATIC — TOP (HARR,1982)
SECTIONAL VIEW - BOTTOM

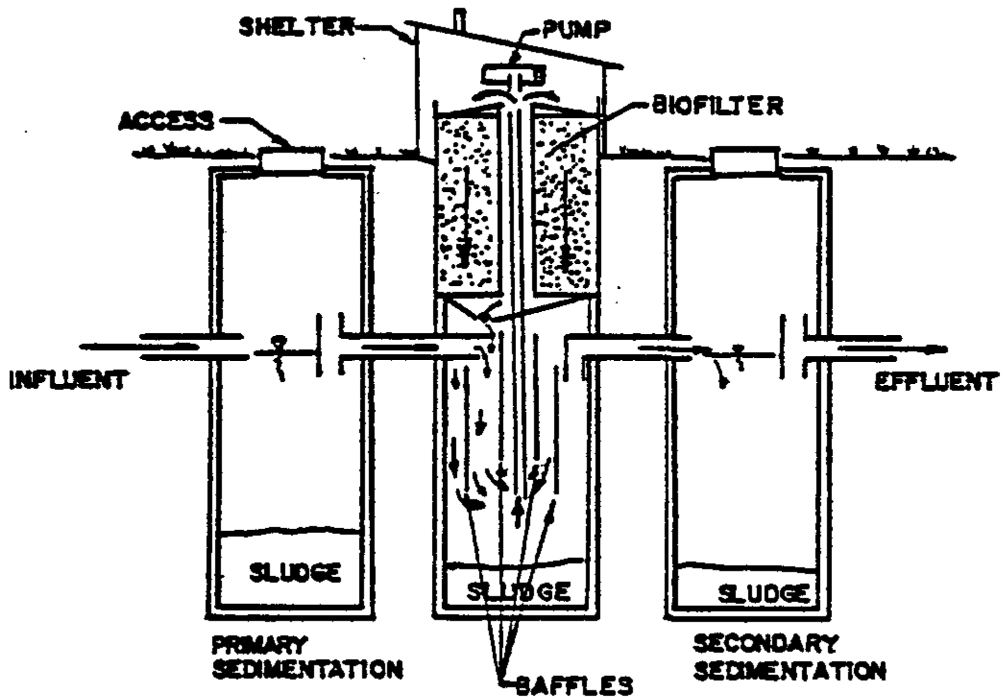


FIGURE 18: UPO-VESIMIES PACKAGE PLANT (HARR, 1982) NO SCALE

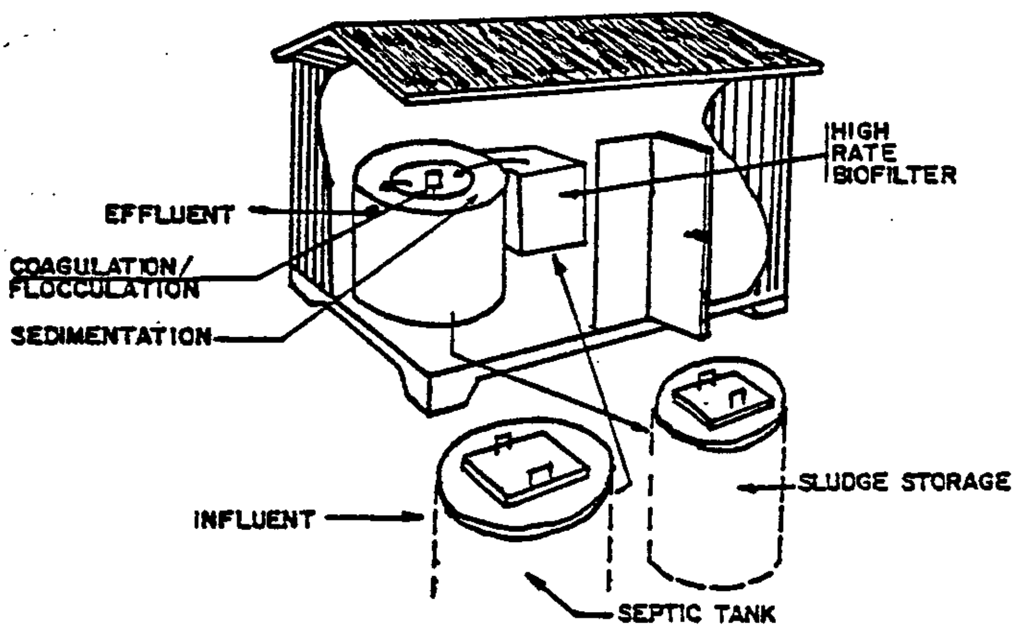


FIGURE 19: EMENDO PACKAGE PLANT (HARR, 1982) NO SCALE

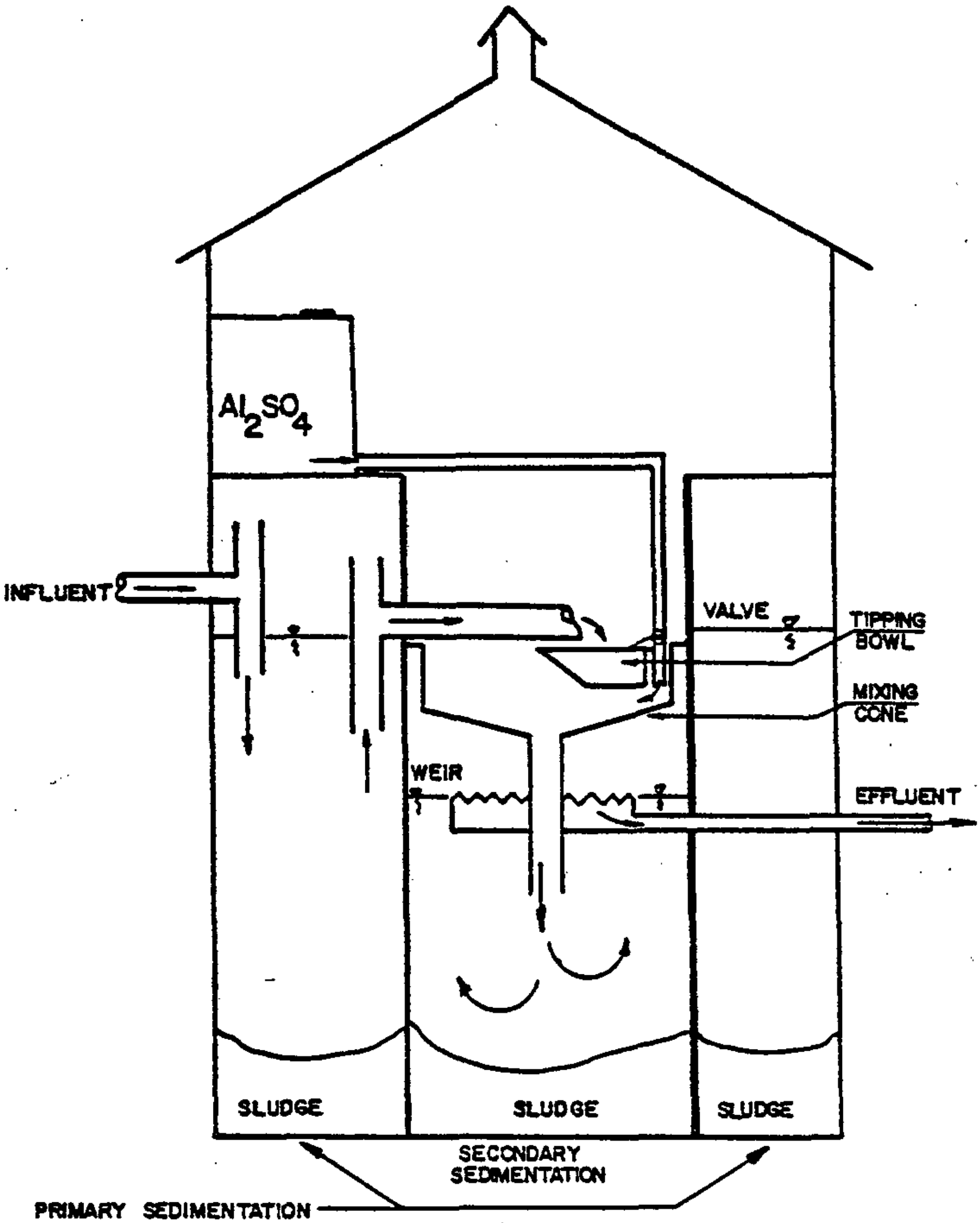


FIGURE 20: WALLAX CHEMICAL/PHYSICAL TREATMENT PLANT (HARR, 1982)

CHAPTER 8

Conclusions and Recommendations

This report has reviewed many topics pertinent to small scale wastewater management. As such, its greatest use may be as a comparative tool, allowing regulators and designers to be certain that proposed systems are conceptually sound. These conclusions and recommendations, presented on a chapter by chapter basis below, will concentrate on the major topics and questions this report addresses. More pertinent information and specific answers to the reader's questions can be gained by reviewing appropriate sections of this report.

As mentioned in the introduction of this report, the purpose of this report is not to review Title 5, the Massachusetts subsurface disposal regulations. However, during evaluation of the wastewater management systems this report considers, some regulatory inadequacies become obvious. The reader should recognize that this report is not sufficient for a complete Title 5 review. Some conclusions and recommendations, intended to be a step towards its improvement, are presented below.

1). Existing Massachusetts subsurface disposal regulations (Title 5) do not reflect the current knowledge of the performance, correct design and operation of septic tank - soil absorption systems. Because of its inadequacies, Title 5 does not provide the degree of environmental protection that it should, and can provide.

2). Changes can be made to improve the ability of Title 5 to protect groundwater supplies and their receiving surface waters.

A). A septic tank design, incorporating two compartments, baffles and surface area design requirements will improve the ability of subsequent treatment processes to perform reliably by providing better wastewater pretreatment at minimal increase in cost over current designs. Therefore, such a design should be used in on-site wastewater management systems in Massachusetts.

B). Current inspection and maintenance procedures mandated in Title 5 are unnecessary (to maintain adequate system performance) and ineffective (due to a general public disregard for this annual cleaning). Only those septic tanks serving larger than residential flows should be required to be cleaned annually. Title 5 regulations should be changed to require an annual inspection of residential septic tanks with cleaning as required. A public information and/or enforcement campaign (perhaps by local Boards of Health) should be undertaken to improve compliance with such regulations.

C). Improved, low-cost techniques for assessing the ability of a site to accept and renovate septic tank effluent are available. There are numerous problems with the existing procedure (percolation test) mandated in Title 5 and therefore, revision, incorporating these improved techniques, is suggested.

D). An improved procedure for absorption system design incorporating the site's long term acceptance rate, soil classification and a flow net analysis (to determine the site's hydraulic capabilities during worst case conditions) is suggested.

E). Title 5 should be modified to consider the performance of soil systems built in excessively permeable soils in renovating septic tank effluent. Under current design criteria, excessively permeable soils do not provide sufficient attenuation to treat septic tank effluent. Title 5 does not now consider this effect. Placement of less permeable soils in the absorption field or as a mound may achieve better waste purification.

F). Traditional soil absorption fields, when properly constructed, can be implemented in less permeable soils than are now required for soil absorption field construction.

G). The Title 5 suggested soil absorption system configuration should be a trench configuration, not a leaching pit since trenches provide better overall performance than leaching pits.

H). A wastewater disposal mound can provide adequate renovation of septic tank effluent at locations that are now unsuitable for disposal field construction (according to current Title 5 regulations). Title 5 should be modified to permit the use of wastewater disposal mounds

I). Because Title 5 is overly restrictive with regard to what soil conditions are necessary for construction of on-site soil absorption systems, Title 5 in some cases is effectively a land use control law rather than an environmental protection law.

Conclusions and recommendations, by chapter, about the major topics this paper discusses are:

Chapter Two

3). The characteristics of rural domestic wastewater, for most on-site wastewater management system design or evaluation purposes, can be approximated by Tables 1 and 2.

4). Rural wastewater generation can generally, and fairly accurately, be estimated at 45 gpcpd.

Chapter Three

5). The primary purpose of septic tanks in on-site wastewater management is sedimentation. Secondary to this is anaerobic digestion.

6). The characteristics of solid materials in septic tank effluent are markedly different than those of raw sewage solids.

7). Properly designed septic tanks can provide significant flow equalization and, when placed before pumping units, a significant quantity of wastewater storage.

8). Septic tank design should provide at least 24 hours hydraulic retention, minimize turbulent flow patterns, minimize solids carry over, and provide storage for several years accumulation of grease and solids.

Chapter Four

9). Past failures of septic tank - soil absorption systems are due more to improper site evaluation, construction errors, and misapplication of technology rather than inadequacies in the technology.

10). The development of a stable, homogeneous bacterial mat at the distribution trench / soil matrix interface is essential to on-site soil absorption system treatment of wastewater. It provides wastewater purification and helps to maintain unsaturated soil conditions below it. The treatment performance of a soil absorption system is a fuction of its ability to support a bacterial mat (clogging layer).

11). Unsaturated conditions are preferable to saturated conditions below the clogging mat, both for flow characteristics and purification processes.

12). Saturated permeability tests provide insufficient information for soil absorption system design. Additional information is required regarding soil texture, depth to seasonal high groundwater, and the groundwater flow regime.

A). There are significant precision and accuracy problems with current percolation test procedures. Sole reliance of soil absorption system design on this data invites failure.

B). Improved percolation test procedures are readily available and would cause little inconvenience or additional cost to on-site wastewater disposal system engineers.

13). Improved construction procedures can limit damage to soil absorption sites during construction.

14). An improved absorption system design procedure (use of LTAR) evaluates both flow through the bacterial mat (empirically derived) and site hydraulic capacity during worst case, saturated conditions.

Chapter Five

15). Phosphate detergent bans can substantially reduce a household's phosphorus production at little cost or inconvenience to consumers.

16). Total effluent phosphorus concentrations of less than 1.0 mg/l can reliably be achieved in centralized wastewater treatment facilities where chemical precipitation followed by conservatively designed clarification processes are employed.

17). In rapidly permeable or saturated soils, phosphorus may not be significantly retained on-site and can become a significant phosphorus load to receiving waters.

18). Unsaturated soil conditions, together with soils of high sesquioxide content and clay surfaces, can remove 99 percent of total phosphorus from a wastewater. Initial removals are by adsorption processes. Subsequent precipitation to aluminum, iron and calcium compounds further "fixes" phosphorus and may provide additional phosphorus sorption sites. Because of this regeneration mechanism, the long-term ability of a soil to retain phosphorus is often in great excess of that predicted by adsorption tests.

19). Organic materials in soils are not important to phosphorus retention unless they contain significant amounts of available aluminum and iron.

Chapter Six

20). Where on-site soil treatment of household wastewaters is not practical, community wastewater management systems often become necessary for habitation of that region. Alternative collection system design can enable habitation of otherwise uninhabitable areas and can be used to upgrade on-lot systems to higher technology treatment systems where environmental conditions require that improved treatment be provided.

21). Three sewerage systems that are viable alternatives to conventional gravity sewerage systems are: pressure sewers, vacuum sewers and small diameter gravity sewers. These systems generally require a greater degree of maintenance than conventional sewerage systems, but since substantial cost savings may be achieved (in construction) and since these maintenance costs should not be a significant burden to the homeowner, they are feasible alternatives that should be encouraged where on-site systems are not practical. Their reliability has been proven in several demonstration projects.

22). Variable grade, small diameter gravity sewers are a proven reliable method of transporting partially treated wastewater at very low cost. The design of such systems is along the hydraulic grade line, somewhat more complicated than conventional gravity flow sewer design. Variable grade sewers are generally preferable over other gravity sewer systems because of their substantial construction cost savings.

23). In many situations requiring small flow conveyance technology, a hybrid system, consisting of more than one of these alternative sewer systems will be the most cost effective alternative.

Chapter Seven

24). Fixed film package plants are preferable to suspended growth package plants because of their lower operational costs and better reliability.

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