TECHNICAL REPORT STANDARD TITLE PAGE

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA/TX-88+442-3		

4. Title and Subtitle	5. Report Date
WATER AND WASTEWATER SYSTEMS AT HIGHWAY REST AREAS	November 1987
	6. Performing Organization Code
7. Author(s)	8. Performing Organization Report No.
Carl W. Scharfe and Joseph F. Malina, Jr.	Research Report 442-3
9. Performing Organization Name and Address	10. Work Unit No.
Center for Transportation Research The University of Texas at Austin Austin, Texas 78712-1075	11. Contract or Grant No. Research Study 3-18-86-442
	13. Type of Report and Period Covered
12. Sponsoring Agency Name and Address Texas State Department of Highways and Public Transportation; Transportation Planning Division P. O. Box 5051 Austin, Texas 78763-5051	Interim
	14. Sponsoring Agency Code

15. Supplementary Notes

Study conducted in cooperation with the U. S. Department of Transportation, Federal Highway Administration

Research Study Title: "Design of Rest Area Comfort Stations"

16. Abstract

The majority of Texas highway rest areas were built in the 1960's. The water and wastewater systems at these rest areas reflect the technology available at that time. This report summarizes the current state-of-the-art technologies for water and wastewater systems at highway rest areas in the United States. Methods for determining rest area water demands, wastewater flows, pump sizes, storage tank volumes, and fixture requirements were explored. Various wastewater systems used at rest areas in outside states were evaluated. The two problems most frequently encountered in rest area water systems are inadequate water supply and/or water pressure. Water demand data for Texas rest areas is non-existent and thus water meters need to be installed at all Texas rest areas. Meters should separate the volume of water used in rest rooms from outside water demands at the rest area. For more immediate purposes, water demands can be estimated using the Zaltzman method. Ideal water pressure at rest areas is 40 psi with 20 to 60 psi being acceptable. Water system component sizing should be based on peak water demands. Rest area wastewater systems best suited for Texas, in order of preference, are (1) evaporative ponds, (2) overflow ponds, (3) overland flow or spray irrigation, and (4) evapotranspiration beds. Failed septic systems can be renovated using the systems listed above during rest area high use periods. Land requirements for rest area wastewater disposal systems are a minimum of approximately 3 acres and can be upwards to 10 acres. Recreational vehicles and water saving toilets will increase concentrations of organic wastewater constituents delivered to wastewater systems and will require changes in the operation of the treatment systems, such as more frequent septic tank pumpout.

17. Key Words	18. Distribution Statement
highway rest areas, water and wastewater systems, water demands, wastewater flows, pump sizes, storage tank volumes, fixture requirements	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161.

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	84	

Form DOT F 1700.7 (8-69)

WATER AND WASTEWATER SYSTEMS AT HIGHWAY REST AREAS

by

Carl W. Scharfe
Joseph F. Malina, Jr., P.E.

RESEARCH REPORT NUMBER 442-3

Research Project 3-18-86-442

Design of Rest Area Comfort Stations

conducted for

TEXAS STATE DEPARTMENT OF HIGHWAYS AND PUBLIC TRANSPORTATION

in cooperation with the

U.S. DEPARTMENT OF TRANSPORTATION

FEDERAL HIGHWAY ADMINISTRATION

by the

CENTER FOR TRANSPORTATION RESEARCH

Bureau of Engineering Research
The University of Texas at Austin

November 1987

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

PREFACE

The information and recommendations presented in this report are the result of the cooperation of several people. The literature search for this report was largely performed by the library staff at the Texas State Department of Highways and Public Transportation and by graduate student Sarah Winkler. Report figures were drawn by Jean Gehrke.

LIST OF REPORTS

Report 442-1, Volume I, "Investigation of Rest Area Requirements," by W. T. Straughan, David W. Fowler and Kirby W. Perry, October, 1988.

Report 442-1, Volume II, "Investigation of Rest Area Requirements, Appendix - Pertinent Rest Area Literature," by W. T. Straughan, David W. Fowler and Kirby W. Perry, October, 1988.

Report 442-2, "Evaluation of Energy Sources for Roadside Rest Areas," Brian A. Rock and Gary C. Vliet, December, 1986.

ABSTRACT

The majority of Texas highway rest areas were built in the 1960's. The water and wastewater systems at these rest areas reflect the technology available at that time. This report summarizes the current state-of-the-art technologies for water and wastewater systems at highway rest areas in the United States. Methods for determining rest area water demands, wastewater flows, pump sizes, storage tank volumes, and fixture requirements were explored. Various wastewater systems used at rest areas in outside states were evaluated.

The two problems most frequently encountered in rest area water systems are inadequate water supply and/or water pressure. Water demand data for Texas rest areas is nonexistent and thus water meters need to be installed at all Texas rest areas. Meters should separate the volume of water used in rest rooms from outside water demands at the rest area. For more immediate purposes, water demands can be estimated using the Zaltzman method. Ideal water pressure at rest areas is 40 psi with 20 to 60 psi being acceptable. Water system component sizing should be based on peak water demands.

Rest area wastewater systems best suited for Texas, in order of preference, are (1) evaporative ponds, (2) overflow ponds, (3) overland flow or spray irrigation, and (4) evapotranspiration beds. Failed septic systems can be renovated using the systems listed above during rest area high use periods. Land requirements for rest area wastewater disposal systems are a minimum of approximately 3 acres and can be upwards to 10 acres. Recreational vehicles and water saving toilets will increase concentrations of organic wastewater constituents delivered to wastewater systems and will require changes in the operation of the treatment systems, such as more frequent septic tank pumpout.

SUMMARY

Water and wastewater systems for rest areas are described in this report. The report contains methods and recommendations which can be used in choosing and designing water and wastewater systems at rest areas. Design information is included and should be helpful to design engineers. Costs are only covered briefly in this report because costs are highly variable and dependent on local conditions.

IMPLEMENTATION

This report presents methods useful for the design of water and wastewater systems at rest areas. State-of-the-art systems used in other states are presented and reviewed. The information in this report has supported the use of certain methods and systems presented in the report. That there is a need for more environmental and traffic data collection at rest areas is a major conclusion of this report.

TABLE OF CONTENTS

CHAPTER 1. INTRODUCTION

1.1 PURPOSE

In the 1960's Texas was one of the first states to establish rest area comfort stations along state and Interstate highways. State-of-the-art technology available at that time was used to design water and wastewater systems at those early rest areas with varying degrees of success. States that followed Texas in building highway rest areas were able to take advantage of technological advances and the Texas experience. Texas, once a leader among the states in the designing and building of rest areas, now lags behind other states.

This report is a compilation and evaluation of information on the types of treatment systems used at rest areas in other states. Guidelines and recommendations regarding state-of-the-art systems used in other states have been formulated and are presented here.

1.2 REPORT FORMAT

This report has been written in a format which allows its use as both a reference and a general set of guidelines. Chapter 2 emphasizes guidelines on water systems. Equations and methods for estimating water demands provide a basis for selecting system components. Chapter 3 deals with wastewater treatment systems and includes a description and a review of the performance of various treatment systems. Chapter 4 reports on the relative costs of rest area wastewater treatment systems. The appendices give detailed explanations of various methods proposed.

CHAPTER 2. REST AREA WATER TREATMENT SYSTEMS

2.1 INTRODUCTION

Rest area comfort stations must be supplied with water in adequate quantities and of acceptable quality. The selection of a location of a rest area must consider the availability, cost, and quality of the water supply to the area. The water demand at a rest area will determine the required components in the water systems. Rest area water systems are classified as "non-community systems" and, therefore, the minimum drinking water quality standards established by the Texas Department of Health must be satisfied.

2.2 SOURCES

The water supply of a rest area can be supplied by a municipality or withdrawn from a well or surface source. A municipal supply for the rest area often is the best choice, if the rest area can be serviced by the municipal system. Municipal sources ensure water of high quality and are systems with low maintenance and operation costs. In most cases storage is not required. When using a municipal supply it is important that the pipeline to the rest area be large enough to provide enough water at a sufficiently high pressure to permit simultaneous use of water closets and lawn sprinkling equipment.

A well supply is the second choice if a municipal supply is not cost effective or not available. Wells should be located a "safe" distance from possible contaminants. The Texas Department of Health spells out the minimum distances for sources classified as "Public Water Supplies," and rest area well distances from contaminants can be modified slightly from these standards. The well supply should be located at least 100 feet from wastewater treatment facilities and sewer lines and an enclosure should be used to protect the supply if livestock are nearby. Wells should be located on higher ground to avoid flooding from storm generated surface waters. Figure 2.1 depicts a drilled well in sand or gravel, and Fig. 2.2 shows a drilled well in bedrock. Folks recommends use of drilled wells because (1) they can penetrate an aquifer located far below the water table, (2) the yield is not as influenced by fluctuating water tables, and (3) they are more protected from surface water pollution hazards than other types of wells [1].

Surface water supply sources specifically for rest areas should be avoided; surface waters usually require treatment prior to use, resulting in high capital, operating, and maintenance costs. Watersheds for surface supply should be free of septic tank drainfields, livestock lots, and agricultural runoff and preferably should be wooded or grass covered.

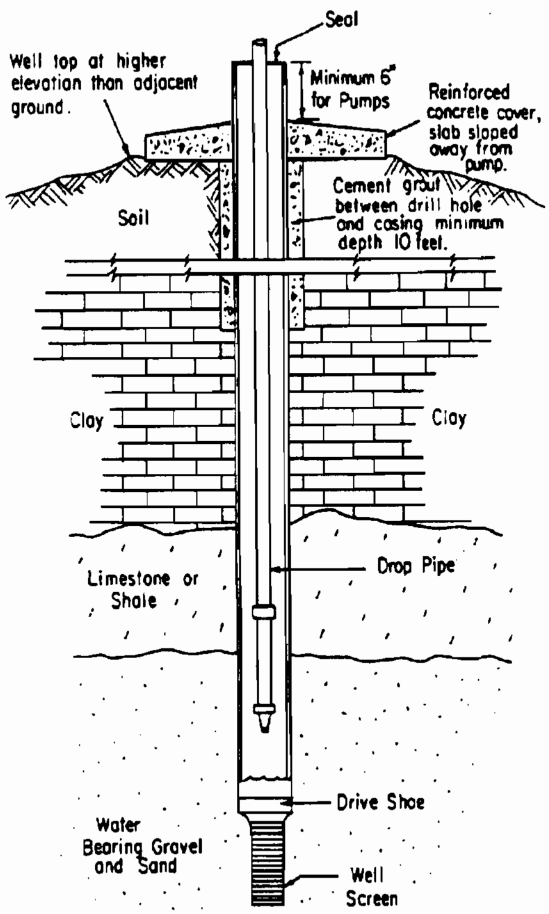


Fig 2.1. Proper construction of a drilled well obtaining water from sand or gravel [1].

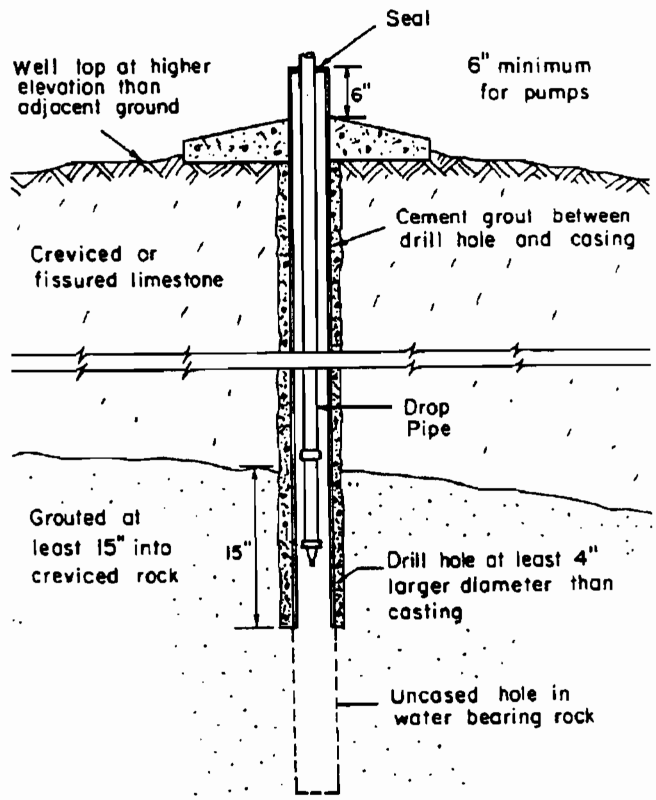


Fig 2.2. Proper construction of a drilled well obtaining water from rock [1].

Ponds should be greater than 8 feet deep, be large enough to store a year's supply of water, and be free of algae and weeds. River and stream sources should be upstream of wastewater treatment plant discharges and should be pumped out when the silt load is low (at high stage shortly after a storm).

2.2.1 Disinfection

Disinfection is recommended for all well and surface supplies and water stored in tanks prior to use. Chlorination is the most common and most cost effective method of disinfection of water at rest areas. The form of chlorine to be used depends on the location of the rest area, water demands, the skill level of the maintenance force, and available funds.

Chlorine gas injection is recommended if pump capacities are 60 gpm or more [1] and if an automated system is desired. Figure 2.3 depicts a typical chlorine gas injection system. Liquid chlorine (common household bleach) or powered chlorine is the preferable form of chlorine to use when water demands are below 60 gpm. These forms of chlorine are easy to mix and inexpensive and the hypochlorinator system in which they are used is easier to maintain than a gas injection system. A typical hypochlorinator system is shown in Fig. 2.4. It is recommended that, as a minimum, the water be tested monthly and have a chlorine residual of 1 mg/l as HOCL after 30 minutes contact time, have a turbidity level of less than 5 Nephelometric turbidity units (NTU), have a pH equal to 8, and meet state standards for coliform counts [1]. Therefore, equipment to adjust the pH of the water also may be required in the water system.

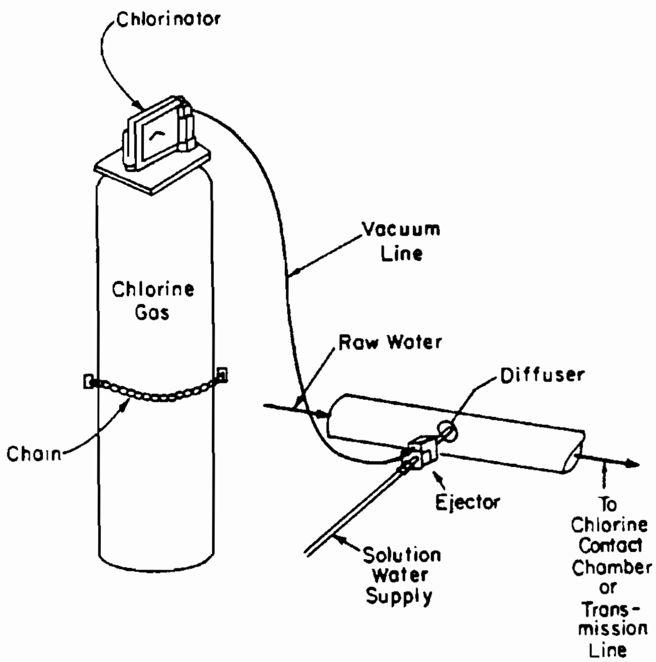


Fig 2.3. Gas injection chlorination system [1].

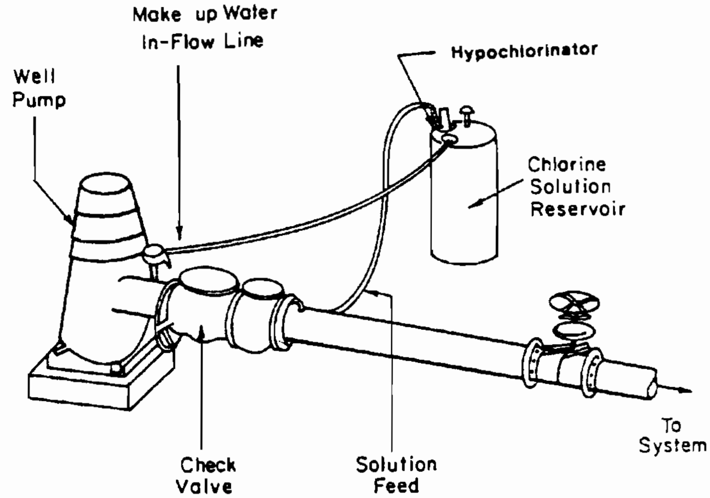


Fig 2.4. Chlorinated water supply system — solution feed [1].

2.2.2 Softening

Water softening is required for waters having a hardness greater than 300 mg/l as $CaCO_3$. Ion exchange systems (zeolite) should be used; treatment with lime is difficult to control because of fluctuating water flows which are experienced at rest areas. Figure 2.5 shows a typical zeolite softening system. Softening costs can be reduced by mixing softened water with raw water to a residual of 75-100 mg/l as $CaCO_3$. Additional information on alternate softening and disinfection methods, well types, well construction requirements, and well draw down and yield can be found in Folks [1].

2.3 WATER DEMANDS AT REST AREAS

2.3.1 Introduction

In order to select components of water supply systems at rest areas the peak instantaneous, peak hourly, and peak daily water demands must be known or estimated. The peak instantaneous demand is used in sizing the mainline pipe that connects all the fixtures and can be used in sizing well pumps. The peak hourly demand is used to calculate storage volumes and to size well pumps. The peak daily demand determines the required capacity of the water system.

At existing rest areas the number of fixtures will determine the instantaneous and hourly demands possible. For proposed rest areas the number of fixtures is determined based on traffic data and on water usage data or estimates of water use. Peak daily demands can be determined by using rest area flow data or from traffic data indirectly.

2.3.2 Water Demands from Flow Data

Daily and hourly peak water demands can be determined from rest area flow data only if a daily hydrograph (which has flow data on an hourly basis) for the peak day of the year is available. A peak daily hydrograph is illustrated in Fig. 2.6. Peak daily demands can be determined by calculating the area under the curve for the day while the peak hourly demand can be determined by calculating the area under the curve for a one-hour period around the peak of the curve. Storage required in the system is the shaded region in Fig. 2.6 and is the area above the average daily demand (peak daily demand/1440 minutes per day) and below the hydrograph curve [1]. The peak hourly demand is also shown in Fig. 2.6.

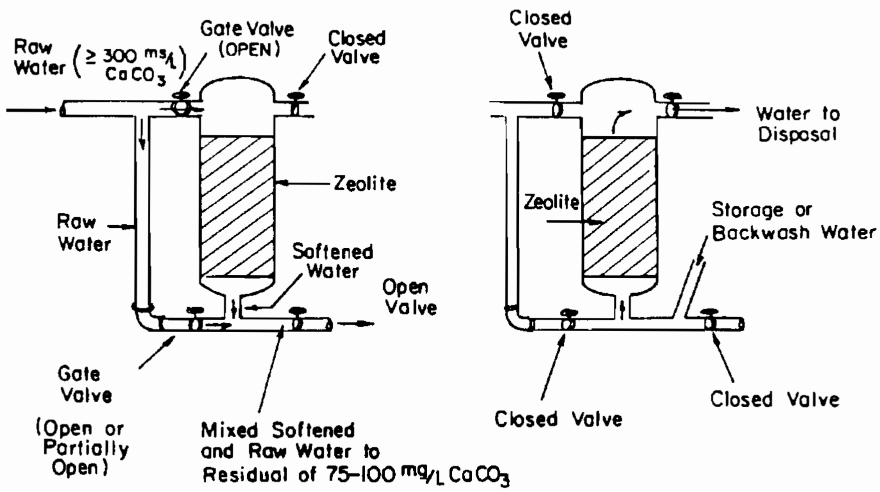


Fig 2.5. Zeolite softening system schematic [1].

If several days of peak demand are experienced consecutively at the rest area (e.g., there is heavy weekend use), then a cumulative mass demand diagram can be constructed to determine storage volumes required at the rest area. In this method daily hydrographs are constructed for each 24-hour period and the cumulative volume demanded is calculated by summing hourly flows cumulatively over the number of peak days (i.e., each hour's flow is added to the previous flow total) and a graph is constructed as shown in Fig. 2.7. The average flowrate is determined by drawing a line from the endpoint of the cumulative demand curve to the origin. The required storage volume for the water system is found by drawing lines parallel to the average flowrate line tangent to the low and high points of the inflow mass diagram; the vertical distance from the lines of tangency represents the storage volume required [2].

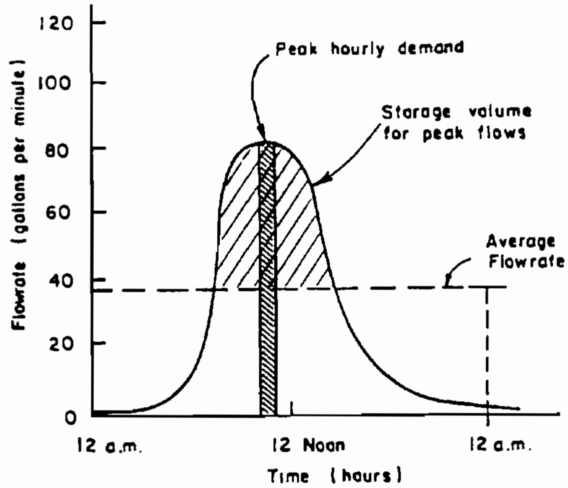


Fig 2.6. Peak daily hydrograph [1].

2.3.3 Calculation of Water Demands Using Indirect Methods

If flow data are not available or cannot be monitored at an analogous rest area, one of the following methods can be used to calculate water demands.

2.3.3.1 The Fixture Method. The fixture method was devised by Hunter [3] in the 1940's. This method is easy to use and the required data are (1) the number of fixtures in a water system and (2) the water demands of each fixture type. The method can be used to calculate peak instantaneous demands for existing rest areas or for proposed rest areas based on the number of fixtures to be installed. Hunter used statistical analysis to develop a relationship between peak instantaneous demand and fixture units. In this method each fixture is assigned a finite number of fixture units (a fixture unit = 7.5 gpm) and the total units are summed for the rest area. The peak instantaneous demand is found by using Table 2.1 and Fig. 2.8.

Johnson demonstrated the use of this method in 1969 to estimate water demands at rest areas in Iowa [4]. Table 2.1 and Fig. 2.8 are used in the following way to determine instantaneous demand for the rest area near San Marcos:

Given:	6 toilets,	Solution:	total fixture units
	2 urinals,		(from Table 2.1) =
	2 sinks, and		6(10) + 2(5) + 2(2) +
2 drinking fountains			2(1) = 76 fixture units

From Fig. 2.8, the peak instantaneous demand for 76 fixture units equals ~62 gpm.

If water saving devices are used at the rest areas, the fixture units assigned to each fixture will be different than in Table 2.1. For example, a water saving toilet that uses two gallons of water and flushes in four seconds has a flowrate of 30 gpm. If this value is divided by 7.5 gpm (Hunter's fixture unit demand equivalent) then each water saving toilet has 4 fixture units. In the example above the fixture units for the toilets would be 6(4) instead of 6(10), total fixture units would then be 40, and the resulting peak instantaneous demand would be 50 gpm.

2.3.3.2 Maximum Demand Using Toilets as the Control. The peak hourly demand at an existing rest area can be determined by using the fact that the number of fixtures (toilets and urinals) installed and operated in a facility will determine the maximum number of people that can be cycled through the rest room. The following equation can be used to calculate the peak hourly demand (PHD):

$$PHD = UPH \times NF \times GPU \qquad \text{(Eq. 2.1)}$$

where

UPH = users per hour
NF = number of fixtures (urinals + toilets)
GPU = gallons per user (maximum water use per fixture—usually the toilet).

The state of Minnesota uses this method to calculate peak hourly water demands at rest areas, based on a UPH of 30 persons per hour and a GPU of 3 gallons (water saving toilets). This value for UPH usually is recommended and was verified by usage studies in Washington [8]. Outside demands, such as lawn sprinkling, must be included in the peak hourly demand (sprinkler demand can be approximated as 5 gpm.)

2.3.3.3 The Zaltzman Method. The Zaltzman method [6] is useful in calculating peak daily demands for existing or proposed rest areas and peak hourly demands at proposed rest areas. The method is shown in Appendix A, with the peak daily demand corresponding to WATER 24 and the peak hourly demand corresponding to PK VOL 1. Note that this method requires knowledge of traffic data and peaking factors.

2.4 PRESSURE REQUIREMENTS, STORAGE, AND PUMP SELECTION

Water pressure in rest area water systems should be between 20 and 65 psi, with 40 psi recommended. A minimum of 20 psi is needed so that all fixtures can clear and automatic flush valves will shut off. Water pressure can be maintained by elevated storage tanks, non-elevated tanks with pumps, or hydropneumatic tanks.

2.4.1 Elevated Storage

The bottom of gravity tanks must be a minimum of 50 feet above the rest area. At this elevation the water pressure will be 22 psi (1 psi per 2.307 feet of elevation at standard conditions). To provide 40 psi the bottom of the tank must be 100 feet above the rest area. Head losses in the piping system must be included in determining the elevation required. The storage tank should be sized to include the percentage of the peak daily demand that is in excess of the daily average demand. This procedure allows for the use of a pump with a lower capacity. Vents and protection against freezing are required for gravity storage tanks. A sketch of a typical gravity tank is shown in Fig. 2.9. Gravity storage tanks are

TABLE 2.1. WATER DEMAND LOAD OF FIXTURES, PUBLIC OCCUPANCY [4]

Fixture	Supply Control	Fixture Unit (1)
Water closet	Flush valve	10
Water closet	Flush tank	5
Urinal	Flush valve	5
Urinal	Flush tank	3
Lavatory	Faucets	2
Service sink	Faucets	3
Drinking fountain	Valve	1

(1) The given weights are for total demand. For fixtures with both hot and cold water supplies, the weights for maximum separate demands may be taken as three-fourths of the listed demand for supply.

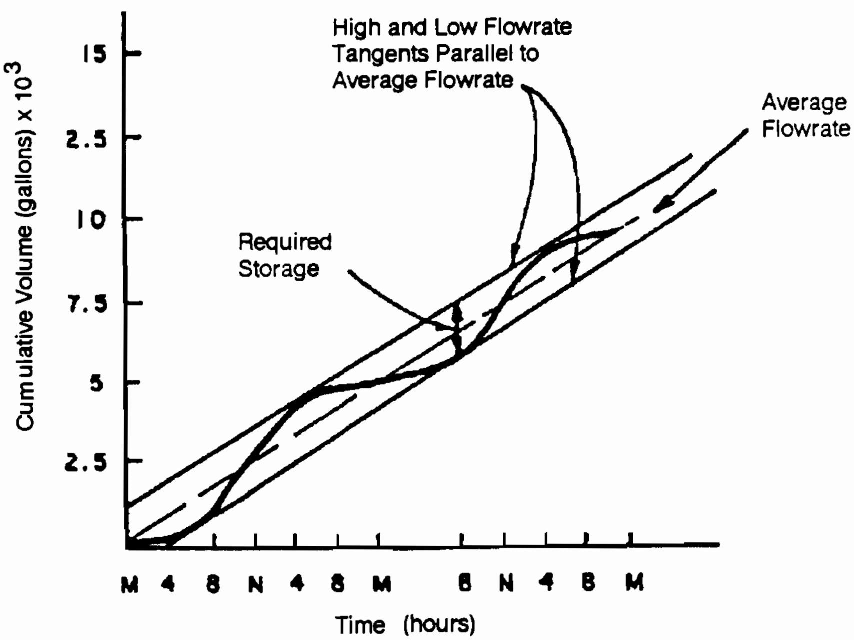


Fig 2.7. Cumulative mass inflow demand diagram [2, p 190].

less sensitive to variable and peak-type water demands than are hydropneumatic tanks.

2.4.2 Non-elevated Storage Tanks

Non-elevated tanks are used in flat terrain when a water source cannot provide enough water for peak hourly demands. Water pressure can be maintained directly by pumps or by use of a hydropneumatic tank. Tanks should be installed underground as shown in Fig. 2.10 or enclosed in a protective building. Tanks should hold at least the percentage of peak daily flow in excess of the average daily flow. Figure 2.11 shows some vent and overflow configurations that are used for non-elevated storage tanks.

2.4.3 Hydropneumatic Tanks

Hydropneumatic tanks are included among the methods most commonly used to meet pressure requirements at rest areas. Compressed air is used to maintain the water pressure while the pump operation cycle is controlled by the water level in the hydropneumatic tank. When water is used the air pressure inside the the tank drops as the water level falls. When the water level falls below a prescribed level, which is associated with a minimum pressure, the pump is activated. The storage available in the tank is that volume of water contained between the maximum and minimum allowable water levels in the tank. Figure 2.12 depicts a typical hydropneumatic tank.

The required capacity of the pump(s) supplying the hydropneumatic tank depends on the nature of the water demand at the rest area. Johnson [4] suggests that there is no appreciable storage in the tank and that the pumps supplying the tank should be sized for the peak instantaneous demand. The state of Minnesota [5] uses peak hourly demand to size pumps because peak instantaneous demands occur very infrequently at the rest areas. The well pumps at new rest areas in Minnesota, which are designed for peak hourly demand, have not created difficulties in five years of operation [5]. In general, if peak instantaneous demands are expected frequently, the pumps should be designed to deliver water to meet the peak instantaneous demand. Rest areas with heavy commuter traffic may require this design based on instantaneous demand; otherwise peak hourly demand should be used to size the pumps.

The sizing of the hydropneumatic tank depends on the usable storage volume in the tank and pump cycling times. The usable storage volume can be calculated by the following formula:

$$\text{Cycle time (min) b} = V/(P-Q) + V/Q \qquad \text{(Eq. 2.2)}$$

where

V = usable volume (gallons)
P = pumping rate (gpm)
Q = water demand (gpm)

In Eq. 2.2 the cycle times are based on allowable starts per hour for the well pump (cycle time = 60/starts per hour). The pump size and characteristics determine the pumping rate (P) and the cycle times that can be tolerated by the pump. The lower end of the allowable range of starts per hour should be used for a conservative design. The maximum

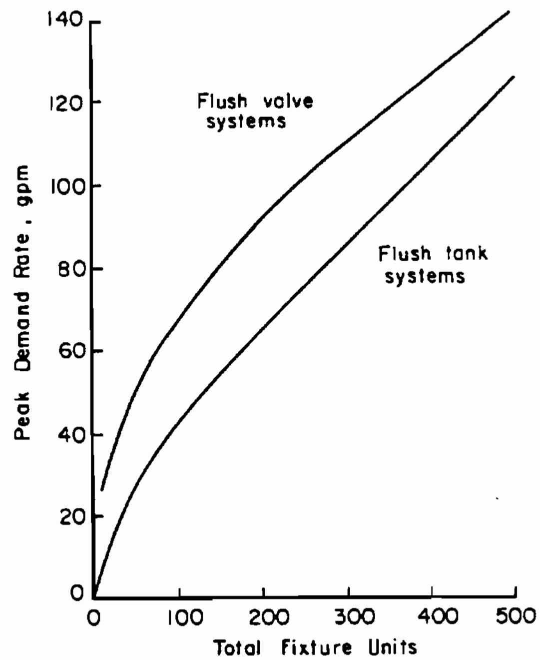


Fig 2.8. Demand loading estimate curves [3].

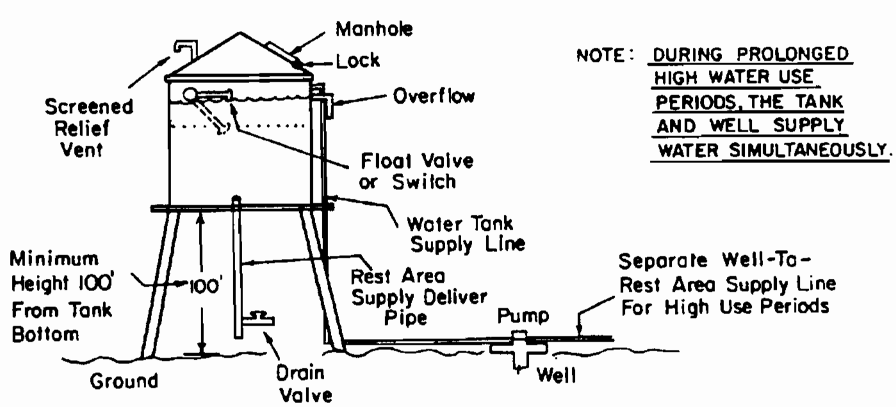


Fig 2.9. Typical gravity storage tank for flat terrain [1].

cycle time will occur when the water demand is half the pumping rate, and Eq. 2.2 becomes

$$V = (P \times \text{cycle time})/4 \quad \text{(Eq. 2.3)}$$

The state of Minnesota uses a cycle time of 4 minutes (15 starts per hour) for 5 HP pumps, which gives a usable volume equal to the pumping rate. The usable volume is used to calculate the volume of the tank. A storage tank and booster pumps will be required in the system if the source of the water supply cannot meet the peak instantaneous or peak hourly demand.

In addition, in order to get more water per flush the hydropneumatic tank(s) should be pre-charged with air. According to Folks, pre-charging with air can increase water delivery by 21 percent [1]. The bladder type tanks are recommended because the bladder prevents air from dissolving into the water, which causes loss of pressure and release of gas in the pipes.

2.5 FIXTURE CALCULATIONS

The total number of fixtures required at a proposed rest area can be calculated as

$$T = (ADT \times UV \times DH \times PF \times P)/UHF \quad \text{(Eq. 2.4)}$$

where

- T = Total number of fixtures
- ADT = Average daily traffic (veh/day)
- UV = Rest room users per vehicle (users/veh)
- DH = Design hour usage/design day usage
- PF = Peak factor (peak daily usage/ADT)
- P = Percent mainline traffic stopping(decimal)
- UHF = Rest room users per fixture per hour (users/fixture/hr).

The state of Minnesota uses a DH of 0.15, a PF of 1.8, and a UHF of 30. These values are based on traffic data collected in Minnesota over a number of years. The design hourly usage is the peak hourly usage and is the factor used in the Zaltzman approach to find PK VOL 1. DH, PF, and UHF values should be determined for individual states; however, Minnesota values can be used for Texas rest areas until more traffic data on Texas rest areas are obtained. The distribution of urinals and toilets of the total fixtures (from Eq. 2.4) is shown in Appendix B.

2.6 TEXAS REST AREA WATER SUPPLY

Texas has 35 pairs of rest stations on Interstate highways, five pairs of rest areas on U. S. highways, and 14 U. S. highway locations with one rest area serving both directions and two rest areas serving only one direction. Well water is

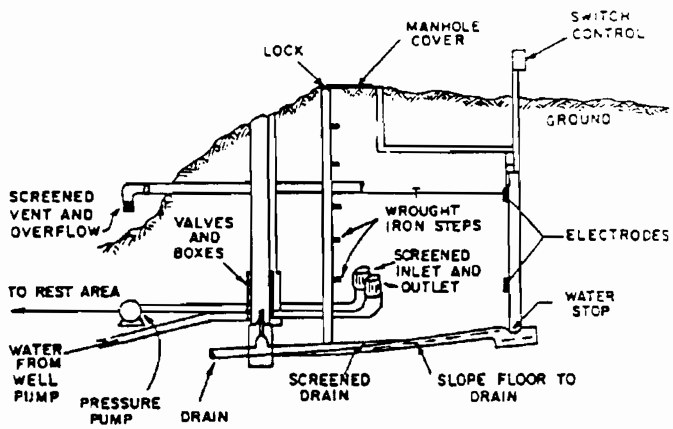


Fig 2.10. Underground non-elevated storage tank [1].

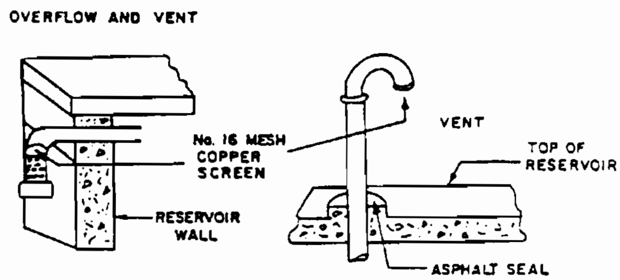


Fig 2.11. Vent and overflow configurations for underground storage tanks [1].

used as a water supply at twenty-seven rest area sites, and twenty-seven rest area sites have municipal water supplies available. The water supply source for one rest area (on U. S. 290) was not identified in the records of the Texas SDHPT.

Many of the rest areas served by municipal water supplies actually are being served by rural water districts, which may not always supply a constant amount of water. Several rest areas which obtain water from rural water districts have experienced water shortages during high use periods in the summer, specifically, Interstate rest areas in Kaufman, Callahan, Nolan, Medina, and Ellis counties. In order to alleviate the problem the Texas Department of Highways and Public Transportation has installed water storage tanks at these rest areas. Thus, it is important to know the ability of the municipality to meet peak summer water demands that can be expected at the rest area.

A rest area in Bowie county is experiencing water shortages at the present time and a storage tank-hydropneumentic system is under construction. The system will utilize gas injected chlorine as a disinfectant method. The total cost of improvements at the rest area is estimated at \$80,000 [7]. The chlorination system cost is \$1,500, the cost of the hydropneumatic tank is \$2,000, and the cost of two 7.5 HP pumps is \$4,000. Similar systems are operating satisfactorily at rest areas in Nolan and Callahan counties [7].

Water demands at rest areas in Texas are estimated by a method developed by the Department of Civil Engineering at The University of Washington and used by the Washington State Department of Highways [8] (1972). Peak hourly demands are calculated as

PEAK HOURLY DEMAND

$$= A \times B \times C \times D \times E \times F \qquad \text{(Eq. 2.5)}$$

where

- A = Average daily traffic, ADT(veh/day)
- B = Percent vehicles entering per day (decimal)
- C = Peak hour as percentage of ADT (peak hour vehicles/daily vehicles)
- D = Number of persons per vehicle(occupants/veh)
- E = Percent people using rest-rooms (decimal)
- F = Water use per person (gal/person).

The state of Washington uses a C value of 0.12, which is similar to the 0.15 value reported by Zaltzman. The value for B of 0.12 used in the Washington method is larger than the 0.09 figure used by Zaltzman. The Washington method does not calculate a peak daily demand but uses a peaking factor instead. Thus, the Zaltzman method is comparable to the University of Washington's method. In addition, the Zaltzman method can be used to calculate peak daily water demand using the 6 peak weekend traffic ADT.

The number of rest room fixtures required in rest rooms in the state of Texas is determined from charts developed by the state of Washington. These charts are based on persons per hour using the rest rooms and directly give the number of toilets and urinals. The state of Minnesota (1977) has developed a more detailed chart that is more recent than the Oregon chart (1972). The total number of fixtures is based on Eq. 2.4, and the fixture distribution versus ADT and percentage stopping is given. The Minnesota Rest Area Design Chart is presented in Table 2.2; the chart is used to determine fixture distributions.

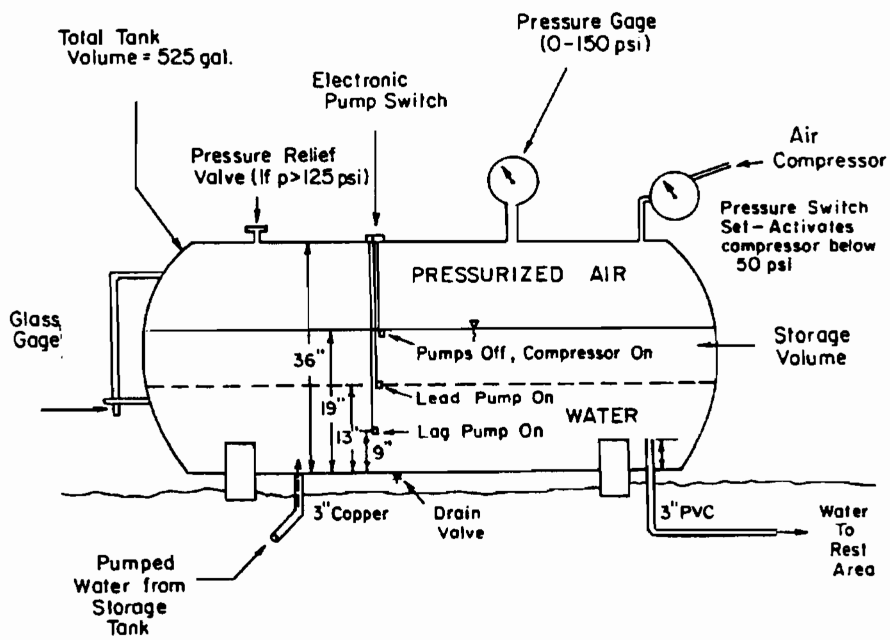


Fig 2.12. hydropneumatic pressure tank — Bowie County rest area, Texas [7].

TABLE 2.2. MINNESOTA REST AREA DESIGN CHART

MINNESOTA REST AREA DESIGN CHART

PROCEDURE FOR USING CHART

1. ENTER CHART AT PERCENT STOPPING BEING USED FOR DESIGN
2. DROP DOWN VERTICALLY TO ADT BEING USED FOR DESIGN
3. FOLLOW CHART TO RIGHT FOR DESIGN COMPONENTS

ADT AND PERCENT STOPPING AT REST AREA (1.)											FIXTURE DISTRIBUTION (2.)						WATER SYSTEM DESIGN	
											TOTAL	MEN (3.)			WOMEN (3.)		MINIMUM FLOW TO PRESSURE TANK	MINIMUM DRAWDOWN
% STOPPING 6	7	8	9	10	11	12	13	14	15	16	URINALS TOILETS	URINALS	TOILETS	HAND DRYERS WASH BASINS	TOILETS	HAND DRYERS WASH BASINS	GALLONS/MIN (4.)	GALLONS (5.)
8547	7326	6410	5698	5128	4662	4274	3945	3663	3419	3205	6	1	2	2	3	2	19	19
9972	8547	7479	6648	5983	5439	4986	4602	4274	3989	3739	7	2	2	3	3	2	21	21
11396	9768	8547	7597	6838	6216	5698	5260	4884	4558	4274	8	2	2	3	4	2	22	22
12820	10989	9615	8547	7692	6993	6410	5917	5495	5128	4808	9	2	3	3	4	2	24	24
14245	12210	10684	9497	8547	7770	7123	6575	6105	5698	5342	10	2	3	4	5	3	25	25
15670	13431	11752	10446	9402	8547	7035	7232	6716	6268	5876	11	3	3	4	5	3	26	26
17094	14652	12820	11396	10256	9424	8547	7890	7326	6838	6410	12	3	3	4	6	3	28	28
18519	15873	13889	12346	11111	10101	9259	8547	7937	7407	6944	13	3	4	5	6	3	29	30
19943	17094	14957	13295	11966	10878	9972	9204	8547	7977	7479	14	3	4	5	7	4	31	32
21368	18315	16028	14245	12821	11655	10684	9862	9158	8547	8013	15	4	4	5	7	4	32	34
22792	19536	17094	15195	13675	12432	11396	10519	9768	9117	8547	16	4	4	6	8	4	34	37
24216	20757	18162	16144	14530	13209	12108	11177	10379	9687	9081	17	4	5	6	8	4	35	39
25641	21978	19231	17094	15385	13986	12820	11834	10989	10256	9615	18	4	5	7	9	5	37	41

(1.) $ADT = T\left[\dfrac{(UHF)}{(UV)\,(DH)\,(PF)\,(P)}\right]$ OR $T = \dfrac{(ADT)\,(UV)\,(DH)\,(PF)\,(P)}{(UHF)}$

(2.) DISTRIBUTION IS BASED ON USAGE ACTIVITY AND CYCLING TIMES

(3.) QUANTITY SHOWN IS NOT A TOTAL, BUT THE NUMBER OF EACH REQUIRED

(4.) THE FLOW RATES INDICATED ARE THE MINIMUM REQUIRED FOR THE PUMP SUPPLYING THE PRESSURE TANK WHERE WATER SAVING TOILETS (2-3 Gal. per Flush) AND URINALS (1 Gal. per Flush) ARE USED. A 10 G.P.M. DEMAND FOR AN ON-SITE WATER DISTRIBUTION SYSTEM IS INCLUDED.

(5.) DRAWDOWN INDICATED IS FOR 1 ø OR 3 ø MOTORS FOR WELL PUMPS OR BOOSTER PUMPS UP TO 5 H.P. IT IS BASED ON THE LONGER DRAWDOWN DETERMINED BY A MAXIMUM 15 CYCLES per HOUR OR 150 CYCLES per DAY. THE DESIGN PRESSURE RANGE IS 40-60 P.S.I.

CHAPTER 3. REST AREA WASTEWATER TREATMENT SYSTEMS

3.1 INTRODUCTION

Historically most rest area wastewater systems have been either pit privies or septic tanks with drainfield systems. These systems have performed well in some areas and poorly in others, depending on soil types, waste loadings, and other factors. In the late 1960's and early 70's exploration and use of new rest area wastewater systems was begun in response to old system failure. The strengths and weaknesses of the various wastewater systems used at rest areas are discussed and the circumstances under which a particular system can be applied are identified. Rest area wastewater treatment systems must (1) be designed for low capital, operating, and maintenance costs, (2) discharge an effluent that meets federal or state quality standards, (3) not pollute groundwater, and (4) not cause unacceptable odors.

3.2 WASTEWATER CHARACTERISTICS AT REST AREAS

The composition of rest area wastewater is variable because of the nature of rest area usage. Slug flows are common during heavy daytime use while at night and in the early morning flows can be low or non-existent. Compared to domestic wastewaters, rest area wastewaters contain higher concentrations of nitrogen, chemical oxygen demand (COD), and settleable solids but have lower concentrations of phosphorus, suspended solids (SS), and biological oxygen demand (BOD) [8]. Scum and grease are usually absent in rest area wastewaters [8]. The higher concentrations of COD and settleable solids can be accounted for by the paper content in rest area wastewaters while higher nitrogen levels are caused by a high percentage of urine in the wastewater.

These characteristics were described by Sylvester & Seabloom in 1972 [8]. Subsequent studies made at rest areas have supported these results. Wastewater characteristics reported in several studies conducted at rest areas are listed in Table 3.1. The table also includes characteristics for domestic wastewaters. Texas state domestic wastewater effluent standards for BOD and TSS are presented in Table 3.2.

High nitrogen in rest area wastewater levels is of special concern because nitrogen in the ammonium form (NH_4+) can be converted to ammonia (NH_3) or nitrate (NO_3). Ammonia can cause odor problems, and nitrates are a potential groundwater pollutant. In addition, nitrification processes (conversion of ammonia to nitrate) require oxygen, which may be a problem in treatment systems such as recycle and/or package plant systems.

In general, rest area wastewater should not present any major treatment difficulties. In most cases toxic materials are not found in rest area wastewaters. If recreational vehicles are served by the rest area there is the potential for upset of the treatment systems caused by formaldehyde in RV wastes.

3.3 WASTEWATER FLOWRATES AT REST AREAS

At present wastewater flowrates at rest areas are estimated in terms of gallon per person or gallons per vehicle. The state of Washington uses 3.5 gal/person for both water use and wastewater production [8]. Zaltzman recommends

TABLE 3.1. MEAN REST AREA UNTREATED WASTE WATER CHARACTERISTICS (mg/L EXCEPT pH)

Investigator	SS mg/L	BOD mg/L	COD mg/L	TKN mg/L	pH
Sylvester & Seabloom (1972)[1]	165	165	405	140	8.3
Pfeffer (1973)	149	150	-	-	-
Parker (1972	197	176	579	85	8.6
Zaltzman (1975)	199	166	344	-	7.6
Jenkins (1976)	~60	130	-	-	-
Hughes & Averett (1977)[2]	140	124	-	24	7.8
Metcalf & Eddy (1979)					
Domestic - Weak	100	110	250	20	
- Medium	220	220	500	40	
- Strong	350	400	1000	85	

Notes: 1 - Three Values Taken on Three Sample Days at Four Rest Areas
2 - Mean Values for Five States

TABLE 3.2. BOD AND TSS EFFLUENT STANDARDS FOR DOMESTIC WASTEWATER TREATMENT PLANTS IN TEXAS (mg/L)

	30 Day Avg		7 Day Avg		24 Hr Comp	
Effluent Set	BOD	TSS	BOD	TSS	BOD	TSS
X	30	90	45	-	70	-
0	30	30	45	45	70	70
1	20	20	30	30	45	45
2	10	15	15	25	25	40

Notes: X = Oxidation ponds as sole treatment process, capacity < 2 MGD, and best waste stabilization pond technology.
0 - Other Oxidation
1 - Secondary Treatment
2 - Modified Secondary Treatment (Enhanced Solids Separation)

Source: "Texas Natural Resource Reporter," Research and Planning Consultants, 1705 Guadalupe, Austin, Texas, 1981

5.5 gal/vehicle for wastewater production, based on his study in West Virginia [6]. Hutter [36] found a wastewater production rate of 1.26 gal/person in his Colorado study on low flush toilets. The state of Minnesota uses rates of 5.5 gal/veh and 3.6 gal/veh for conventional and low flush toilets, respectively [5].

Obviously flows vary from site to site so that the best guide for design is flow data from an analogous rest area, if available. If data are not available, Zaltzman's method (Appendix A) should be used to calculate daily wastewater flows. Sylvester & Seabloom recommend using a fixture method to calculate average daily flows based on the assumptions of 30 users/hr/fixture, 3.5 gal/user, and a peaking factor of 0.12 (peak hourly/ADT) [8]. The Zaltzman method avoids the use of a peaking factor and calculates a peak daily flowrate. This method provides a more accurate estimate of maximum flows and should be used for design.

In general, wastewater flows can be found from water use rates if outside water uses (irrigation and drinking water) are subtracted from the total water use. Hutter [36] found strong correlations between wastewater flows and vehicles entering the rest area; thus metered flowrate data can be used to make correlations which can be useful for future rest area design or for designing improvements at existing rest areas.

3.4 SEPTIC TANK/DRAINFIELD SYSTEMS

3.4.1 Description of System

Septic tank/drainfield systems consist of a tank and a drainfield. The tank usually has two compartments, has various baffles or tees, and is made of steel, cement, or plastic. Some typical septic tank configurations are illustrated in Fig. 3.1. The drainfield usually consists of a number of trenches (laterals) into which perforated pipes are laid for distribution of the septic tank effluent to the soil. Drainfields are usually laid out in parallel grid patterns but can be put in series. The septic tank removes settleable solids and floating solids. The volume of the solids is reduced by anaerobic digestion, and methane and hydrogen sulfide gases are produced. The septic tank effluent then flows to the drainfield, where physical (straining), chemical (adsorption, ion exchange, and precipitation), and biological (microbial) processes remove the pollutants from the water. Sludge and scum accumulate in the septic tank and require periodic pumpout of the tank.

3.4.2 System Performance

Septic tank/drainfield system performance is a function of septic tank density per area, system design, and drainfield soil and geologic characteristics. System failures usually are caused by (1) nitrate migration to groundwaters, (2) septic tank undersizing, or (3) drainfield clogging.

3.4.2.1 Nutrient Removal and Nitrate Migration. Sylvester & Seabloom studied septic tank removal performance and reported reductions of 62, 43, and 20 percent for suspended solids (SS), chemical oxygen demand (COD), and biological oxygen demand (BOD), respectively [8]. Drainfield removal of constituents after 5 feet of vertical effluent travel through the drainfield soil can be summarized as follows [9, p. 57]:

Constituent	Percentage Reduction
TSS, COD, BOD	70-90
Phosphate	25-50
Ammonium	80-90

Organic nitrogen compounds in the septic tank slowly mineralize to ammonium (NH_4^+) so that the tank effluent contains about 75 percent of the nitrogen in the NH_4^+ form and 25 percent in organic forms [10, p. 59]. The ammonium in the tank effluent is converted by nitrification to nitrates (NO_3^-) if aerobic conditions exist in the drainfield. This ni-

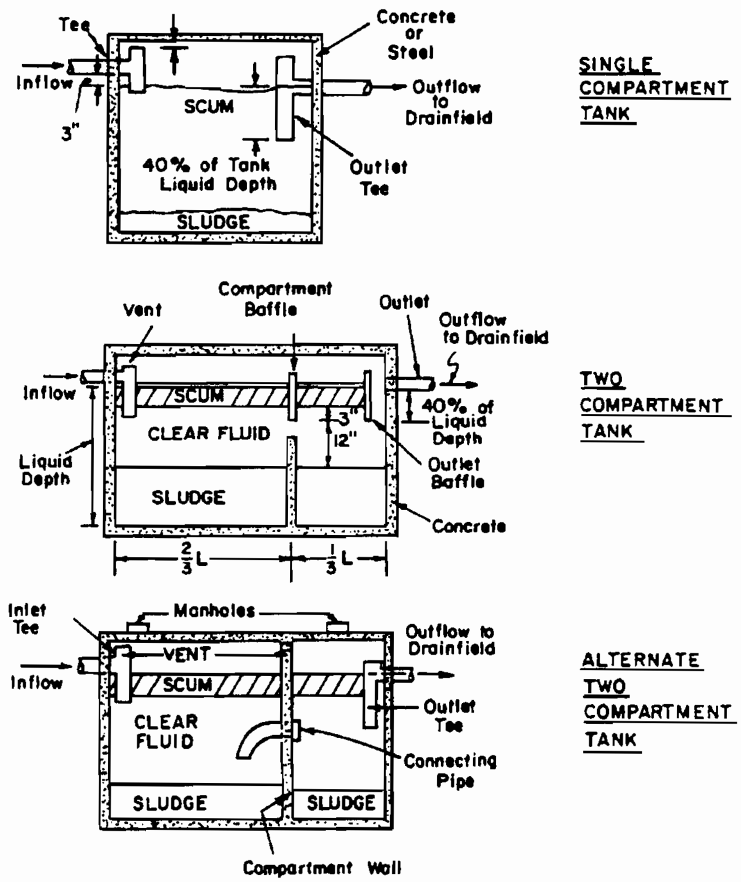


Fig 3.1. Typical septic tank configurations [9].

trification process usually occurs a few inches below the soil-water infiltration surface in the drainfield trenches. Therefore, most of the ammonium leaving the tank eventually is converted to nitrates, which are mobile in the soil and may reach the groundwater.

Palta tracked nitrate movement from a leachfield and reported a decrease in concentration from 30 to 5 mg/l at 43 meters down gradient of the field [10, p. 61]. Pruel (1966) reports that 30 meters down gradient distance is needed in a sandy soil to get nitrate levels below 10 mg/l [10, p. 51]. Denitrification (conversion of nitrate to nitrogen gas) is unlikely to occur in the leachfield system because most soils lack carbon sources and are not in an anoxic condition, but nitrate can be removed from the soil by plant uptake. Conversion of ammonium to nitrate in the drainfield is commonplace; therefore, it is prudent to set up a nitrate monitoring system around the leachfield. A reasonable sampling schedule is given by Cantor [9, pp. 88-91].

3.4.2.2 ***Tank Undersizing.*** Septic tanks are usually designed for 24-hour detention times. If wastewater flowrates are greater than design flowrates, scouring of solids out of the septic tank can occur, causing physical clogging of the drainfield. Extreme waste loadings could clog the tank outlet pipe, resulting in rest room toilet overflows during flushing.

The United States Public Health Service (USPHS) *Manual of Septic Tank Practices (1967)* recommends sizing septic tanks using the following formula [11] :

$$V = 1125 + 0.75Q \qquad \text{(Eq. 3.1)}$$

where

V = Tank volume, gallons
Q = Average daily flow, gallon/day, and
0.75 = Slope of a straight line regression [capacity required (gallons) ∏ sewage flow (gal/day)]

This recommendation was based on a five-year study on septic tanks by the USPHS from 1946-1951 [11]. This equation has been widely used in the United States even though no critical review of its validity could be found in the literature. The states of Texas and Minnesota currently use this equation to size septic tanks [5, 18]. Hughes [13] suggests using this equation, except that Q should be first multiplied by 1.25 as a peaking factor. The state of Washington uses a gallon per fixture per day value of 875 to determine rest area septic tank sizes; this fixture method yields tank sizes of 6,000, 8,000, and 12,000 gallons for 6, 8, and 12 fixtures, respectively.

The problem in using the USPHS formula is that it is unclear whether the equation is valid, i.e., no critical reviews of the formula's validity have been performed. The state of Washington's fixture method requires estimation of peaking factors so that the results are only as good as the estimates made in calculating peaking factors. Zaltzman's WASTE 24 value offers a better approach for an estimate of flow to use in sizing tanks. The tank size can be adjusted to reflect future flowrates and to estimate cost differences between different size tanks.

Solids overflow can also be caused by short circuiting flow in poorly designed septic tanks without baffles or tees. Compartment size ratios, arrangement of baffles and tees, and liquid depth limitations are some of the parameters used in tank design and are covered in the USPHS manual. Gas baffles should be incorporated into the design to prevent gas from entering the drainfield distribution system; these devices are relatively low in cost and can be incorporated into the design with little difficulty [14].

3.4.2.3 ***Drainfield Clogging.*** Conventional drainfields usually are designed on the basis of volumetric loadings expressed in terms of gallons per square foot per day. The surface area needed for a drainfield is designed using the results of percolation tests and the volumetric loadings. Ponding effluent in the drainfield is caused by (1) hydraulic overloadings and/or (2) inadequate effluent distribution throughout the field.

The USPHS manual has been and still is a widely used design manual. Some of the design criteria suggested in the 1967 manual are listed in Table 3.3. It is important to realize that no single manual can be prepared to fit all local situations. Proper design involves the use of geologic, soil, and environmental analyses.

3.4.2.3.1 Drainfield Clogging Due to Loadings. Loadings to a drainfield can be volumetric, time, or waste strength loadings. For volumetric loadings, effluent is applied to the field on a volume per area per day basis; for time loadings, dosing and resting periods are used when effluent is applied to the field; and, for waste strength loadings, concentrations of certain wastewater parameters, such as TSS or BOD, are used to calculate application rates. It has been found that pretreatment of tank effluent has little effect on clogging [15] and, so, reduction of BOD and TSS concentrations in septic tank effluent is not a solution to a drainfield clogging problem. Thus, volumetric and time loadings are the important parameters in designing drainfields.

3.4.2.3.1.1 Volumetric Loadings & the Percolation Test. The USPHS 1967 manual recommends using the following method to size drainfields:

1. Perform an onsite percolation test.
2. If the percolation test results are above 60 or below 1 min/in., do not use a septic tank/drainfield system.
3. If the percolation rate is acceptable, calculate the sewage application rate (Q_a) in gal/sq ft/day using Frederick's formula :

$$Q_a = 5/t$$

where

t = percolation rate in min/in.

4. Find the required trench bottom area using

$$A = Q/Q_a$$

where

A = trench bottom area (sq ft)
Q = average daily flow (gal/day)
Q_a = sewage application rate (gal/sq ft/day)

5. Design trenches.

Henry Ryan developed the percolation test in 1928 [16]. Frederick developed the formula used in step 3 based on Ryan's original data, and Ryan's data on percolation rates ranged only from 4 to 45 min/in. [17]. The USPHS extended the acceptable range of percolation rates to 1 to 60 min/in. even though the Frederick formula is valid only in the 4 to 45 min/in. range [15, pp. 80-81]. In addition to this the USPHS 1967 *Manual of Septic Practices* has several other problems.

First of all, the percolation test procedures in the manual contain such ambiguity that the results are extremely variable. According to Winneberger, Ryan's original procedures were significantly modified, making the USPHS method liberal and capricious. A good treatise on the flaws in the USPHS test is given by Winneberger [15, pp. 31-56]. The main flaws of the test are (1) imprecise procedures and (2) large errors in field measurements. These two factors combine to produce low test reproducibility. Winneberger suggests using a modification of Ryan's procedure and has developed an equation relating hydraulic conductivity (k) to percolation rates (for test procedures and percolation-conductivity equation see Appendix C) [15]. The applicability of Winneberger's equation in Appendix C is restricted to saturated mediums.

The USPHS manual also delineates maximum and minimum percolation rates acceptable for drainfields; these are still mandated today (with slight revisions) by the U.S. Environmental Protection Agency [9, p. 31]. Winneberger presents an argument against the 1 min/in. lower test limit, noting that the Santee aquifer (mostly stony soil with sandy loam) in Santee, California, has been used to reclaim secondary sewage effluents for years despite an observed percolation rate of 0.21 min/in. [15, pp. 148-149]. Thus, the cleansing properties of a soil medium can be as important as percolation rates in choosing site suitability. Winneberger also questions the upper percolation rate limit of 60 min/in. by noting that in San Mateo County, California, soils with percolation rates as high as 80 min/in. have been successfully used [15, p. 58].

The USPHS manual uses trench bottom area only in sizing drainfields; this may or may not be appropriate for a particular site. The use of side wall area or a fraction of side wall area will reduce the size of the drainfield needed and appears appropriate for soils that have uniform hydraulic conductivities in the vertical and horizontal directions. The sizing of the field should be based on the *peak* daily flowrate. Clogging of many fields probably is started during peak flow periods.

Lastly, a major problem with the percolation test is that it measures saturated infiltration rates, which *do not* reflect physical processes in a drainfield. In virtually all drainfields a biological mat will form within 2 to 3 inches of the soil-wastewater interface, and the soil beneath the mat, which is the infiltration media of interest, will be *unsaturated* [10, p. 111]. According to Kreissal, the flow rate through the soil is governed by the unsaturated hydraulic conductivity, which is not measured by the percolation test and is generally lower than the saturated hydraulic conductivity.

Bouma [18] has devised an *in situ* test for measuring vertical hydraulic conductivity in unsaturated soils (see Appendix D). Using the test results, a graphical relationship between hydraulic conductivity and soil moisture can be developed for different soil types. Graphs for four soil types tested in Wisconsin are shown in Fig. D.3 in Appendix D. Bouma used measured soil moisture potentials to estimate

TABLE 3.3. COMPARISON OF SEPTIC TANK DESIGN PARAMETERS

Design Parameter	1967 USPHS Manual	1977 Texas DOH Manual
Tank Size	Vol = 1.5 (Q) for Q < 1500 gal/day Vol = 1125 + .75Q for Q > 1500 gal/day	Vol = 2(Q) for Q , 1500 gal/day Vol = 1125 + .75 Q for Q> 1500 gal/day
Dosing Siphon	Yes	Not Mentioned
Pressure Pumps	Not Mentioned	Not Mentioned
Trench Lengths	≤ 100 ft	≤ 100 ft
Trench Widths	12-36 in.	> 12 but < 30 in. 12 - 18 Recommended
Trench Depths	≥ 24 in.	≥ 18 but ≤ 36 Shallow as Possible
Trench Fill		
Gravel Below Pipe	6 in.	6 in.
Above Pipe	2 in.	2 in.
Pervious Barrier	2 in. Hay or Stray	Straw or Butcher Paper
Sand	Not Mentioned	Use if Trench Depth Is > 24 in.
Topsoil	12 in.	6 - 12 in. Max.
Trench Pipe Size	4 in.	3 - 6 in.
Sewage Application Rate (gal/sq ft/day)	$Q_a = 5/t$ t = Percolation Rate (min/in.)	Use Chart Based on Percolation Rate or Soil Type
Trench Center Separation Distance	6 ft	5 ft from Edge of One Trench to Another

TABLE 3.4. RECOMMENDED MAXIMUM LOADING RATES FOR SEPTIC TANK SOIL ABSORPTION FIELDS BASED ON IN SITU MEASUREMENTS[1] [10]

Soil Texture	Loading Rate[2], cm/day (gpd/sq ft)
Sand	5 (1.2)
Sandy Loams	3 (0.70
Loams	2 (0.50
Silty Loams and Some Silty Clay Loams	5 (1.2)[3]
Clays	1 (1.2)[3]

Notes: 1 - Assumes that the high water table is > 90 cm (3 ft) below the infiltration surface.
2 - Bottom Area Only
3 - Should not be applied to soils with expandable clays.

infiltration rates for the four different soils and has proposed maximum loading rates for the soils. These rates are shown in Table 3.4.

This method is sound but expensive for individual sites. In addition, few engineers are familiar with this approach. To use the approach, soils must be classed and hydraulic conductivities correlated with percolation test results for the same soil type. With a correlation established the percolation test can be used to satisfy governmental requirements while the drainfield can be sized according to the unsaturated hydraulic conductivity obtained from the correlation.

3.4.2.3.1.2 Time Loadings. Soil clogging is related to time as well as volume loadings. Sequential dosing and resting periods seem to prolong the life of a drainfield by allowing aerobic biological decomposition of the biological mat in the soil, thus reducing resistance to effluent infiltration into the soil [9, p. 36]. The success from resting a field is a function of the degree of clogging at the beginning of the resting period and the length of the resting period. If a large anaerobic biological mat is present aeration will deteriorate it, but clogging can still occur because of the growth of aerobic organisms during decomposition of the old mat, i.e., a new mat replaces the old mat [10, p. 45]. In sands, restoration of infiltration surfaces requires three to four weeks; however, fine textured soils will probably require months [10].

Alternate dosing of drainfields is the most practical way to provide resting periods. Use of one field with an additional holding tank to store rest area wastewater while the soil rests is unacceptable because of odors associated with storage of the wastewater and because the tank size needed to store the wastewater would be too large. Alternate dosing of fields allows normal functioning of the septic tank and provides ample time for resting. There is much debate concerning dosing and resting cycles, with cycles ranging from as short as one week [13] to as long as one year [15]. The optimal resting period length is not known but testing soil cores for moisture content after different resting periods may be a way to establish an appropriate time period [13].

In the past it has been accepted that septic tank/drainfield systems have a finite life span, but it should be possible to determine an application rate which will not exceed the

TABLE 3.5. RYAN'S PERCOLATION RATES RELATED TO VARIOUS PARAMETERS OF LOADING RATES OF SEPTIC-TANK EFFLUENTS ONTO SOILS OF DISPOSAL TRENCHES [15]

Coefficient of Permeability (ft/min)	Percolation Rate (min/in)	Loading Rate ($gal/ft^2/day$): Bottom Area Ryan(16)	Bottom Area Frederick(17)	Sidewall Area	Functional Area	LTAR
2.0×10^{-2}	1	4.0	5.0	3.0	1.7	0.80
6.7×10^{-3}	2	3.2	3.5	2.4	1.4	0.58
3.6×10^{-3}	3	2.8	2.9	2.1	1.2	0.51
2.3×10^{-3}	4	2.4	2.5	1.8	1.0	0.46
1.6×10^{-3}	5	2.2	2.2	1.7	0.95	0.44
5.5×10^{-4}	10	1.7	1.6	1.2	0.72	0.37
2.9×10^{-4}	15	1.3	1.3	1.0	0.57	0.34
1.0×10^{-4}	30	0.84	0.91	0.63	0.35	0.30
5.3×10^{-5}	45	0.62	0.75	0.46	0.26	0.28
3.4×10^{-5}	60	0.48	0.65	0.36	0.21	0.27
2.4×10^{-5}	75	0.44	0.58	0.33	0.19	0.26
1.2×10^{-5}	120	0.23	0.46	0.17	0.10	0.24
9.6×10^{-7}	600	0.08	0.20	0.060	0.03	0.20

Sidewall Area assumed trenches 1 ft wide and with 8 in. of sidewall height.
Functional Area assumed bottom and sidewall areas are equal.
Long-Term Acceptance Rates (LTAR) were calculated on the basis of the Functional Area.

assimilative capacity of the soil over the long run. Laak has proposed a long term acceptance rate (LTAR) for drainfields [15]. This LTAR range is equivalent to percolation ranges of 1 to 600 min/in., as shown in Table 3.5. It is important to note that the LTAR is based on having both side and bottom wall trench areas act as infiltration surfaces. In addition, this method does not take into account vegetative uptake of wastewater in the drainfield.

3.4.2.3.2 Drainfield Clogging Due to Inadequate Distribution. Conventional gravity distribution systems generally suffer progressive clogging. Although laying the pipe level is supposed to provide uniform distribution of effluent to the field trenches, it usually does not [10, 15]. In most cases part of the trench is inundated first and clogs and then this clogging spreads to the rest of the trench as future doses are applied [10, pp. 68-69]. Pressure distribution systems will allow more control over application rates and will provide more uniform distribution. The pipe and the diameter of the holes can be sized to balance the head loss to each hole [10, p. 69]. Rules of thumb for pressure distribution systems are (1) assume 60 to 90 cm (2 to 3 ft) of head at the terminal end of the lateral, (2) assume 65 to 85 percent of the total head loss in the network occurs across the orifice, and (3) assume 10 to 15 percent of the total head loss in the network occurs delivering the liquid to each hole [10].

Additional guidelines to those presented in Table 3.3 are (1) trenches must be laid level, (2) 4 feet of soil must underlie the trenches, and (3) horizontal setbacks from various structures are necessary [11]. Winneberger argues that many of these guidelines are not needed [15, pp. 117-122]. The USPHS set maximum lateral lengths at 100 feet because of fear that breakage of brittle clay tiles would disable a field, especially in serial trench distribution. Today, stronger perforated pipe is available that can resist breakage from heavy machinery so that laterals can be longer than 100 feet. At a rest area this would allow use of a long trench drainfield along the highway right of way.

Spacing is needed between laterals mainly because of construction limitations. The USPHS set a standard of 6 feet between lateral pipes but did not explain why this figure is appropriate [11, p. 18]. If closer spacing of laterals is allowed more infiltration surface area is available per unit area of land. Thus, if legally possible, laterals should be spaced according to construction limitations only.

The USPHS manual gives no reason for mandating a 4-foot soil depth below the drainfield. Most readers have inferred this guideline to be a prevention measure against groundwater contamination, but it may have been done for administrative convenience [15]. Geologic and soil characteristics have a great influence on effluent cleansing and in some cases aquifers themselves can be used to clean wastewater with no need for 4 feet of soil below the trenches [15, p. 119].

Level trenches are mandated in the USPHS manual but level trenches are hard to construct and do not necessarily ensure uniform distribution. In addition, trenches need not be straight lines but can be built around obstructions [15, p. 121]. Suggested horizontal setbacks of trenches may be useless if an impermeable stratum in the field conducts the waste flow on its top surface with subsequent outlet to the atmosphere (such as in a roadcut section).

3.4.2.4 Texas Septic Tank/Drainfield Construction Guidelines. Construction guidelines for septic tank/drainfield systems can be found in the Texas Department of Health (DOH) manual entitled "Construction Standards for Private Sewage Facilities" [12]. The manual was written in 1977 and some of the guidelines are listed in Table 3.3. These guidelines are the same as those that would be applied for a rest area system. Many of the Texas guidelines are identical to the 1967 USPHS manual. Sewage application rates for different soils and percolation rates are shown in Table 3.6 and are approximately half of those rates allowed by the 1967 USPHS guidelines. Trench construction details and trench configuration on sloped terrain recommended by each manual are illustrated in Fig. 3.2 through Fig. 3.4. The Texas DOH guidelines for trench fill material add a provi-

TABLE 3.6. ABSORPTION TRENCH SIZING — SINGLE FAMILY RESIDENTIAL DWELLINGS [12]

Average Percolation (minutes/in.)	Sewage Application Rate (gal/sq ft/d)	Type of Soil	Minimum Trench Bottom Area for a One or Two-Bedroom House (sq ft/bedroom)	Minimum Trench Bottom Area for Each Additional Bedroom (sq ft/bedroom) *
< 1	Too Great for Consideration	Gravel	See Evapotranspiration Process, Paragraph B-4.1.4	
1-5	2.0	Sand	250	125
6-15	1.3	Sandy Loam	380	200
16-30	1.0	Sandy Clay	500	250
31-45	0.8	Silty Clay	625	300
46-60	0.6	Clay Loam	800	400
> 60	< 0.6	Clay	Absorption Systems Are Not Recommended	

* Minimum trench bottom area is calculated to include capacity for washing machine wastewater, organic material from garbage grinders, and infiltration from average rainfall. Recommended spacing between parallel trenches is 5 ft. Under no circumstances shall this distance be reduced to less than 4 ft. When dwellings consist of a large living area relative to the number of designated bedrooms, the following guidelines should be used to approximate the trench area:

Less than 1,500 sq ft - Use Trench Area for Two-Bedroom House
1,500 sq ft to 1,900 sq ft - Use Trench Area for Three-Bedroom House
For Each Additional 400 sq ft - Add Trench Area Equal to One Bedroom

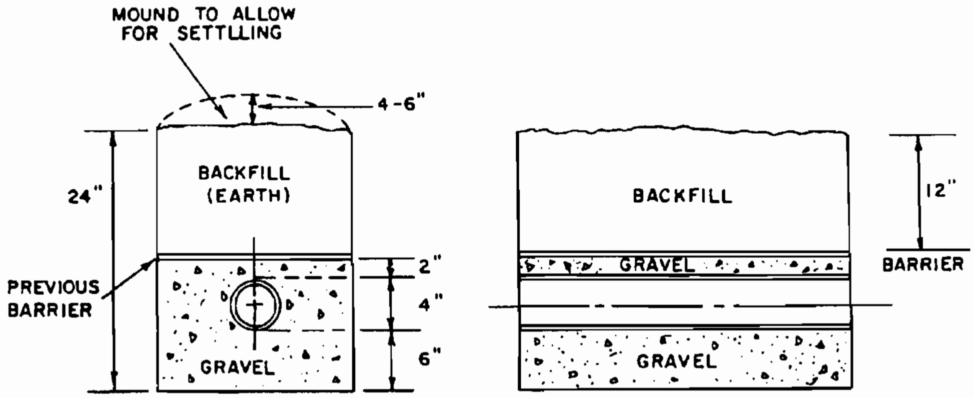


Fig 3.2. Trench construction details (U. S. Public Health Service, 1967).

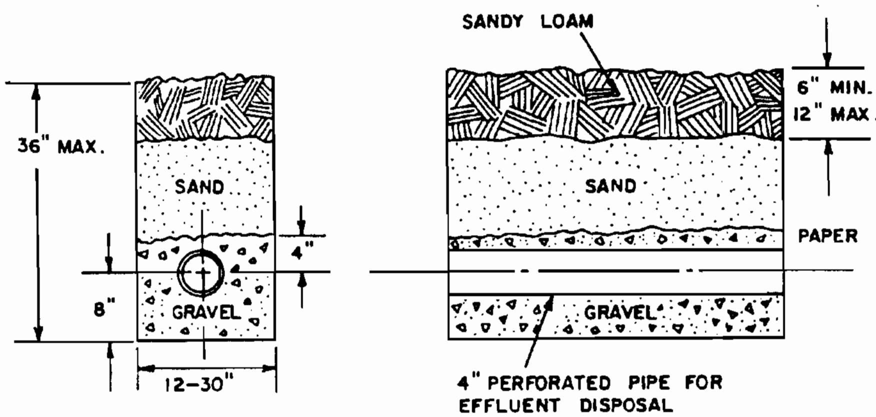


Fig 3.3. Trench construction details [12].

sion that sand is to be used in trenches if their depth is over 24 inches. The Texas guidelines also recommend narrow trenches, to increase the sidewall to bottomwall infiltration surface area ratio.

The Texas guidelines do differ from the USPHS manual in one major aspect, which is illustrated in Fig. 3.5. The flowsheet delineates the use of absorption beds or evapotranspiration beds for percolation test results of 30 min/in. or greater. Systems presented in the Texas guidelines are illustrated in Figs. 3.6 and 3.7. The Texas manual also suggests use of septic tank/drainfield systems only if average flowrates are less than 5000 gal/day. This limiting value is the rate one can expect at a rest area based on reported data.

The 1977 Texas DOH guidelines do not recommend or even mention time loading of drainfields or use of pressure distribution systems. The guidelines also use many of the guidelines mandated by the USPHS which have been subsequently questioned by others. The Texas guidelines also do not discuss the role of geologic or soil characteristics in designing a septic system. The DOH is in the process of developing new guidelines.

3.4.3 Operation & Maintenance of System

The major advantage of septic tank/drainfield systems is low operation and maintenance requirements. Septic tank sludge and scum levels should be checked every six months. Devices for such measurements are illustrated in Appendix E. In general septic tanks require pumping out every 2 to 5 years.

If suspended solids are viewed as a problem, flocculants can be added to the tank, but the practicality of this practice was not investigated in this report. Winneberger estimates the 1976 cost of flocculants used for a 1200-gallon tank to be 8 to 16 cents per week [14]. The benefits gained from flocculation may not be worth the maintenance effort necessary to add the flocculants. Reduction of suspended solids in the tank effluent does not necessarily reduce biological clogging of the field.

Costs of pumping are dependent on location and are not discussed. The man-hours necessary for maintenance (measuring sludge and scum) are about one man-hr/year. High maintenance costs occur only when drainfields clog. Clogged drainfields can be temporarily rehabilitated by use of hydrogen peroxide, as described by Hughes [13, pp. 7-8]. This approach is only temporary and the reason for clogging must be determined for proper operation in the future.

3.4.4 Summary

Septic tank/drainfield systems have been used commonly at rest areas, and failures have occurred in many instances. Failures of these systems are usually caused by tank undersizing and/or drainfield clogging. Design of these systems has usually been based on average daily flows and percolation rates.

The USPHS *Manual of Septic Tank Practices* was published in 1967 and is still used today with slight modifications. Many of the guidelines have not been borne out by experience. Texas guidelines are very similar to the USPHS manual except for inclusion of alternate systems for percolation rates greater than 30min/in. and the limit of daily waste flows to 5000 gallons for use of a septic tank-drainfield system. Maintenance and operational costs for septic tank systems are small and can be considered negligible.

3.5 FACULTATIVE POND SYSTEMS

3.5.1 Description of System

Ponds treating wastewater can be aerobic, facultative, or anaerobic. Aerobic ponds, aerated with mechanical aeration, are expensive and require constant supervision while anaerobic ponds can be undesirable because of odor problems caused by high waste loadings. Facultative ponds are cost effective in treating wastewater at rest areas and, if operated properly, do not produce odors.

Three zones of activities may be identified in facultative ponds. The bottom layer usually is void of oxygen and anaerobic decomposition of settled solids occurs. The top layer is aerobic in daylight hours since algae produce dissolved oxygen, which is available for aerobic decomposition of organic constituents of wastewater. In return, the bacteria supply carbon dioxide and mineralized decomposition products that the algae use for growth processes. Due to this symbiotic relationship of algae and bacteria much of the waste matter that enters the pond is converted into algal as well as bacterial biomass. The middle layer may be aerobic or anoxic, depending on the amount of sunlight and the organic loading. A typical facultative pond is illustrated in Fig. 3.8.

There are two types of facultative ponds: overflow ponds and evaporative ponds. Overflow ponds discharge an effluent while evaporative ponds do not. In evaporative ponds algae dies and settles to the pond bottom to be digested. This process builds up solids on the pond bottom, but the rate of deposition is so small that cleaning of ponds, if required at all, will not be necessary for many years (see Appendix F for calculation). Dissolved salts build-up also occurs in the evaporative pond; these dissolved salts change the algal composition of the pond and eventually will inhibit bacteria and algae [8, p. 46]. In practice, this process is not of great concern because the rate of increase of dissolved salts in most ponds is very low.

3.5.2 System Performance

Stabilization ponds have been used successfully in small communities for the last twenty-five years. Small communities have favored their use because of their low capital and operating costs. The main concerns in selecting ponds for wastewater treatment at rest areas are (1) land requirements, (2) odors, and (3) fluctuating concentrations of effluent parameters, such as BOD or TSS.

3.5.2.1 Overflow Ponds. Overflow facultative ponds must meet Texas state effluent standards of 30 mg/l of BOD and 90 mg/l of TSS (these are 30-day means), as shown in Table 3.2. The best way to meet these standards is to use overflow ponds in series. Pfeffer [19] studied a 3-cell series pond system used to treat rest area wastewaters at an Illinois rest area. Each pond was 3 feet deep and detention times were 47, 12, and 3 days, respectively, for the first, second, and third pond in the series. The system was designed to handle summer maximum flows and loadings.

Pfeffer [19] found that the pond system produced acceptable effluent and was capable of handling surge capacity two to three times the average daily flow until the surge period equaled one-fourth of the pond detention time. A week of peak loadings was necessary before a change of pond operation was observed. Recommendations for a rest area having an average daily traffic of 10,000 vehicles are presented in Tables 3.7-3.9. Pond criteria for one aerobic pond in the series, which can be used if land costs are high, are listed in Table 3.9. BOD values given are average effluent values and the day-to-day BOD can be greater, if algae escape in the effluent. Pfeffer considers this point unimportant because algae in the pond discharge are of the same species as those occurring in the streams.

Erickson [20] and Jenkins [21] have concluded from their studies that two ponds in series will not achieve the degree of treatment necessary to meet state effluent requirements. These studies evaluated spray irrigation, watervalets, and evapo-transpiration units as final effluent disposal

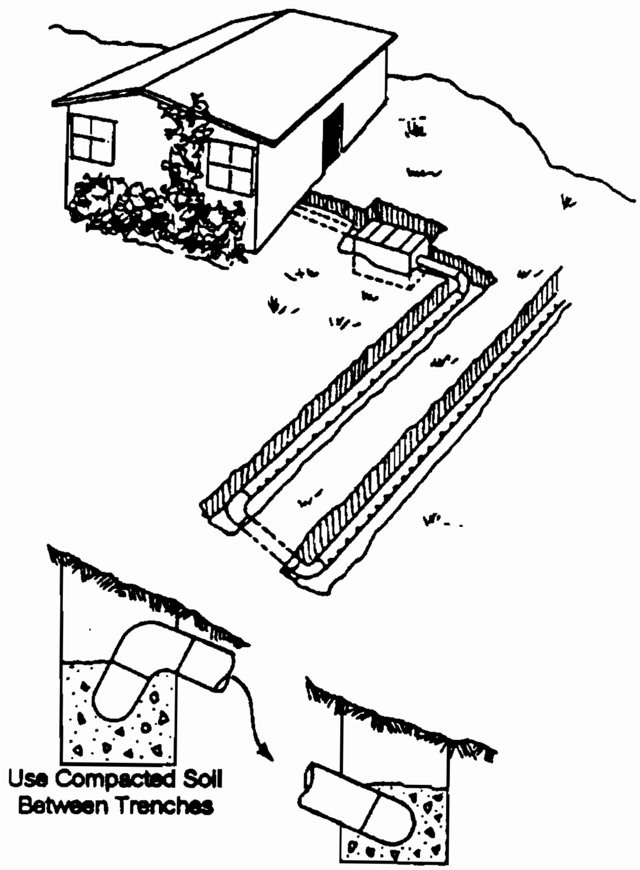


Fig 3.4. Septic tank system for sloping ground [12].

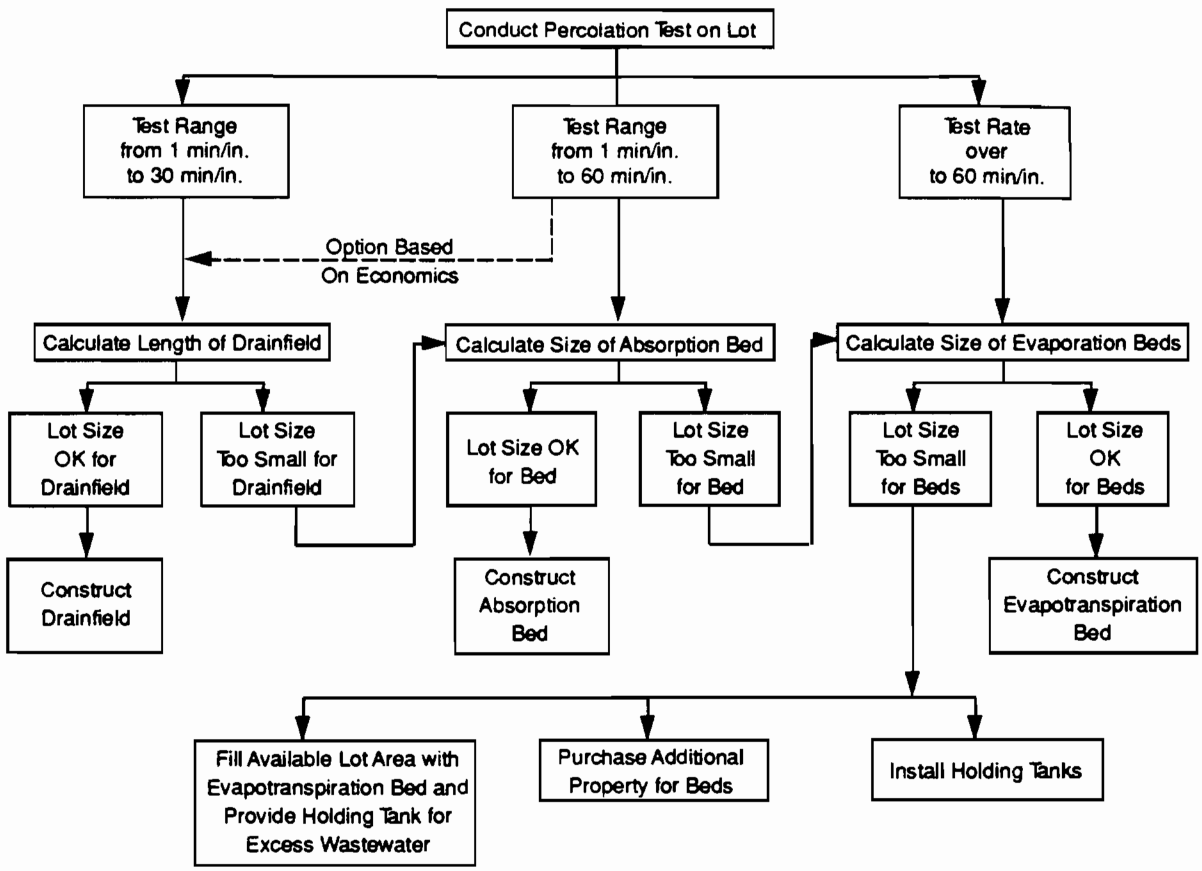


Fig 3.5. Suggested flow sheet for selecting proper subsurface disposal method.

methods. The two-pond systems cause the virtual elimination of fecal coliforms.

The state of Oregon has two rest areas utilizing 3-cell facultative ponds systems, which occupy a total of 3 acres. The pond effluent is sprayed over 3 acres after going through an 8-hour holding period after chlorination; spraying takes place in the summer only. The ponds are lined with 30-mil PVC, which is covered with 6 inches of sand and a top layer of rocks to hold the liners in place [22]. The Army Corps of Engineers has 163 ponds across the United States; they use a membrane type liner on a compacted layer of natural subsoil to prevent seepage [23, p. 29]. Design criteria for overflow ponds as found in Metcalf & Eddy [2] are given below:

Detention time	7-30 days
Depth	1-2 m (3 to 6 ft)
pH	6.5-9.0
Optimal temp.	20 C
BOD loadings	15-80 kg/hectare/day (13.4-71.4 lb/acre/day)
BOD conversion	80-95 %
Algal concentrations	20-80 mg/l
Effluent SS	40-100 mg/l

Solids separation will be necessary for overflow ponds if the systems cannot meet state standards. Rock filters have shown promising results in meeting effluent standards. The rock provides a surface on which biological slime can grow and effect additional BOD and SS removal as the pond water flows through the rock. A typical rock filter is shown in Fig. 3.9. Preliminary results with rock filters show reduction of BOD and SS to 30 mg/l in the final effluent [2, p. 563]. Rock filters can be used if the alkalinity of the ponds is over 200 mg/l as $CaCO_3$; otherwise odors could result. The alkalinity level of the pond depends on the characteristics of the water supplied to the rest area and on the balance between algal oxygen production and nitrification of ammonium (acid production) in the pond. If algal respiration (CO_2 consumption) in the pond is high the pH of the pond water can rise to the point where precipitation of calcium occurs; this lowers the alkalinity.

3.5.2.2 ***Evaporative Ponds.*** Evaporative ponds function as facultative ponds do except that there is no effluent; all losses are through evaporation. Evaporative ponds are currently used in Washington, Oregon, and California. Oregon has one evaporative pond at a rest area in an area where rainfall is below 10 inches a year. Washington has eight evaporative ponds operating at present in the eastern part of the state. California also has several evaporative ponds in the eastern part of the state [22]. All of these sites have annual evaporation rates that exceed annual rainfall rates. Rest areas in west Texas are prime candidates for evaporative ponds while some eastern Texas rest areas could use ponds in the summer.

Sylvester & Seabloom discussed the use of evaporation ponds in eastern Washington [8, pp. 47-49]. Using pan evaporation data from the U.S. Weather Bureau and pan coefficients (actual evaporation/pan evaporation) of 0.7 to 0.8 they found the yearly evaporation rate exceeded the yearly rainfall rate by 24 in. for eastern Washington. The researchers recommended use of this excess, the yearly average evaporation rate minus the yearly average rainfall rate, in sizing pond surface areas (24 in./yr = 87,120 sq ft/acre/yr) in relation to inflow. An example calculation is shown in Appendix F. For a rest area in Maytown, Washington, three acres of land were required, based on the evaporation excess amount above rainfall of 24 in. and the flows experienced at the rest area. Liners were not suggested since it was felt that the ponds will seal themselves with time and seepage will not be a problem.

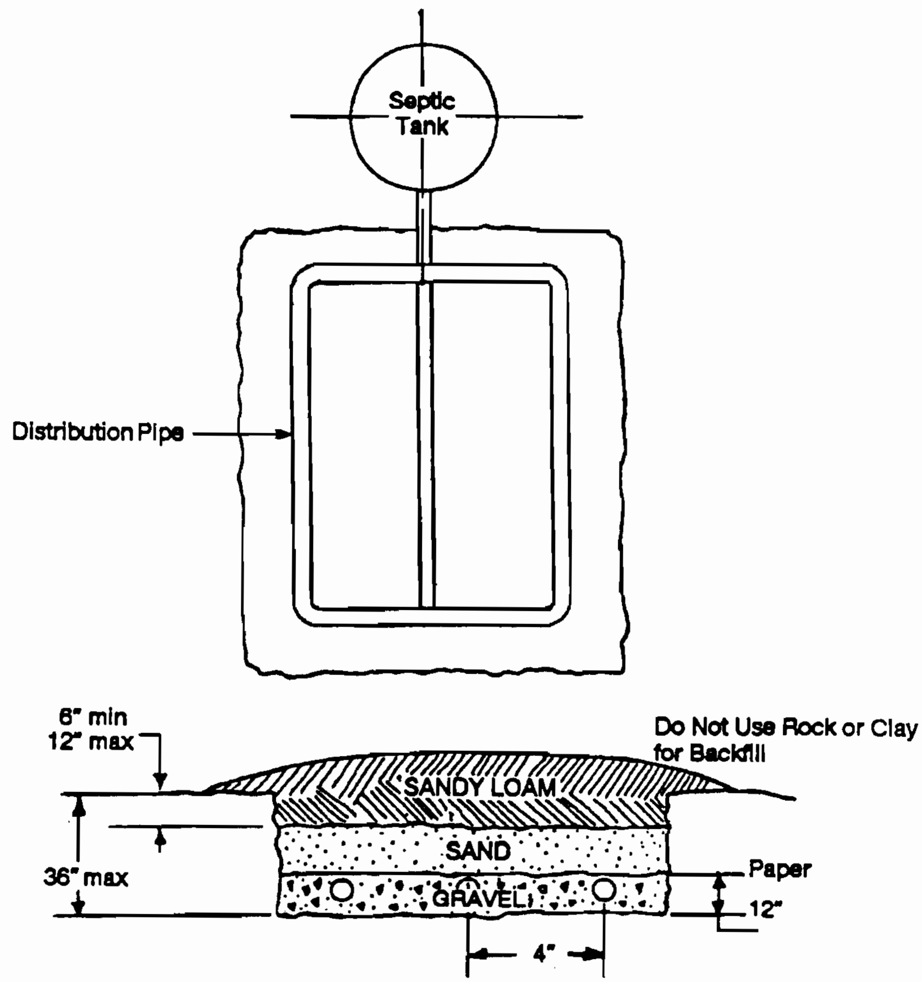


Fig 3.6. Absorption bed system [12].

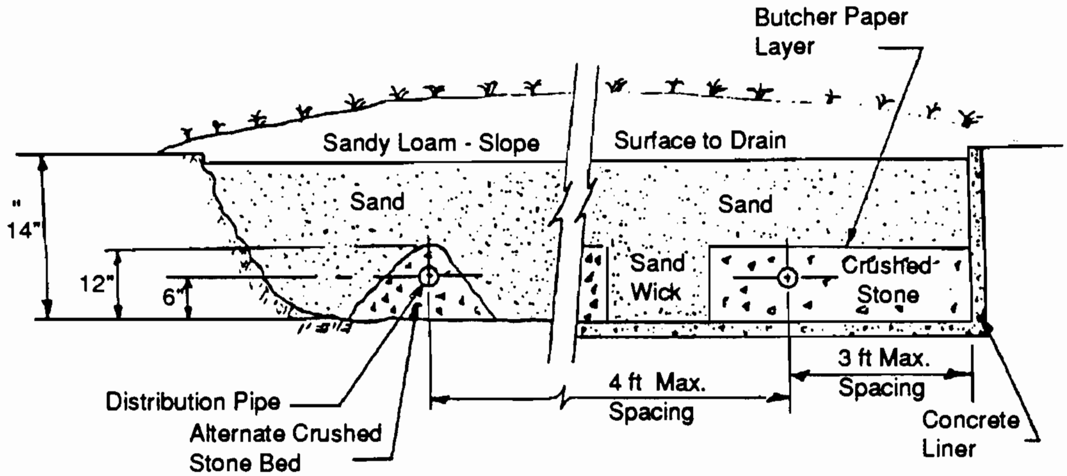
Sylvester & Seabloom recommended use of four ponds in-series. In this pond system two ponds are built first and the others are built later as needed. When the first pond reaches 5 feet in depth it is drawndown to 3 feet by discharge to the second pond in the series. When the second pond reached a depth of 3 feet, both ponds are allowed to fell to 4 feet [8, pp. 49-50]. If this scheme is followed it is estimated that it will take 0.8 years (~9.5 months) to fill the first pond and that both ponds will be filled to a depth of 4 feet in 1.5 years. Estimated BOD loadings to ponds studied by Sylvester & Seabloom were 9.85 lb/acre/day annually and 18 lb/acre/day for the maximum month (August).

Spray irrigation of pond water could be used to reduce the pond area. In practice, spray irrigation is not used in Washington because of land requirements and strict state regulations [22]. Sylvester & Seabloom suggest that the effluent from the second pond is suitable for use as toilet flushing water, if water is in short supply at the rest area [8].

3.5.3 Operation and Maintenance of System

The largest costs associated with pond systems are initial construction costs and land costs. The operating and maintenance requirements of the system include mowing the grass by pond edges, maintaining pond dikes, preventing pond bottom weed growth, inspecting inlet and outlet devices for clogging, and minimizing the formation of algal mats on the pond surface. Hughes estimates that two man-hours per week would be necessary for pond maintenance [13]. Pfeffer estimates from U. S. Environmental Protection Agency data that pond installation costs are about half those of package treatment plants, based on 1973 cost figures [19].

In order to prevent bottom weed growth, pond depths should be greater than 2 feet at all times [8]. Pond dike maintenance is synonymous with keeping muskrats and burrowing rodents away from the pond [19]. Pfeffer recommends weekly water sampling of overflow pond effluent, with the samples analyzed at a centralized state lab for all rest areas [19].



Notes: 1. Where a liner is used rock over or other material that may damage liner, the liner shall be laid on a 4" protective sand cushion, and covered by similar cushion.
2. The crushed stone or gravel bed should be made of 1-1/2"- 2-1/2" size hard stone.
3. Sand columns, formed by a permeable material, should extend completely through the crushed stone or gravel bed. Total column area should be 10 to 15% of the bed area.
4. The surface should be mounded or sloped to drain storm water.

Fig 3.7. Typical evapotranspiration bed cross section [12].

Use of aerobic ponds, as suggested by Pfeffer, will increase costs and maintenance requirements for the pond system. The need for more expertise in overseeing the system may necessitate the hiring of additional personnel or additional training of existing personnel. If this is the case, the septic tank/drainfield option may be more cost effective.

3.5.4 Summary

Overflow and evaporative ponds are attractive treatment systems for rest areas. Evaporative ponds are favored because they do not produce an effluent. Areas in west Texas are more suitable for evaporation ponds whereas rest areas in wetter east Texas may be able to use overflow ponds if effluent can be disposed of in a satisfactory manner. Pond surface area calculations can be based on evaporation rate excesses over rainfall rates for evaporative ponds and by surface loading rates for overflow ponds. Ponds are flexible systems; if land is available the system can be expanded by building additional ponds. About three acres of land are required for pond treatment systems at rest areas.

Pond systems have low operation and maintenance requirements, consisting of mowing, dike and inlet and outlet inspection, and weed control. It has been estimated

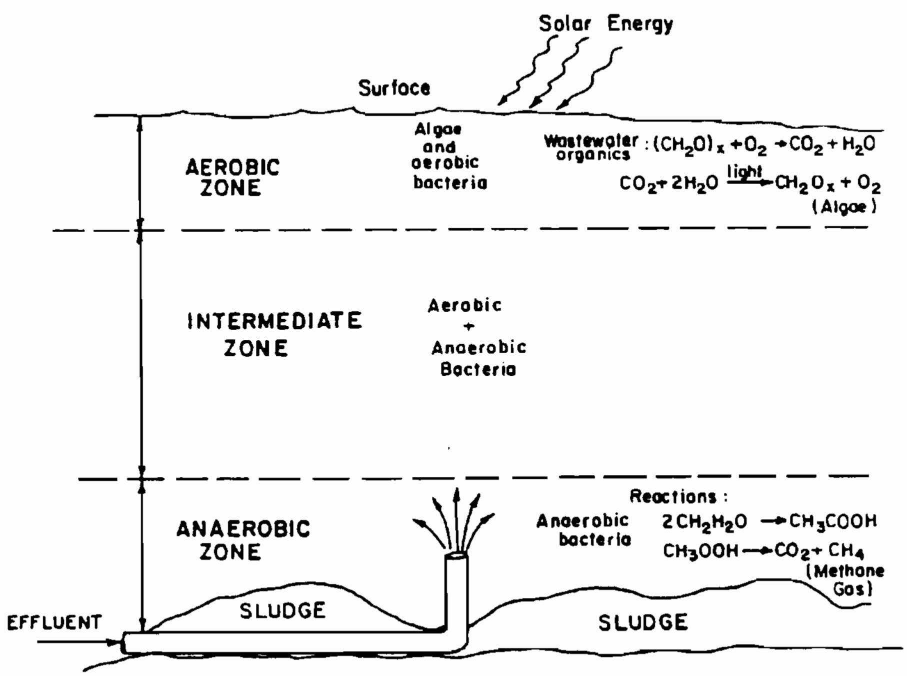


Fig 3.8. Facultative pond [2].

that two man-hours per week would be required to maintain the system.

3.6 EXTENDED AERATION PACKAGE PLANTS (EAPPS)

3.6.1 Description of System

Extended aeration package plants (EAPPs) consist of an aeration tank(s) followed by a sedimentation chamber (clarifier). Bacteria in the aeration tanks consume dissolved organic matter in the wastewater and the biomass is separated from the effluent in the clarifier. All or a portion of the settled sludge is then returned to the aeration tank via a recycle pipe line. A flowsheet, plan view, and cross-sectional view of a typical EAPP are shown in Fig. 3.10.

EAPPs have lower food/biomass ratios, longer hydraulic detention times, longer cell residence times, and larger recycle ratios than their larger municipal treatment plant counterparts. These characteristics (1) enable EAPPs to handle shock hydraulic and organic loadings better than conventional plants and (2) minimize sludge production [23]. An additional bonus in using EAPPs is that nitrification can occur in these systems when they are operated at long cell residence and hydraulic detention times.

Oxidation ditches are similar to EAPPs. Oxidation ditches consist of a "raceway" loop tank which has rotors to provide circulation and atmospheric aeration of the effluent. The carousel method is the same except that vertically mounted mechanical aerators provide both oxygen to the wastewater and sufficient horizontal velocities to prevent settling. These systems are operated on intermittent cycles consisting of (1) closing the inlet valve and aerating the wastewater, (2) stopping the rotor and allowing settling, and (3) opening the inlet and outlet valves, allowing incoming wastewater to displace an equal volume of clarified effluent [2]. Goronszy [24] proposes using an intermittent treatment system consisting of a rectangular tank with surface aerators and an effluent weir that can be adjusted vertically. A typical oxidation ditch and Goronszy's intermittent cycle scheme are illustrated in Fig. 3.11.

3.6.2 System Performance

EAPPs are used frequently at rest areas for waste treatment. Hughes found in a 1977 national survey that only septic tank/drainfield systems outnumber package plant systems at highway rest areas [13]. Approximately twenty package plants are operated at rest areas in Texas. The state of Louisiana uses package plants for virtually all of highway rest areas [22]. Hughes reported 97.5 and 92.3 percent

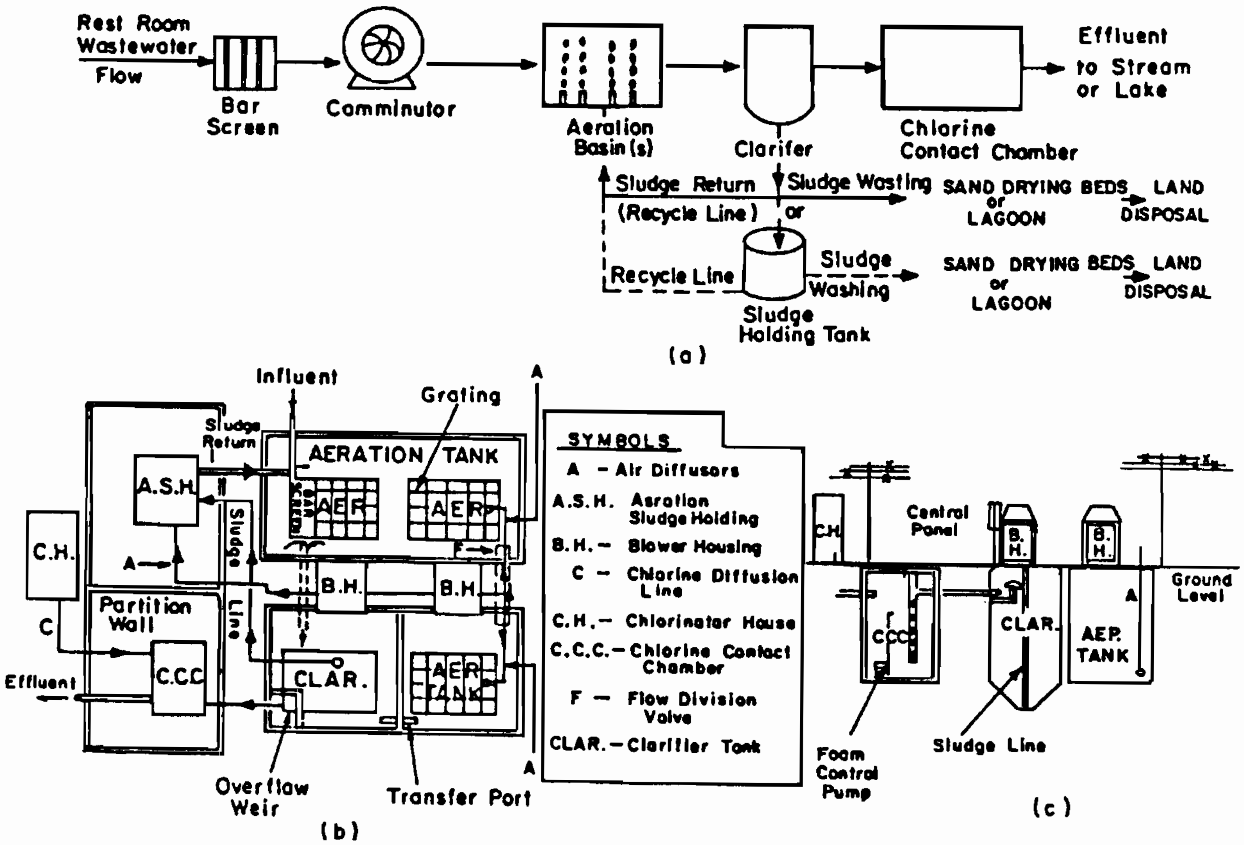


Fig 3.10. Extended aeration package plant — (a) Flowsheet; (b) Plan view; and (c) Cross-sectional view. Source parts (b) and (c): Process Equipment Company, P. O. Box 9549, Corpus Christi, Texas.

reductions of BOD and suspended solids for an EAPP at a rest area in Mississippi. All effluent samples met state discharge standards [13]. In general, package plants are able to exceed discharge effluent standards if they are run as designed. Package plants fail to meet effluent standards when the clarifier performance is reduced and solids flow into the effluent. The clarifier performancer is dependent on (1) hydraulic flowrates and (2) the ability of solids to settle. At rest areas, highly variable flow rates and high nitrification rates can cause inefficient operations or system failure.

3.6.2.1 ***Hydraulic Overdesign.*** EAPPs at rest areas usually are designed based on peak daily flowrates to avoid solids overflow into the effluent during peak flows. During sustained low flow periods, such as wintertime flows, the biomass in the plant may starve, thus causing poor performance. Even if the system operates properly its full capacity is not utilized.

Pfeffer [19] states that the design of EAPPs based on future loads and summertime peak flows will result in an overdesign for winter conditions by about a factor of four. Hughes estimated that the rest area he studied in Mississippi was hydraulically overdesigned by a factor of 3 to 5 [13]. Palaez reported that two extended aeration treatment plants operated by the Texas State Department of Parks & Wildlife were hydraulically and organically underloaded (having normal operating ranges of 25 to 34 and 36 to 47 percent of design capacity, respectively) [23].

Hughes suggests using modular package plants to solve hydraulic/organic underloading problems experienced at rest areas. Modular plants consist of aeration tank modules which are added to the treatment system as they are needed. As an example consider a modular plant which can be expanded to include three aeration tanks. At the beginning of a rest area operation, one tank will be used and the performance will be evaluated within the first five years. Based upon this evaluation a decision is made as to whether a second tank should be installed. If a second tank is added, the system is reevaluated within another five years to determine if a third tank is necessary. In this stepwise fashion the package plant is always running at close to design flow. Hughes recommends that evaluation periods be made five years as a standard [13].

If EAPPs are designed based on average daily flowrates at the rest area, hydraulic/organic overloadings could occur. Pfeffer tested organic overloading by loading a package plant at 40 lb BOD per 1000 cubic feet of the aeration tank volume and achieved 90 percent BOD removal but only 80 percent removal of suspended solids [25]. Thus, overloading could be a problem, if plants are designed on average daily flows at rest areas. Modular plants may be helpful here if additional aeration tanks can be put on line for that part of the year in which peak daily flowrates are expected.

3.6.2.2 ***Rising, Bulking, and Non-flocculant Sludge.*** Long cell residence times, organic underloadings, and high nitrogen levels can lead to rising sludge. EAPPs that are organically underloaded are likely to have nitrification occurring in the aeration tank. Nitrification in the aeration tank produces nitrates which can be converted to nitrogen gas by denitrifying bacteria in the clarifier. The denitrification process occurs in the settled sludge and the nitrogen gas produced buoys the sludge to the surface. This phenomenon is termed rising sludge. Because rest area wastes have high nitrogen concentrations and because package plants have long detention times and are often organically underloading, rising sludge is likely to occur if preventive measures are not taken. Rising sludge problems at rest areas can be remedied

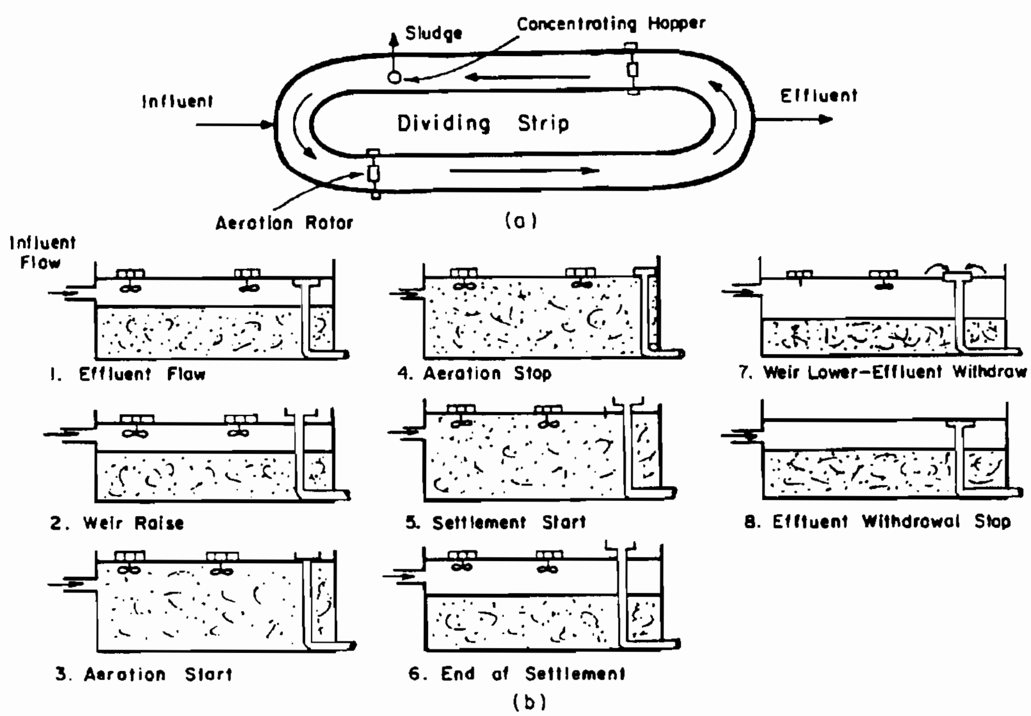


Fig 3.11. (a) Oxidation ditch and (b) Goronszy's intermittent cycling schematic [24].

by (1) increasing the sludge wasting rate or (2) increasing the sludge return rate.

Bulking sludge is a sludge that has poor settling and compactibility characteristics. Nitrification reactions consume dissolved oxygen so that, if nitrification is occurring in the aeration tank, the concentration of dissolved oxygen (DO) in the aeration tank can fall. If the oxygen level drops below 2 mg/l then growth of filamentous bacteria is favored. These types of bacteria have poor settling characteristics and thus are a cause of bulking sludge. Maintaining a DO level of 2 mg/l in the aeration tank can help alleviate sludge bulking problems, but the bulking process is complicated and is not totally understood at present.

If an EAPP is organically underloaded to such a low food/mass ratio that there is not a sufficient concentration of bacteria to flocculate in the clarifier then the sludge is non-flocculant and solids overflow will occur. Pfeffer [25] found that 9 lb BOD per 1000 cu ft of aeration tank volume was necessary to achieve a 95 percent removal of suspended solids in the clarifier. Hughes suggests that BOD loadings be in the range of 10 to 25 lb per cubic feet per day for proper EAPP operation [13]. Palaez found good sludge settling characteristics at the recreational park package plants he studied despite organic underloadings experienced at the plants [23]. He concluded that a high percentage of fixed solids (mineral solids) in the mixed liquor suspension accounted for the good settling characteristics of the sludge. Thus it appears that the mineral composition of the wastewaters may be an important factor in sludge settleability.

3.6.2.3 Intermittent Loading Performance. Intermittent loading systems have been explored by Maloch [26] and Goronszy [24]. Maloch subjected a bench and pilot scale intermittent system to severe underloadings during the five-day week and to peak loadings during the weekend. Periods of underloading varied in length while peak loadings lasted eight hours. Low flows (weekly flows) were set at 10 percent of design flow while peak flows were set at 300 percent of design flow. Maloch found that intermittent loadings imposed every two to three days caused biological system failure. However, intermittent loadings every six to seven days did not disrupt the system because of long cell residence times (which allowed a resting period for the organism). Although effluent quality was lower during shock loadings the system returned to normal operations rapidly after the shock loadings ceased.

Goronszy [24] reported that intermittent loading systems offer high flexibility because cyclical operation schedules can be changed if unexpected hydraulic conditions occur. Nitrification/denitrification cycling times can solve the problem of sludge bulking by reducing the production of nitrates through denitrification *before* the settling cycle. An example of a six-hour nitrification/denitrification schedule is given in Fig. 3.12.

3.6.3 Operation & Maintenance

Operation and maintenance requirements for EAPP systems are high compared to the other systems discussed. Strong [30] found labor costs to be 83 percent of the total operating costs for a package plant. Sylvester & Seabloom state that trained operators are necessary for package plants and so they favor pond systems [8]. Conversely, Hughes states that operators can be trained in the principles and mechanics of operation of EAPPs with a minimum of formalized training [13].

Operational requirements for EAPPs include (1) periodic surveillance of aeration tank for debris escaping the bar screen or comminuter, (2) maintenance and regulation of air blowers and diffusers to match loading conditions, (3) regulation of sludge wasting and recycling to maintain optimal sludge age, (4) sampling of aeration tank water for various parameters, (5) periodic checks for denitrification in the clarifier, (6) periodic restocking of chlorination supplies, (7) effluent sampling, and (8) sludge waste disposal.

Hughes suggests the following routine maintenance procedures for EAPPs: (1) daily removal of scum off clarifier water surface by airlift pump (operated by rest area attendant), (2) periodic (twice daily) scraping of the clarifier walls to remove solids (to prevent denitrification), (3) weekly wasting of a portion of the aeration liquor, (4) daily cleaning and inspection of scum airlift pump lines, and (5) periodic inspection of aeration tank diffusers. Hughes estimates daily cleaning and inspection would require one man-hour per day [13].

Goronszy estimates that intermittent systems will require five hours of semi-skilled labor per week. These

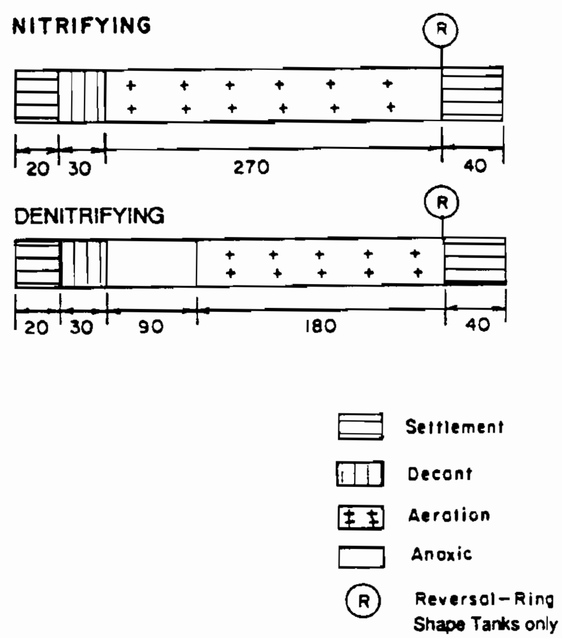


Fig 3.12. Goronszy 6-hour time base cycling for nitrification and denitrification [24].

systems require the following operational tasks: (1) setting of sludge pump cycles, based on indication of sludge growth from sludge settlement tests, (2) periodic suspended solids determinations (to check for sludge bulking), and (3) periodic comprehensive analysis to determine overall efficiency and the need for changing aeration times, based on residual ammonia levels [24].

3.6.4 Summary

At present, package plants are one of the most frequently used types of waste treatment system at rest areas. The major problem in designing the plants is correctly sizing the plant. Many EAPPs at rest areas are hydraulically overdesigned and organically underloaded. The performance of an EAPP is dependent on how well the sedimentation unit performs. Sedimentation problems include rising, bulking, and non-flocculant sludge settling. Intermittent cycle plants hold promise in dealing with the problems of hydraulic loading fluctuations and poor sludge settling. Design criteria for EAPPs are given in Table 3.10.

Package plants can be expected to have higher operation and maintenance requirements than pond or septic tank/drainfield systems. It is estimated that one man-hour per day will be needed for this system.

3.7 LAND TREATMENT SYSTEMS

3.7.1 Description of System

Land application systems take advantage of the ability of plants, soil surfaces, and the soil matrix to treat wastewaters. Mechanisms for treatment and utilization of wastewater applied to land systems are vegetative uptake, infiltration, evapotranspiration, microbial action, and chemical exchange. The three principal processes of land treatment are irrigation (slow infiltration), rapid infiltration, and overland flow. These processes are shown schematically in Fig. 3.13. Table 3.11 compares characteristics of the three systems.

3.7.1.1 ***Irrigation.*** In the irrigation method, effluent is applied to a crop cover and treatment is accomplished primarily through plant nutrient uptake and evapotranspiration. Effluent is usually applied by periodic sprinkling or surface spreading techniques, with the former being more common. Sprinkling systems can be fixed or mobile; surface spreading techniques include flooding, and ridge and furrow application. Irrigation is considered a low infiltration process, with wastewater application rates of 1 to 3 inches per week used if crop production is emphasized or 2.4 to 4 inches per week for maximizing hydraulic loadings [2, p. 765].

3.7.1.2 ***Rapid Infiltration.*** In the rapid infiltration method effluent is applied to the treatment area at high rates (4 to 84 inches per week) and is treated mainly through percolation and chemical exchange. Highly permeable soils are necessary for the method and groundwater quality is sure to be affected. Rapid infiltration systems are used for groundwater recharge as well as for wastewater treatment.

3.7.1.3 ***Overland Flow.*** In overland flow systems, effluent is applied at a high elevation point in a field and the sheet flows over the surface to a collection point. Treatment is accomplished primarily by microbial action and vegetative uptake, with some evapotranspiration and chemical change. In typical systems about 40 to 80 percent of the effluent applied runs off and the remainder is lost through evapotranspiration. Application rates are usually 6 to 16 inches per week [2, pp. 766, 808].

3.7.2 System Performance

The ability of land treatment systems to treat wastewater depends on many factors, including soil structure response to applied effluents, nitrate removal mechanisms in the system, and hydraulic and organic loadings. Soil structure can be affected by cation exchange, especially the exchange of sodium for calcium and magnesium. This exchange process can cause soil particles to disperse, thus reducing soil permeability [2, p. 768]. Nitrate removal mechanisms include plant uptake and denitrification; these mechanisms are prominent in irrigation and overland flow systems. The degree of hydraulic and organic loadings applied to a land treatment system will determine if anaerobic or aerobic conditions will prevail. In most cases effluent

TABLE 3.10. EXTENDED AERATION DESIGN CRITERIA [13]

Aeration Tank	
Detention time, hours	18-36
Sludge age, days	20-30
Food-to-microorganism ratio, lb BOD/lb MLVSS-day	0.05-0.15
BOD loading, lb BOD/1000 cu. ft. aeration tank	10-25
MLSS, mg/l	3000-6000
Sludge return rate, % of influent flow	50-300
Air required, lb O/lb BOD-day	> 1.5
MLVSS, mg/l	2100-4200
Recycle flow/average flow	0.50-2.0
q	1.0-1.3
Clarifier	
Overflow rate, gpd-sq. ft.	100-300
Detention time, hrs.	4
Solids loading, lb/sq. ft - hr	0.5-1.24
Weir loading, gpd-ft	10,000

is applied to the land intermittently to prevent anerobic conditions, which cause odors. Estimated removal efficiencies for several parameters in land treatment systems are given in Table 3.12.

At rest areas the majority of land treatment systems used are spray irrigation, with a small number of overland flow systems employed. Rapid infiltration is not used at rest areas, presumably because of fear of groundwater contamination. Thus, the only systems that are reviewed in this report are spray irrigation and overland flow systems.

3.7.2.1 ***Spray Irrigation Performance.*** Spray irrigation performance is governed by either hydraulic or nitrogen loadings. Hydraulic loadings are determined through a water balance where rainfall and applied wastewater are inputs and percolation and evapotranspiration are outputs. Nitrogen loadings are determined from wastewater samples and flowrates at the rest area. Hydraulic underdesign will result in ponding of effluent while nitrogen overloading (beyond that which the plants can uptake) results in the production of nitrates, which can be dangerous.

3.7.2.1.1 Lagoon-Spray Irrigation System. Jenkins [21] monitored the performance of a lagoon-spray irrigation system (Fig. 3.14) at an Alabama rest area. In this system, rest area wastewater enters the lagoons, goes through pond treatment processes in each pond, and then is pumped through a 3/4-inch hose to a 180 degree sprinkler head for spray application. Two automatic electronic timers control the spraying schedule. One timer is a 24-hour timer which sets two one-hour spray periods per day on the second timer. The second timer is an hourly timer and it activates the pump to operate over five-minute intervals over the hour period (i.e.,

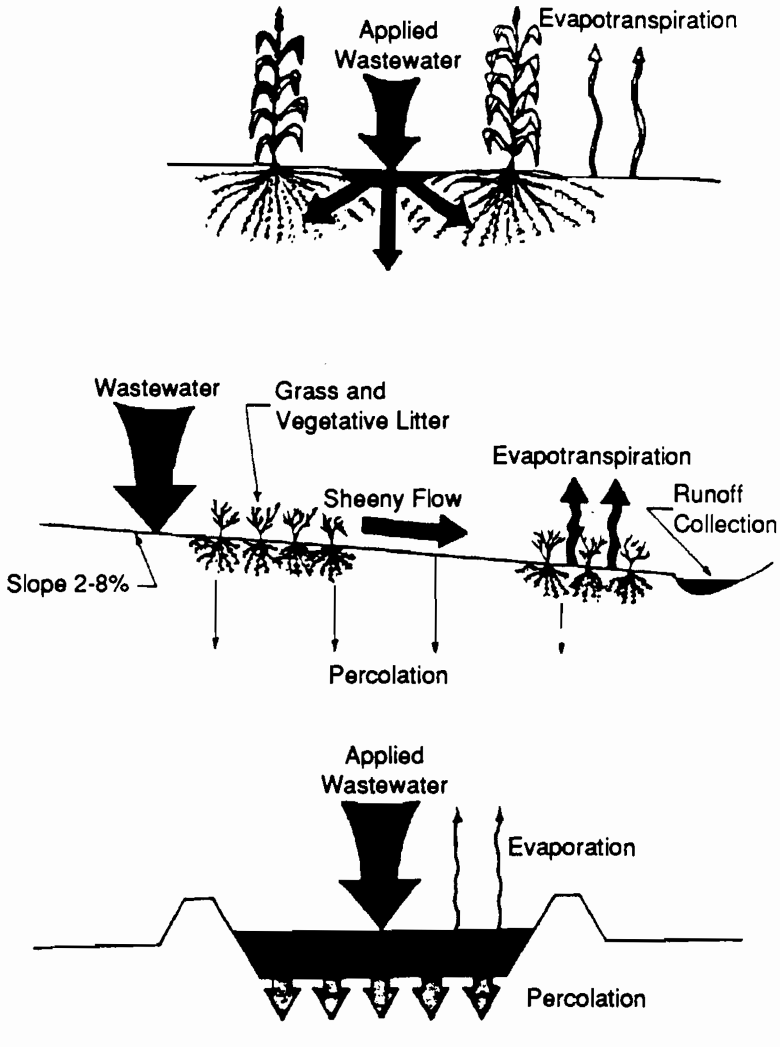


Fig 3.13. Land application methods [42].

TABLE 3.11. COMPARISON OF LAND APPLICATION APPROACHES [13]

Factor	Spray Irrigation	Overland Flow	Rapid Infiltration
Liquid-loading rate	0-5-4 in./wk	2-5.5 in./wk	0.3-1.0 ft/wk
Annual application	2-8 ft/yr	8-24 ft/yr	18-500 ft/yr
Application techniques	Spray or surface	Usually spray	Usually surface
Soils	Moderately permeable with good productivity when irrigated	Slowly permeable soils such as clay loams and clay	Rapidly permeable soils such as sands, loamy sands, and sandy loams
Probability of influencing groundwater	Moderate	Slight	Certain
Needed depth to groundwater	About 5 ft	Underdetermined	About 15 ft
Wastewater losses	Predominantly evaporation or or deep percolation	Predominantly surface discharge but some evaporation and percolation	Percolation to groundwater

five minutes ON, five minutes OFF). A schematic of the timer system is shown in Fig. 3.15.

The spray site was a flat grassy area with sandy loam soil and the depth below grade to groundwater was 2 feet in the winter and 5 feet in the summer. The site was chosen because of land availability and easy accessibility. In the spray and control areas lobby pines and silverberries were planted as crop covers. Shallow (3-ft, 5-in.) and deep (10-ft) wells were dug in the control and spray areas. Six vacuum type porous ceramic cup type soil water samplers were used (two in the control area, two in the spray areas, and two at an appreciable distance from the system).

The system was operated from April to October in 1976. Water samples were collected weekly and soil samples collected periodically (to look for changes in soil structure). Evapotranspiration rates were estimated using U. S. Weather Bureau pan data. The spray application rate was one inch per week.

Results of the monitoring showed that the concentrations of COD, SS, and nitrates in well samples were influenced more by changes in lagoon water quality than by the irrigation system. Thus, pond seepage was a major problem in the system. The results indicate a runoff rate of 0.07 inch per spray period over 0.03 acre, an evaporation rate of 0.7 inch per week, and a four-fold increase in plant growth in the spray area versus the control area. The hydraulic loading determined the land area required for spraying and no soil clogging or groundwater contamination occurred.

Jenkins proposes use of a lagoon-spray irrigation system with alternating spray (grassy) and buffer strip (woodland) areas, as shown in Fig. 3.16. The grassy areas facilitate evapotranspiration while the wooded areas act as aerosol buffers. For a 20,000-gallon-per-day wastewater flowrate and an application rate of one inch per week, Jenkins suggests using 5.16 acres of spray area with an additional 3 to 5 acres of buffer area [21]. Jenkins does not recommend use of spray irrigation for septic tank effluent because of odor problems.

3.7.2.1.2 Barriered Landscape Water Renovation System (BLWRS). Erickson et al [20] developed the BLWRS shown in Fig. 3.17 to polish lagoon effluent and to recharge a shallow aquifer. In this system, pond effluent is sprayed onto the treatment area and travels through the aerobic soil and produces a mounded water table. The aerobic soils decompose organic nitrogen into nitrates and absorb phosphorus. The water flows from the mound to the anaerobic barrier trenches (made of peat and corn), where denitrification takes place.

TABLE 3.12. ANTICIPATED REMOVAL EFFICIENCIES FOR WELL-DESIGNED AND PROPERLY OPERATED TREATMENT SYSTEMS, % [13]

	Application Method		
Constituent	Spray Irrigation	Overland Flow	Rapid Infiltration
BOD	98+	92+	85-90
COD	95+	80+	50
Suspended Solids	98+	92+	98+
Nitrogen (Total as N)	85+	70-90	0-50
Phosphorus (Total as P)	80-99	40-80	60-95
Metals	95+	50+	50-95
Microorganisms	98+	98+	98+

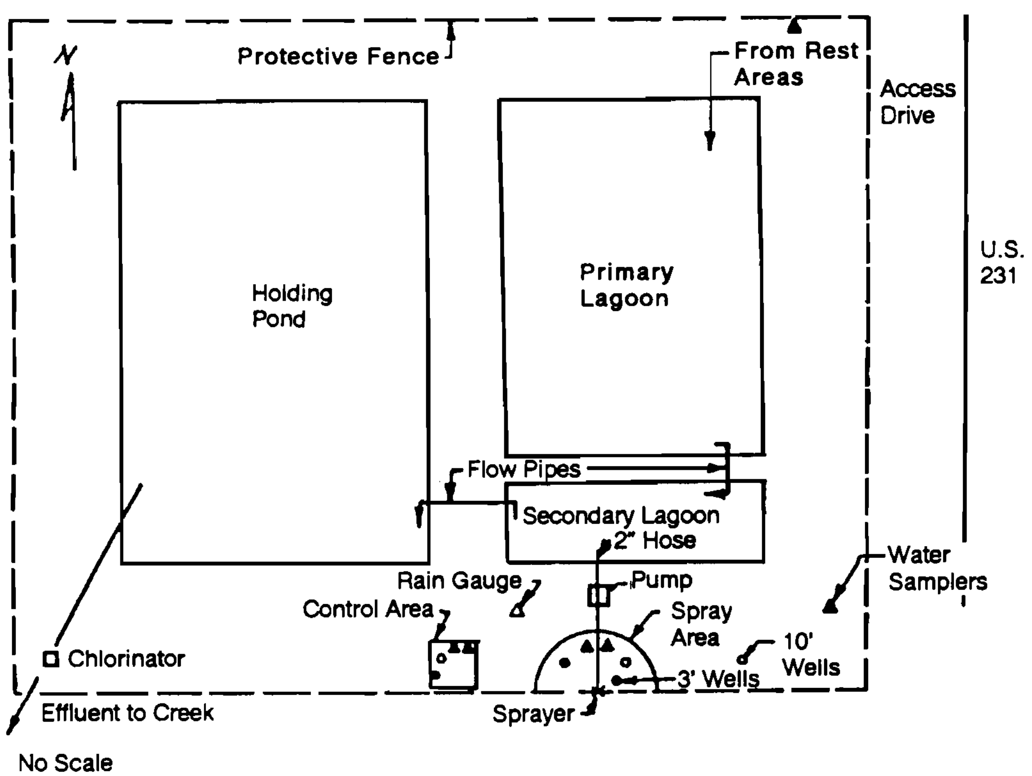


Fig 3.14. Jenkins' spray irrigation system [21].

The study site was in Lansing, Michigan, and the system was operated for eight weeks, from June 15, 1979, to August 10, 1979. A 12,000-gallon holding tank followed the lagoons and effluent was pumped from this tank (after ozonation) and sprayed over .67 acre at an application rate of 2.4 inches per week. Spray periods lasted for 6 to 8 hours, with a fourteen-hour rest period between spraying. Wells were constructed at 6-and 18-inch depths inside the spray area, inside and outside of the barriers, and in groundwater areas outside the treatment system. Well samples were tested for total Kjeldahl nitrogen, nitrates, ammonia, phosphorus, BOD, total organic carbon, and fecal coliforms.

Results of the monitoring showed that the system performed well. Nitrate concentrations in groundwater ranged from 1 to 7 ppm in deep wells (18 inches) to 2 to 8 ppm in shallow wells; the total nitrogen removal was 92 percent. Removal of total phosphorus was ~97 percent, and 67 percent of the BOD and TOC applied was removed. Ozonation of the effluent in the holding tank had negligible effects on the reducing of fecal coliforms.

Nitrification in the upper 6 inches of the soil reduced ammonia levels and increased nitrate levels. Denitrifcation processes reduced the nitrates by 50 percent over the one-half-foot stretch between the depths of 6 and 12 inches. The barrier trenches also reduced nitrates, with the deeper wells showing a much greater reduction of nitrates across the barrier than the shallow wells (6 to 17 ppm reductions in deep wells vs. 1 ppm in shallow wells). Nitrate reductions were attributed to plant uptake or denitrification processes (in the rhizosphere, anaerobic zones, and trench areas).

3.7.2.1.3 Spray Irrigation Systems at Texas Rest Area. At present there are ten rest areas on Interstate and state highways in Texas using spray irrigation as a final effluent disposal method. Eight of these rest areas use highway right of way land for the spray area. At recreational parks operated by the Texas Department of Parks and Wildlife 73 percent of the wastewater treatment systems have spray irrigation. Land areas used for irrigation

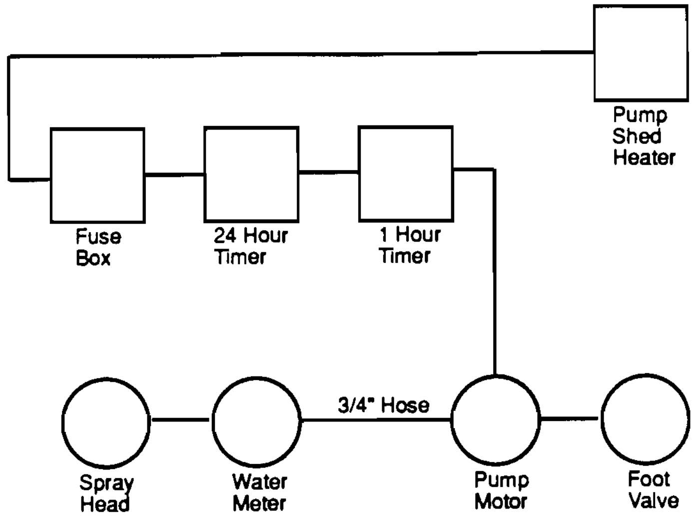


Fig 3.15. Jenkins' spray irrigation automatic electronic control system [21].

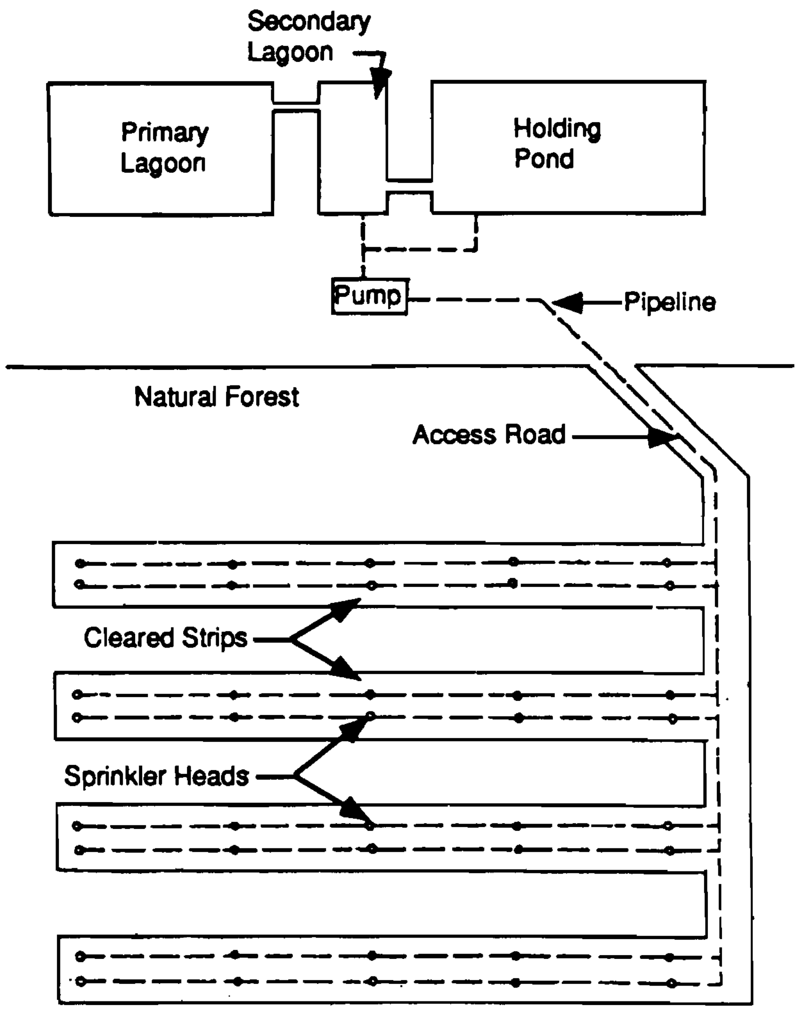


Fig 3.16. Geometry for a rest area spray irrigation system [21].

vary from 2 to 15 acres [23].

Rodman of the Texas Highway Department discussed spray irrigation at Texas rest areas in a 1975 paper [27]. Six rest areas were using spray irrigation following package plants. At that time, the Texas Water Quality Board (TWQB) permits for spray irrigation allowed a maximum application rate of 1500 gallons per day per acre (0.4 in./week) in east Texas and a maximum of 5000 gallons per acre per day (1.2 in./week) in arid west Texas. These requirements meant 2 to 5 acres of land were necessary for irrigation. At the time of the report a rest area in Colorado County was spraying at three times the east Texas application limits in the summers of 1973 and 1974 with no adverse effects. Rodman recommended package plants with spray irrigation for final discharge as replacement systems for failed septic tank/drainfield systems. Present guidelines for spray irrigation permits vary, depending on secondary waste treatment systems used and on geographic location. Wastewater permit rules are jointly formulated by the Texas Water Commission and the Texas Department of Health.

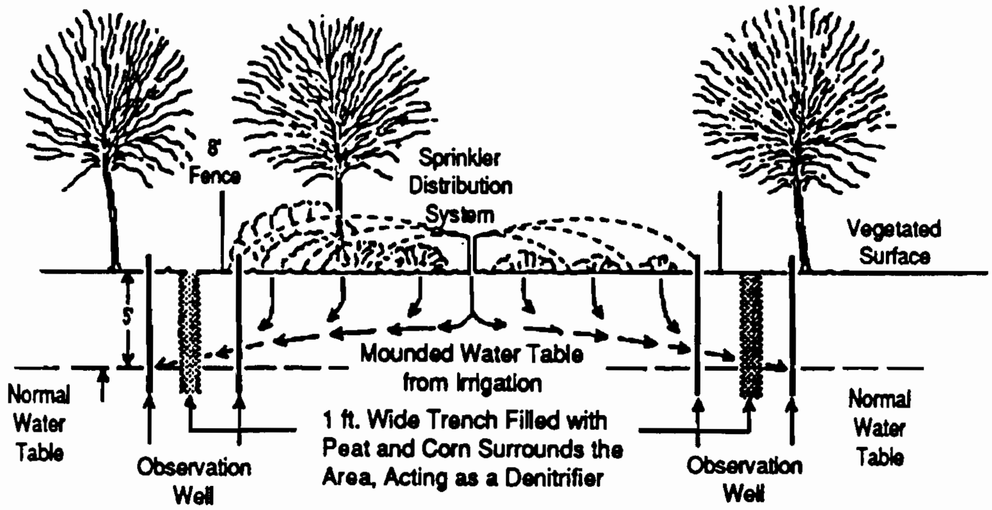


Fig 3.17. Diagram of a barriered landscape water renovation system [20].

3.7.2.2 Overland Flow Performance. Performance of overland flow systems is governed by biological processes. Overland flow is used in areas where soil infiltration is poor so that treatment is accomplished by microbial action and crop uptake. A big advantage in using overland flow is that the renovated water can be easily monitored as it leaves the site. Hydraulic and nitrogen balances are used to calculate land areas, with the larger area chosen. The water balance includes a runoff component as well as those discussed for spray irrigation. Slopes for overland flow are usually 2 to 8 percent and effluent can be sprayed on land or distributed using pipes. Overland flow is an effective treatment method at Paris, Texas, and is able to function at freezing temperatures [2].

3.7.2.2.1 Overland Flow-Evapotranspiration (OF-ET) System. Erickson [20] studied an overland flow-evapotranspiration (OF-ET) system at the Clare Rest Area and Travel Information Center near Lansing, Michigan, in 1978. The system, shown in Fig. 3.18, operated as follows. Rest area effluent entered and passed through two overflow ponds and then was pumped to a 23,000-gallon chlorination tank. The effluent was applied to the field at an application rate of 2.4 inches per week for five days during a week. The weekends were used as resting periods. The effluent was applied via a six-ditch distribution system (which was built to avoid channeling) to a 4-acre overland flow-evapotranspiration field having a slope of 4 percent. The soil profile was 1 to 3 feet of sandy loam overlaying a less pervious clay.

The study was conducted from June 22 to July 31, 1978. Water samples were taken twice a week from the ditches, catchment area, shallow wells, perimeter wells and perched

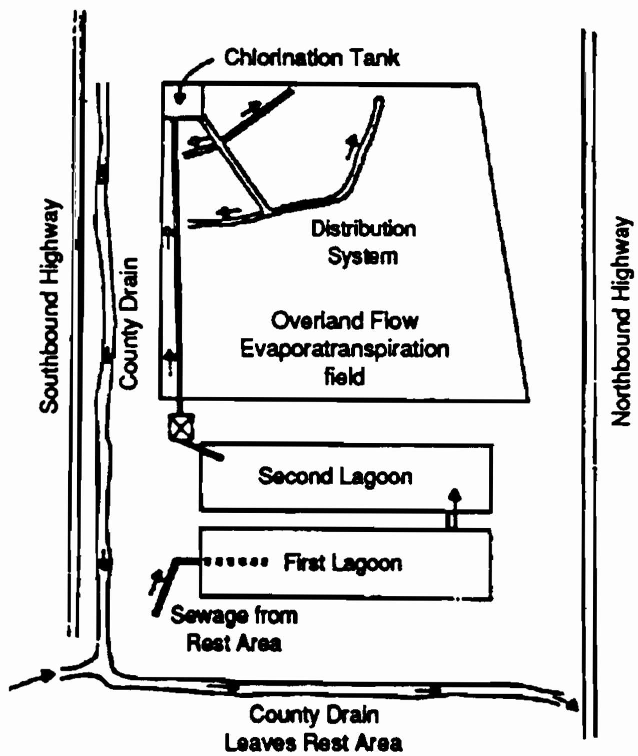


Fig 3.18. Plan view of lagoons and overland flow-evapotranspiration system, Clare Rest Area and Travel Information Center [20].

TABLE 3.13. SUMMARIZED RESULTS FROM WATER SAMPLING DATA FROM OF-ET SYSTEM [28]

Pollutant	Concentration (ppm)	Reductions (%)
BOD	2.5	96
TOC	32.7	48
i-PC 4	.09	97
TKN	1.9	97
NH_3- N	.12	99
NO_3- N	.45	0
TN	2.35	94
Water	-	87

water table under the field. Moderate (2.4 in./wk) and heavy (7.1 in./wk) application rates were tested [20, 28].

Monitoring data showed that the system was never hydraulically overloaded; in fact, during much of the study, there was very little runoff. Water losses were 10, 42, and 48 percent for runoff, evaporation, and infiltration. Therefore, the system was not an overland flow if a strict definition is followed. The overall system final concentration and removal efficiencies are listed in Table 3.13. The land treatment system had removal percentages of 89, 95, 86, and 98 percent for BOD_5, PO_4, TKN, and NH_3. Treatment by infiltration occurred in the soil in a vertical direction while surface water treatment was a function of distance traveled down the field. Infiltration water quality was better than surface runoff quality. Evaporation was large because the field was located on a high point and was subjected to windy conditions.

Nitrate levels were low in groundwater and in the field (~1 ppm) under all hydraulic loadings, with the canary grass in the field being able to uptake heavy nitrogen and phosphorus loadings (37.2 and 4.8 lb respectively). Crop harvesting to remove nitrogen from the system was recommended. Fecal coliform levels in surface waters were high but were attributed to non-human sources.

The report concluded that the limiting factor in the system was the hydraulic loadings. The discharge stream that passed through the rest area and the ground-water beneath the rest area were not contaminated by the overland flow-evapotranspiration system. For a completely evaporative system a 50 percent increase in land area is required (i.e., 6 acres), although the study land area produced effluent that exceeded all state discharge effluent standards.

Erickson also studied use of a watervalet system consisting of a lagoon system followed by a 1600-foot-long vegetated ditch with final discharge to a stream. From extensive sampling, reductions of PO_4 and TKN were found to be 50 and 12 to 35 percent respectively. Nitrates formed from ammonia were effectively removed by reed canary grass planted in the ditch. Erickson concluded that allowing wastewater to flow through a long, well-vegetated ditch can perform as a polishing system for pond effluent [20].

3.7.3 Operation & Maintenance

Land treatment systems generally will require little additional time over that required for the secondary treatment system. Operation of both spray irrigation and overland flow systems is automated with timers so that setting timers and doses is the only operation requirement. Harvesting or cutting of grass is only an intermittent process, which is not expected to require over a few man-hours per week. Chlorination tanks need to be monitored for residuals and chlorinators need to be stocked periodically; however, these tasks are not necessarily additional requirements of the land treatment system, depending on whether the secondary treatment system chlorinates the effluent. Jenkins found no encrustation in pumps, pipes, or nozzles after 5 months of spray irrigation [21].

3.7.4 Summary

Land treatment systems are categorized as irrigation, rapid infiltration, or overland flow systems. Only spray irrigation and overland flow systems are being used at rest areas. These systems are capable of treating pond and package plant effluent but have not been recommended for septic tank effluent. Pollution of groundwaters has not occurred to any discernible amount, because spray irrigation or overland flow and nitrate standards have been met. Both systems have demonstrated the ability to remove high percentages of BOD, nitrogen, phosphorous, and total organic carbon. Land requirements for these systems range from 2 to 10 acres and depend on hydraulic loadings. Examples of climatic summaries and informational needs used in designing land treatment systems are illustrated in Tables 3.14 and 3.15 [13].

Operation and maintenance requirements for these systems are low. Land treatment systems are usually automated so that pump maintenance and setting of dosing times are the main operational requirements. Harvesting of crop covers may be necessary but this will probably take only a few man-hours per week.

3.8 EVAPOTRANSPIRATION BED SYSTEMS

3.8.1 Description of System

Evapotranspiration bed systems are similar to trench or bed absorption systems except that loss of effluent is through evapotranspiration, with losses to the soil being minimal. A typical evapotranspiration (ET) bed is shown in Fig. 3.7. Other ET bed configurations are shown in Figs. 3.19 and 3.20. Evaporation losses are accomplished through the sand and topsoil that is above the distribution pipes. Transpiration losses are by means of cover crops planted over the beds. Sand "wicks" penetrating the lower gravel layers are provided so that continuous capillary action occurs in the sand

TABLE 3.14. CLIMATIC SUMMARY

Parameter	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total (or Avg)
Total ppt, in.	4.3	3.5	5.0	4.6	3.9	3.3	3.8	4.0	4.2	4.6	4.8	4.2	50.2
Mean Days, >0.5 in.	3.0	2.0	4.0	3.0	3.0	2.0	2.0	3.0	2.0	3.0	4.0	3.0	34.0
Evapotrans, in.	0.0	0.0	0.2	1.4	3.2	4.6	5.4	4.3	3.3	1.9	0.8	0.0	25.1
Mean Temp, °F	26.0	28.4	34.3	47.3	57.5	66.3	72.0	69.8	62.2	51.8	41.6	29.4	(48.9)
Mean Daily Min Temp, °F	16.7	16.0	25.0	35.7	46.2	55.3	60.7	58.3	51.4	40.4	31.0	20.8	(38.1)
Min Temp, °F	-21.0	-9.0	2.0	13.0	27.0	35.0	46.0	40.0	28.0	21.0	0.0	-9.0	-21.0
Mean Days, < 32 °F	30.0	26.0	26.0	7.0	1.0	0.0	0.0	0.0	1.0	7.0	16.0	28.0	142.0
Overland Flow	No Application (110 Days)			Low Rate (9 Days)			High Rate (141 Days)		Low Rate (41 Days)		No Application (46 Days)		
Rapid Infiltration						Full Year Operation (365 Days)							
Spray Irrigation	No Application (110 Days)					Operation Period (170 Days)					No Application (61 Days)		

TABLE 3.15. INFORMATIONAL NEEDS AND SOURCES FOR LAND APPLICATION OF WASTEWATER [13]

INFORMATION NEEDS \ INFORMATION SOURCE	LOCAL					COUNTY					STATE						REGIONAL					
	Consultants	Municipal/Community Planning	Chamber of Commerce	NOAA, U.S. Weather Station	Library	Agricultural Extension Service	County Planning	Public Health	Assessor	U.S. Soil Conservation Service	Environmental Protection/Public Health	Historical Society	State Geological Survey	State Div. or Dept. Water Resources	State University Extension Specialists	State Planning Office	Corps. of Engineers	U.S. Forest Service	U.S. Geological Survey	USDA, Agricultural Service	U.S. Environmental Protection Agency	Planning Agency (COG, River Basin Resource Management Districts)
Climatic data				•		•									•							
Soil classification — mapping	•	•					•			•												
Soil infiltration — permeability	•					•				•					•					•		
Soil depth 0–5 ft										•												
Soil drainage and water table < 5ft										•												
Soil properties (chemical & physical)	•					•				•					•							
Agricultural land use capacity	•	•				•	•			•												
Depth to bedrock										•			•						•			
Unconsolidated materials													•						•			
Bedrock type & structural characteristics													•						•			
Jointing & permeability of rock	•												•	•					•			
Rock outcrops										•			•						•			
Surface slope, categories (ex. C-3 pcs.)										•									•			
Floodplain, flood hazard	•	•					•							•			•		•			
Streamflows														•					•			•
Groundwater yield	•												•	•					•			•
Groundwater elevation & contours	•												•	•					•			
Groundwater aquifers	•						•						•	•					•			•
Irrigation methods	•					•				•					•					•		
Crops	•					•				•					•			•		•		
Interpretation of soil suitability	•					•				•					•					•		
Interpretation of groundwater	•												•	•					•			
Land use		•					•		•							•						•
Land values	•	•	•			•	•		•													•
Guidelines for land application	•							•			•						•				•	
Sensitive environmental areas	•	•					•				•	•				•					•	
Socioeconomic factors	•	•	•				•		•							•						•
Institutions (any organization)			•		•		•						•			•						•
Aesthetics		•			•		•															
Data	•	•			•		•															•

layer. Bed liners are used for soils with high permeabilities but are not necessary (unless required by state law) for very low permeable soils [12]. ET beds require more total ground surface area than soil adsorption systems so that usually they are the favored system if soil permeabilities are very low or high (low or high permeabilities are likely to cause ponding or groundwater contamination if soil absorption systems are used).

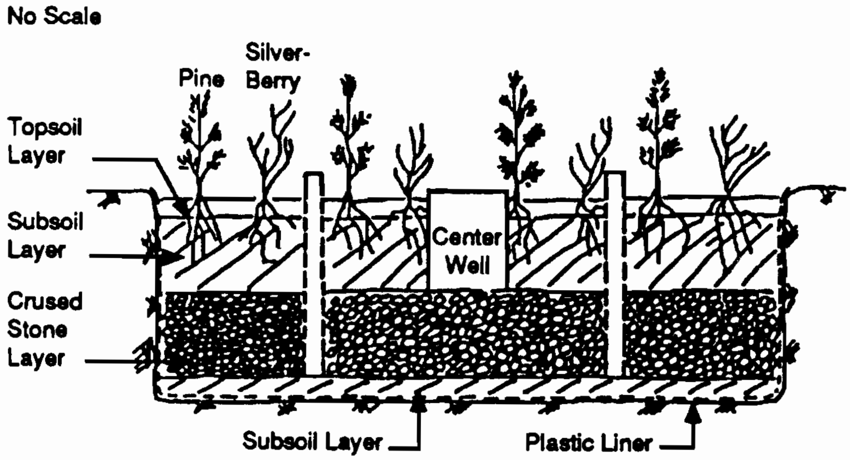


Fig 3.19. Cross-sectional view of ET unit [21].

3.8.2 System Performance

The performance of the ET system is dependent on the ET rate determined for the bed; seepage is assumed to be zero. The ET rate is dependent on many factors, including humidity, wind, temperature, the bed water level, and the cover crop. Pan evaporation rates depend on humidity, wind, and temperature; therefore some correlation should exist between pan evaporation and ET rates.

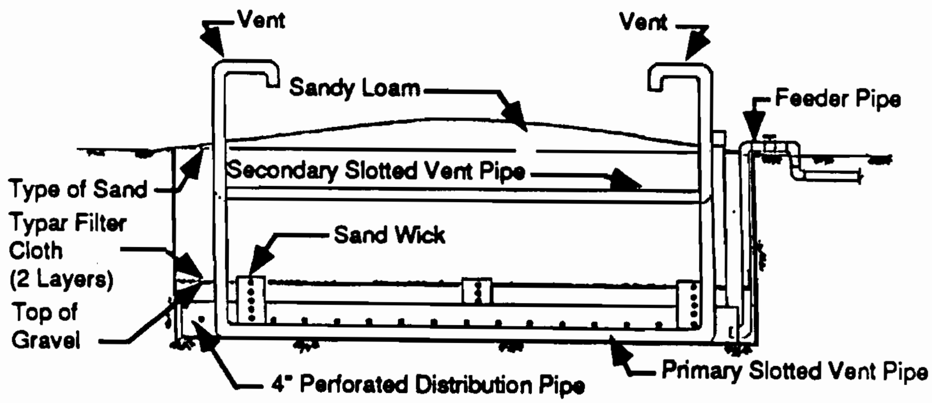


Fig 3.20. Side view of typical test tank [29].

3.8.2.1 Calculation of the ET Rate. Jenkins et al [21] studied ET beds at a rest area in Alabama. Figure 3.19 depicts the beds and Figs. 3.21 and 3.22 show plan views of the system. Water levels in the beds were kept constant by using pumps when necessary (during heavy storms or drought periods). The three units shown in Fig 3.22 were monitored for 74 weeks to determine ET rates. Using data from unit #2 and the following formula Jenkins calculated ET rates as follows::

$$ET\ (\text{in./wk}) = (A - B + C + D)\ (12\ \text{in./surface area of bed, ft}^2) \quad \text{(Eq. 3.3)}$$

where

A = Volume pumped into unit, ft^3 (gallons pumped/7.48 gal/ft^3)

B = Volume pumped out of unit, ft^3 (gallons pumped/7.48 gal/ft^3)

C = Rainfall volume, ft^3, [(in. of rainfall/12) x surface area of catchment area]

D = Change in pool elevation, ft^3, [change in pool elevation in piezometer x porosity in decimal form x surface area of bed).

In this equation D is positive for a decrease in water level in the piezometer.

Jenkins calculated ET rates for 74 weeks on a weekly basis and compared them to U.S. Weather Service pan evaporation rates over the same periods. A pan coefficient of 0.7 was found by dividing monthly ET rates by monthly pan rates. Thus the ET rate, according to Jenkins, is equal to the pan evaporation rate multiplied by 0.7.

Rugen et al [29] went a step further in calculating ET rates, by observing changes in the ET rate for different bed effluent levels and for different crop covers at a site in San Antonio, Texas. The ET bed configuration shown in Fig. 3.23 illustrates the four groups of crop covers tested at three different water levels within each group. Water levels were maintained at 7, 10.5, and 14 inches above the tank bottoms in each cover crop group. Steel tanks were 6 feet long, 32 inches wide, and 24 inches high. The tank used is similar to that shown in Fig 3.20 A weather station provided data on maximum and minimum air temperatures, pan evaporation, maximum and minimum pan water temperatures, wet and dry bulb temperatures, rainfall, and wind speed (miles/day).

Bed effluent levels were maintained by adding effluent through piezometers.

Study results showed that higher ET rates occur for higher tank effluent levels. Bare cover tanks (the control) showed the most significant relationship, which is shown in Fig. 3.24. Pan evaporation and ET rates were compared for different tank effluent levels and were graphed as shown in Fig. 3.25. The concave curves (in the positive x direction) for effluent levels of 11, 13, and 15 inches were not expected, based on Fig. 3.24.

It appears that at high ET rates (high effluent levels) the water is lost so quickly that the top of the bed dries out, which blocks capillary movement of effluent upward through the bed [29]. Thus, at high pan evaporation rates the ET rate decreases. At low pan evaporation rates the level of ET increases as pan evaporation rates do; ET increases because of a slow but steady capillary flow of water [29]. The researchers recommended using a bed effluent level above 10 inches below the ground surface for ET systems.

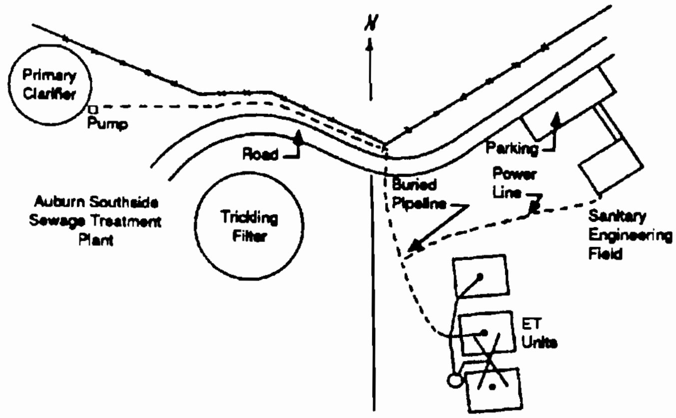


Fig 3.21. Site of ET units (Jenkins).

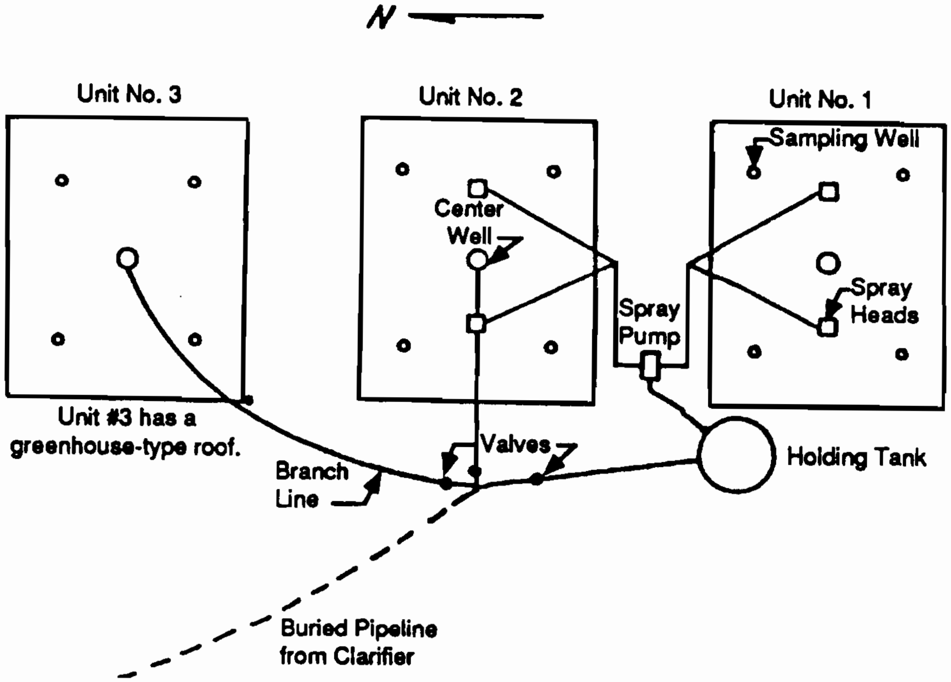


Fig 3.22. Plan view of ET units (Jenkins).

Vegetative cover had an effect on ET rates. Cover grasses used were natural grasses of Texas, coastal bermuda, and carpet grass. Percentage increases of ET for each cover crop were compared to the bare tank ET rates for the same bed effluent level. ET rate increases occurred for each cover type for each pan evaporation rate tested. Carpet grass was the best cover crop (Fig. 3.26).

3.8.2.2 Calculation of Bed Surface Area. The Texas Department of Health guidelines (1977) suggest the following formula to calculate bed surface area:

$$\text{Area (sq ft)} = (310 \times Q)/EA \qquad \text{(Eq. 3.2)}$$

where

Q = average daily flowrate int the system (gal/day)

EA = local pan evaporation (in/yr)

The manual does not define the units of area, and therefore, square feet is assumed. The value of 310 is mysterious - it is not a conversion factor alone. The manual does not mention an effluent bed level to maintain for the bed area calculated above.

Rugen et al developed a successive approximation method to size ET beds, which is presented in Appendix G. In the approximation method a suitable area and depth are selected and then successive adjustments are made to the first guess.

3.8.3 Operation & Maintenance

No information could be found on maintenance and operation costs of ET systems. If the system is based on average daily flowrates, a holding tank and pump will be necessary. The planting, establishment, and cutting of cover crops are necessary. The man-hours needed for the system are expected to be between those for septic tank/drainfields and for pond systems. Maintenance requirements will be high if flooding occurs.

3.8.4 Summary

Use of ET beds for rest area wastewater treatment systems is virtually non-existent. The success of the system is dependent on the correct estimation of ET rates for the ET beds. ET rates can be correlated to pan evaporation rates for different effluent levels and crop covers according to Rugen et al [29].

The maintenance of a constant bed effluent level is necessary for proper cover crop growth and may require the use of pumps. Grasses may be less sensitive to bed effluent level fluctuations than trees. Treatment efficiencies of ET beds were not calculated in studies reviewed so that groundwater contamination risks from ET beds are not known. Maintenance and operation requirements of ET systems are expected to fall somewhere between those for pond and septic tank/drainfield systems.

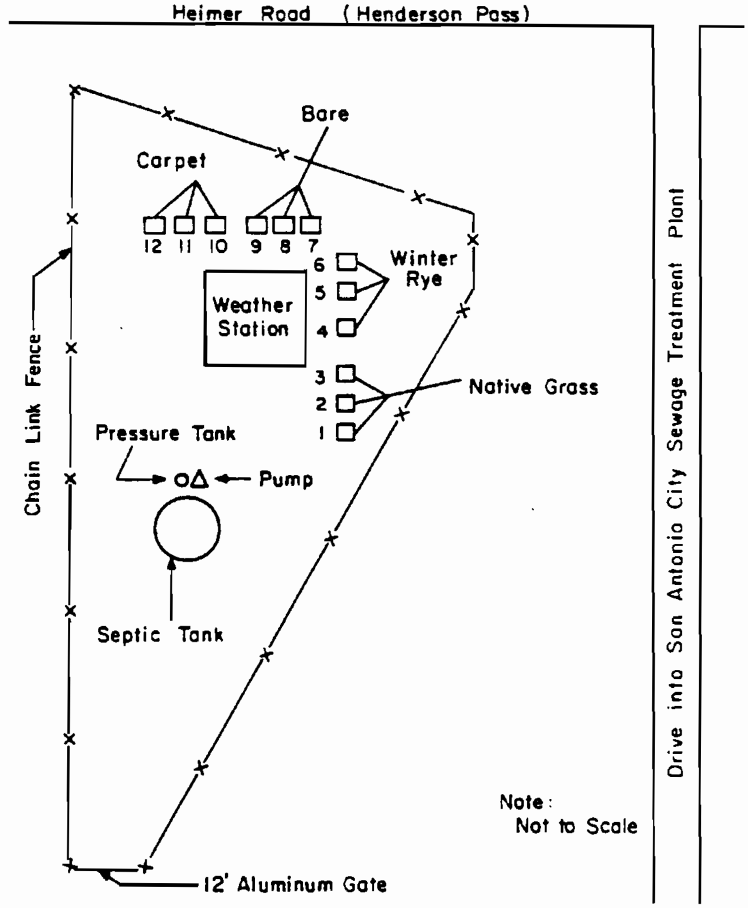


Fig 3.23. Evapotranspiration research site plan, San Pedro Hills Sewage Treatment Plant [29].

3.9 RECYCLE/REUSE SYSTEMS

3.9.1 Description of System

Recycle/reuse systems may use water or mineral oil, which is recycled in the treatment system. Recirculation systems using toilets as individual holding tanks of wastes until they reach capacity and are pumped out are not covered in this report because it is felt these systems are undesirable for rest area wastewater treatment systems. The approach in this section is to review in detail two studies which evaluated the performance of recycle/reuse systems. The first system reviewed is the Monogram Magic Flush mineral oil sewage disposal system [30] and the second system reviewed is a water recycle system used at a rest area in Virginia [31,32].

3.9.2 The Monogram Magic Flush Mineral Oil Sewage Disposal System

3.9.2.1 Reason for Use of the System. In 1979 the North Carolina Department of Transportation elected to use the Monogram Magic Flush system at a rest area in north-central North Carolina. The rest area was located in a remote densely forested area with a very high water table and low soil percolation rates. A septic system was ruled out because of low percolation rates and a package plant was ruled out because of strict state discharge effluent standards, which required tertiary treatment for stream discharge. In addition, the amount of flow allowed by government authorities was only half of what the rest area would produce. Thus strict regulations and natural conditions of the site led to the choice of the mineral oil system.

3.9.2.2 Description of System. The mineral oil system includes

1. a waste holding assembly,
2. a central filtration unit, and
3. an accumulator module.

The waste holding assembly consists of a 2000-gallon holding tank, a mixer motor assembly, a float assembly, and a pump-out assembly. The central filtration unit consists of a 3-HP filtration pump, two coalescer filter assemblies, a filter media tank, and a 40-gallon clean oil reservoir. The accumulator module consists of a 3-HP accumulator charging pump, two 61-gallon hydropneumatic accumulator tanks, eight water closets, and eight service fixtures. A flowsheet of the system is shown in Fig. 3.27.

3.9.2.3 Operation of the System. The mineral oil system operates as follows:

1. Wastes flow from the toilets to the waste holding tank, where oil and wastes are separated by virtue of their differing specific gravities (i.e., oil floats and waste sinks). The mixer motor is used to break up waste aggregates and enhance uniform settling.
2. When the tank float reaches a predetermined level the filtration pump is activated and pumps the oil to the coalescer filters, which remove large solid contaminants and water droplets from the oil.
3. The oil then travels to the filter media tanks (molecular sieves), where fines droplets of gaseous and aqueous matter, and color and odor, are removed.
4. The oil enters the clean oil reservoir to await pumping.
5. Oil is pumped from the reservoir upon demand to the hydropneumatic tanks, which operate in a pressure range of 20 to 40 psi.
6. The hydropneumatic tanks supply oil to the toilets upon demand and the cycle is repeated.

3.9.2.4 Performance of the System . The mineral oil system was operated from May 8, 1979, to December 31, 1981, under both high and low rest area use conditions. Manufacturer claims for waste holding tank pumpout schedules (every 30,000 toilet uses) and filter replacement in the coalescer and sieve filters (every 500 hours of use) were surpassed. The average time between pumpouts was 21 days, which translated to an estimated 56,916 vehicles. Filters were replaced for the first time after thirteen months of operation and only three replacements were necessary over thirty months of operation.

A major source of problems in the system was the waste holding tank. The manufacturer suggested keeping the pH of the tank at pH=9 or above. Mounding of wastes in the tank caused the pH to drop below pH=9 for the first six months of operation. Ammonium hydroxide was added to the tank in December, 1979. By September, 1981, the addition of ammonium hydroxide had caused: (1) corrosion of service fixtures, (2) failure of the central filtration unit, when stainless steel screens holding the filters corroded in place and collapsed with expulsion of the filter media, and (3) deterioration of brass float balls in the holding tank to such an extent that they fell to the bottom of the tank, which resulted in loss of the pump triggering mechanism.

A second source of problems was the electrical system. Gases from the waste holding tank passed through a conduit to the main control panel causing corrosion of electrical components inside the control panel. There also was a corrosion problem with the waste holding tank electrical sensor contract probes. In early 1981 replacement of the sensors and continual sanding of breaker contacts was necessary for the probes to function. The tank mixer rotor also was estimated to need replacement in 1982.

Oil loss was another major concern. The oil replacement schedule for the system is shown in Table 3.16. If the system had operated without any major breakdowns it was estimated that 400 gallons of oil would have been consumed through waste absorption.

In summary, although the system worked well in terms of pump-outs and filter replacements there were concerns over the problems of oil loss, central filtration unit integrity, and corrosion of electrical system components. Loss of oil is an inherent feature of the system and is directly related to intensity of use. A major failure in the central filtration unit in September, 1981, raised questions about long-term filtration performance, and electrical maintenance will be a problem in the system.

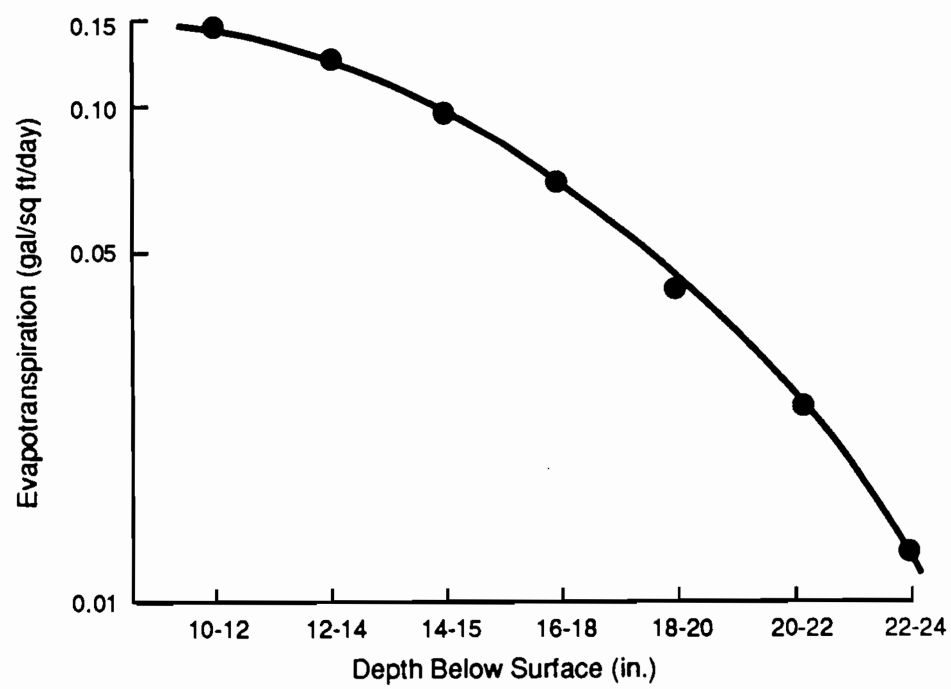


Fig 3.24. Relationship between effluent level and evapotranspiration, bare tanks, April 1976 [29].

3.9.2.5 Recommended Design Changes. The following design changes were suggested:

1. Provide dual service lines and fixtures from the coalescer filter units onward to the hydropneumatic accumulators. This change would permit split service to each building compartment and would preclude total system shutdown in the event of one central filter failure.
2. Increase the waste holding tank size to reduce oil absorption losses in the tank and increase the thickness of the oil layer above the waste.

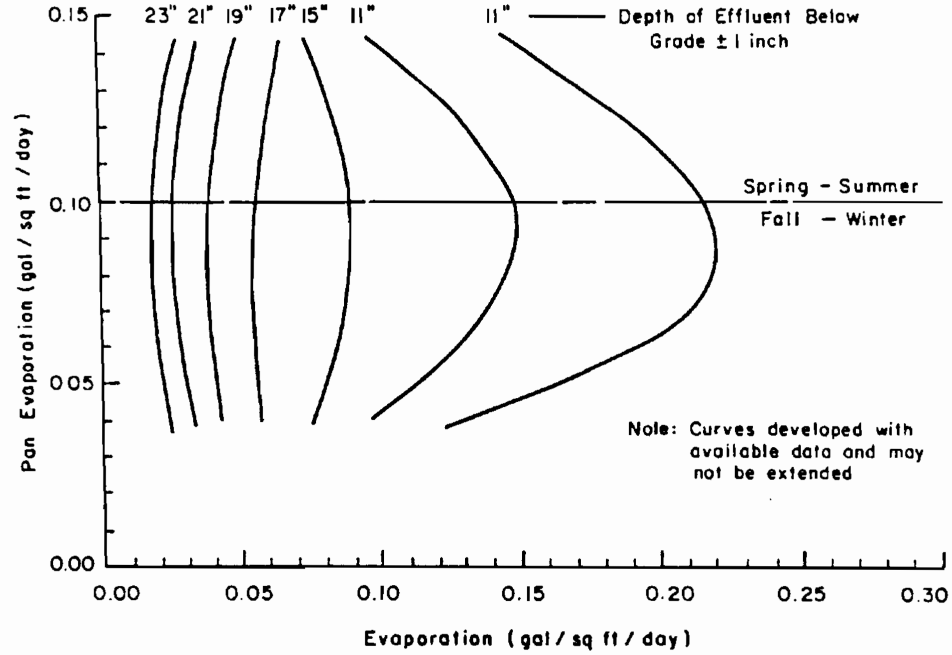


Fig 3.25. Relationship between pan evaporation and evapotranspiration, bare tanks.

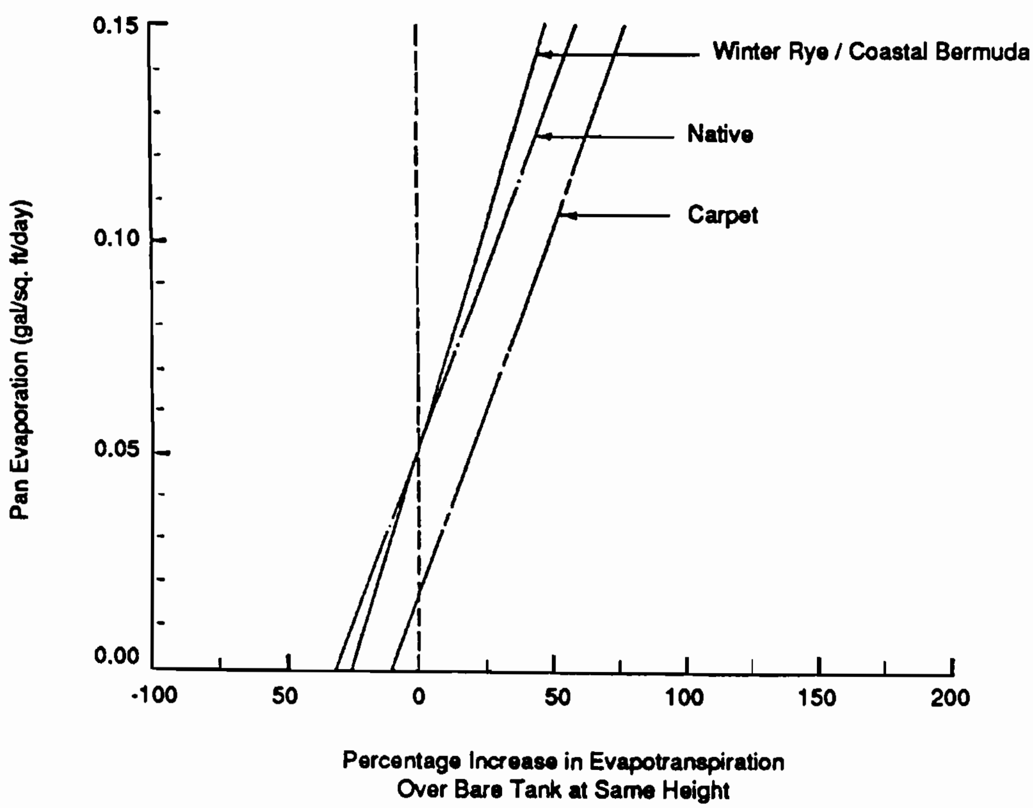


Fig 3.26. Influence of grass cover on evapotranspiration [29].

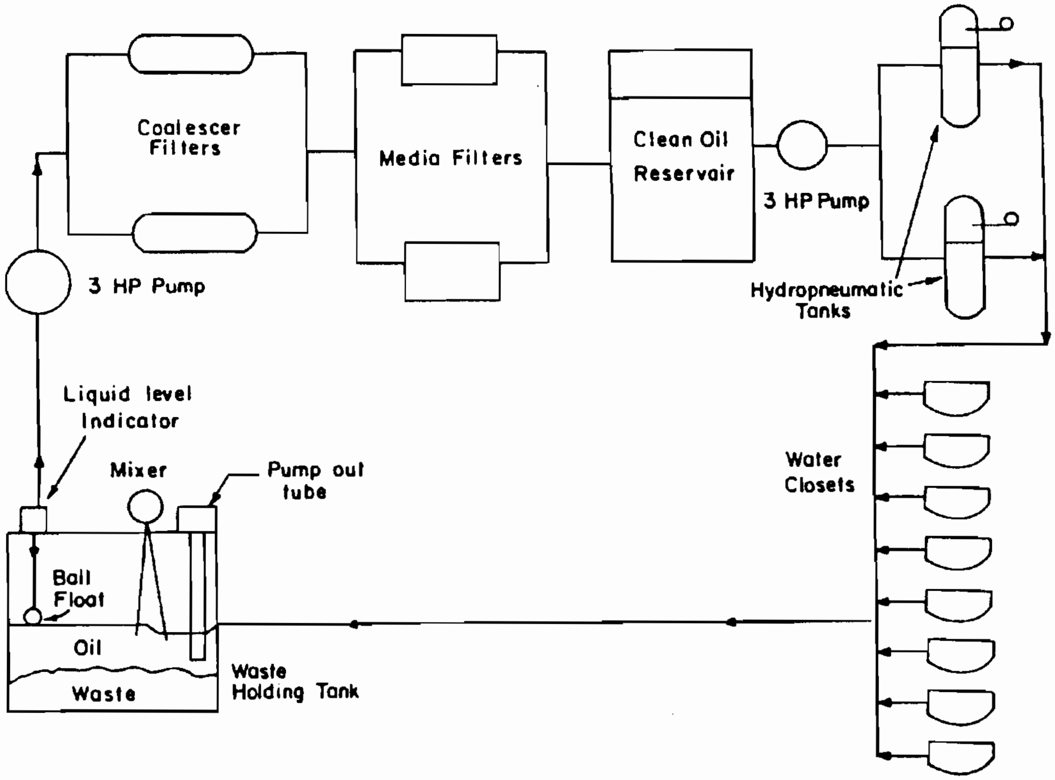


Fig 3.27. Monogram magic flush flowsheet.

3.9.3 Water Re-use System at Fairfield Rest Area, Virginia

3.9.3.1 Reason for Use of System. The purpose of a re-use system at the site was to conserve water. Water shortages were experienced at the rest area and high costs associated with hauling water by truck, expanding well capacity, or hooking up to the municipal system would have resulted.

3.9.3.2. Description of System. After a successful bench-scale study by Parker [31, 32] reuse components were added to the existing 10,000-gallon-per-day extended aeration package plant used at the rest area. The system was modified by adding a pressure filter unit, pre- and post-filtration tanks, a stabilization tank, a hydropneumatic tank, and a final holding pond. A diagram of the system is shown in Fig. 3.28.

TABLE 3.16. RECIRCULATING OIL REPLENISHMENT SCHEDULE [30]

Date	Amount	Comments
August 22, 1979	495 gallons	Total pumpout recommended by manufacturer
January 3, 1980	80 gallons	Leakage around water closets and absorption by waste materials
January 9, 1980	85 gallons	Leakage around water closets
March 18, 1980	20 gallons	Absorption by waste materials
May 13, 1980	35 gallons	Absorption by waste materials
June 4, 1980	35 gallons	Absorption by waste materials
June 30, 1980	15 gallons	Absorption by waste materials
July 20, 1980	40 gallons	Absorption by waste materials
May 10, 1981	190 gallons	Total pumpout of system; recharging with 440 gallons of new oil and removal of 250 gallons of used oil
August 6, 1981	30 gallons	Absorption by waste materials
September 9, 1981	495 gallons	Rupture of separator screens in filter media tank required total pumpout of system

Total Replenishment Volume - 1,520 gallons

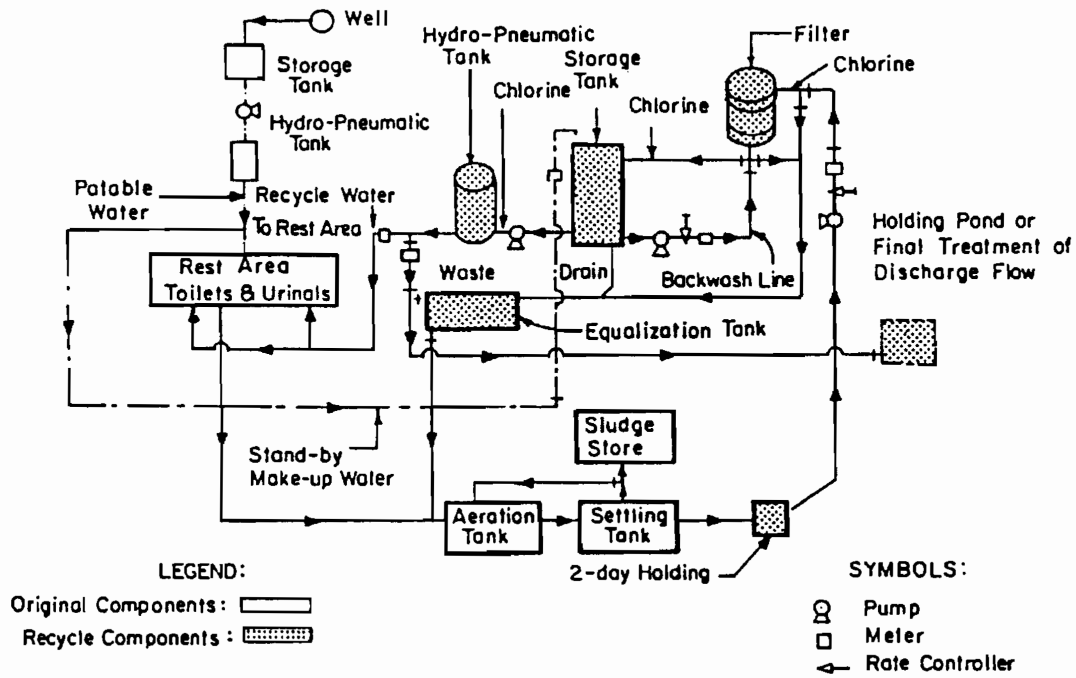


Fig 3.28. Parker's modified package plant recycle system [31, 32].

3.9.3.3 Operation of the System. The system shown in Fig. 3.28 operates as follows.

1. Raw wastewater is treated in the package plant and flows to a 20,000-gallon pre-filtration storage tank and remains there until the filter is triggered.
2. When the post-filtration tank float gauge reaches a pre-determined level, water from the pre-filtration tank is drawn through the filter to the post-aeration tank. If water cannot be supplied fast enough from the filter to the post-filtration tank to meet hydropneumatic tank demands for water, the potable water source is tapped to assist in meeting demands.
3. The 6-foot-diameter pressure filter runs on a filtration rate of 2.3 gpm/sq ft when the post-filtration tank initiates filter runs. Backwash cycles are initiated if the pressure difference between the top and bottom of the filter is 6 psi. The backwash cycle consists of a surface cleaning rate of 30 pm/sq ft for 5 to 10 minutes which is followed by 5 to 10 minutes of backwash at a rate of 8.5 gpm/sq ft.
4. Water from the post-filtration tank is pumped to the hydropneumatic tanks when the tank pressure drops below 40 psi. Chlorination is provided at the points shown in Fig. 3.28 and all chlorine is supplied by the same chlorinator.
5. Water flows from the hydropneumatic tank to the toilets upon demand and the cycle outlined here is repeated. A percentage of the water in the system must be wasted each day to the holding pond.

3.9.3.4 Performance of the System - 1st Evaluation. The reuse system was operated from November 15, 1976, to January 21, 1977, and from March 15, 1977, to August 31, 1977. The rest area was closed during a period of freezing experienced at the site in January.

3.9.3.4.1 Water Recycle Ratios and Water Quality. The water recycle ratio is defined as the recycle water used divided by the total water used, as follows:

$$\frac{\Delta \text{ Accumulated Flush Vol}}{\Delta \text{ Accumulated Flush Vol } + \Delta \text{ Potable Water Vol}}$$

The method used to calculate the recycle ratio is presented in Appendix H. Parker found a recycle ratio of 95 percent on the average. Water was reused 20 times on the average and did not have any detectable odors. Less than 5 pounds of turquoise dye was added to the flush water during the study period and there were no complaints about the flush water from rest room users. The recycle water was clear and stable and had a constant relationship between total suspended solids and total volatile suspended solids. System evaluation indicated that an equilibrium existed between user source residuals and potable water inputs versus evaporative loss of water and residual materials loss through overflow of water to the lagoon.

3.9.3.4.2 Biological Treatment Performance. A shift in the microorganism populations was observed with changes in aeration tank conditions. In the summer, filamentous fungi were dominant in the aeration tank, since the pH ranged from pH=5.6 to pH=6.5 and the alkalinity was less than 50 mg/L as $CaCO_3$. In the winter, organisms which survived best in the pH range of pH=7 to pH=8.3 and alkalinities of 100-500 mg/l as $CaCO_3$ dominated. It was observed that ammonia, nitrites (NO_2^-), and nitrates (NO_3^-) accumulated in the system but not to a level toxic to heterotrophic organisms in the aeration tank.

Nitrogen build-up in the system was less than expected, based on results of bench scale studies. Complete conversion of ammonia to nitrate did not occur because of the low pH and alkalinity. Increased oxygen uptake by nitrifiers was modified by the low summer pH and alkalinity. Ammonia stripping, which occurs at a pH > 7, also occurred in pipes where sufficient agitation and a pH=8.3 caused the ammonia to volitalize.

Sludge wasting over the period March 15 - August 31, 1977, was 6000 gallons. Filamentous fungi were adequately separated from the effluent by gravity settling and thus there were no settling problems at high mixed liquor suspended solids (MLSS). The supernatant was clear and contained no more suspended solids than non-recycle/reuse plants. Foaming in the aeration tank was controlled by defoaming agents and the dye added to flush water was not removed by microorganisms in the system.

The standard BOD test was a poor indication of system performance due to nitrifier oxygen uptake. The most useful parameters for system evaluation were pH, alkalinity, MLSS, and settleable solids, and adequate effluent was produced for the following values of these parameters:

Parameter	Value
pH	5.5-8.3
Alkalinity	50-500 mg/L
MLSS	3000-5000 mg/L
Settleable solids	200-850 ml/L

(MLSS settled in graduated cylinder after 30 minutes)

3.9.3.4.3 Filter Performance. Evaluation of the pressure filter revealed that is was overdesigned. Properly sized filters should backwash every 2 to7 days; in this study [31, 32] three backwash cycles occurred at intervals of 28, 90, and 40 days. The total water treated in the study was 880,000 gallons and the suspended solids concentration after filtration was 10 to 15 mg/l. The researchers concluded that the pumps, tanks, and filter had been overdesigned by a factor of 2.

3.9.3.5 System Performance - 2nd Evaluation. A follow-up study from September 1, 1977, to August 31, 1978, was completed. The overall results were the same as those for the first study except for a problem detected in the recycle ratio calculations.

The recycle ratio had been underestimated previously. The error was due to meter measurements. The meter that measured potable water added to the system did not separate potable water from the water used for landscape irrigation and therefore less water was added to the reuse system than previously calculated (see Appendix H for recycle ratio calculation example). As a result potable water was added to the system in an effort to maintain a 95 percent recycle ratio. Sludge wasting also was increased to maintain a 95 percent recycle ratio.

A transient increase in alkalinity was experienced. The added alkalinity caused extensive nitrification, with increases of nitrates and oxygen uptake. After the transient period the nitrification level was reduced to former levels.

3.9.3.6 Overall Conclusions on the Water Reuse System. The overall conclusions on the water reuse system were:

1. Reuse systems can produce acceptable effluent quality for flush water.
2. Design parameters for reuse systems are pH, alkalinity, MLSS, and settleable solids.
3. The biological system can still perform satisfactorily despite seasonal changes in microorganisms in the system.
4. Separate meters to determine water used in landscape irrigation and in rest rooms are necessary.
5. A recycle ratio of 95 percent is optimal, with a water reuse of 20 times being an optimal level for the biological system.
6. The system is capable of zero discharge with a holding lagoon.
7. Optimal sizing of the hydropneumatic tank, the filter, and pumps should be based on water use analysis. A clear distinction between instantaneous flow sed for pipe design and lower flowrates for other system components must be made.

3.10 RECREATIONAL VEHICLE WASTE DUMP FACILITIES

3.10.1 Problem

Recreational vehicle (RV) waste dump facilities present a problem in the design of rest area wastewater treatment systems. RV wastes have high organic strength (i.e., high COD, BOD, and SS concentrations) and contain toxic or inhibitory substances. RV waste stations will increase organic loading to the rest area treatment system while at the same time introducing inhibitory substances which will affect the growth and reproduction of microorganisms that degrade the wastes. The effects of RV wastes may be short term or long term, depending on the loading from RVs and the ability of microorganisms to acclimate to RV wastes.

3.10.2 RV Wastewater Quantity and Quality

RV wastewaters can vary greatly in composition and quantity. The quantity of wastewater per RV is dependent on the sum of black water (toilet), grey water (sink & kitchen), and rinse water. Some RV owners will use more rinse water than others and will clean out the holding tank at different levels of waste, and, thus, wastewater production per RV can vary significantly. RV wastewater quality is also highly variable due to the practices of different owners.

RV wastewater characteristics are listed in Table 3.17 [33, 34]. The wastewater production from RVs is between 16 and 21 gallons per RV and the organic strength in black water is very high. Formaldehyde concentrations in black water vary from 250 to 280 mg/L, with a large standard deviation. Note in Table 3.17 the high standard deviations for individual samples, which show the variation of RV waste characteristics from one vehicle to the next.

TABLE 3.17. RV WASTEWATER CHARACTERISTICS (BLACK AND GRAY WATER)

Parameter	Pearson et al (1980) [33]	Keirnan et al (1983) [34]
Number of composite samples	14	72
Water use per RV, liters (gallons)	80 (21)	62 + 10 (16 + 2.7)
Standard Deviation (L)	-	43 (11)
TSS, mg/L	3850	3120 + 490
Standard deviation	3735	2120
VSS, mg/L	3330	2460 + 410
Standard deviation	3130	1780
COD, mg/L	6210	8230 + 1430
Standard deviation	1715	6140
BOD, mg/L	3080	3110 + 530
Standard deviation	2700	2200
Formaldehyde, mg/L	18	-
Standard deviation	11	-
Formaldehyde, mg/L (black water only)	280	250 + 60 (2)
Standard deviation	310	180

Notes (1.) Standard deviation for individual RV waste samples.
(2.) Samples from RV owners using formaldehyde only; not combined with wastes from RVs using other types of additive substances for antiseptic purposes.

3.10.3 Dump Station Usage and Loading Estimates

Kiernan et al studied RV dump station usage in Washington. Disposal station use during maximum monthly usage (July and August) was 3 and 6 percent of the vehicles entering the rest area for western Washington (heavy commuter) and eastern Washington rest areas respectively. The percentages of average daily traffic stopping at rest areas in western and eastern Washington rest areas are estimated to be 5 and 10 percent respectively, so that RV users as a percentage of average daily traffic (ADT) are 0.15 and 0.6 percent for eastern and western Washington rest areas [34].

The maximum hourly rest area usage was 11 RVs per hour and the maximum daily use 230/80 times the daily average of RVs stopping (230/80 is a peaking factor based on usage data at a high use rest stop). Caution should be used in applying the peaking factor because maximum ADT and maximum dump station usage do not coincide. Dump station usage in California was 2.26 (1979) and 1.41 (1980) percent of ADT; the lower value in 1980 is related to higher gas prices [35].

Hydraulic and organic loadings can be computed using mean values for holding tank plus rinse water. A mean value of 16.5 gallons per RV was reported (Table 3.17) by Kiernan. Estimated loading rates per RV for COD and suspended solids were 0.63 and 0.39 pounds for RV black/gray wastes (not including flush water) [33]. The estimates shown in Table 3.17 agree for suspended solids but differ for COD concentrations [33,34]. The maximum RV use per day was used to calculate maximum loadings at rest areas in western

TABLE 3.18. MAXIMUM ORGANIC LOADINGS, TWO WASHINGTON REST AREAS

Parameter	SeaTac Rest Area	Selah Creek NB Rest Area
Disposal Station Use (RVs/day)	80	25
Volume	5.0 m /d (1320 gal/d)	1.6 m /d (423 gal/d)
TSS	15.2 kg/d (33.5 lb/d)	4.75 kg/d (10.5 lb/d)
VSS	12.0 kg/d (36.5 lb/d)	3.75 kg/d (8.27 lb/d)
COD	40.8 kg/d (90.0 lb/d)	12.8 kg/d (28.2 lb/d)
BOD	15.2 kg/d (33.5 lb/d)	4.75 kg/d (10.5 lb/d)
Formaldehyde 0.88 kg/d (1.9 lb/d)	0.28 kg/d (0.62 lb/d)	

(SeaTac) and in eastern (Selah Creek NB) Washington; results are shown in Table 3.18.

3.10.4. RV Holding Tank Additive Effects on Biological Treatment

Holding tank preservatives, such as formaldehyde (HOCH), zinc sulfate, and phenols, often are used by RV owners for odor control, for preservation of holding tank wastes prior to disposal, and to enhance liquefaction [34]. The majority of products used have formaldehyde or paraformaldehyde as the active ingredient, and zinc and phenol compounds have all but disappeared from the market [34]. Formaldehyde partially or totally can inhibit microbial activity in treatment systems and thus is a very important design parameter for RV dump stations. Formaldehyde toxicity to anaerobic and aerobic bacteria has been studied by a number of investigators and is summarized in Table 3.19. These data can be used to estimate that continuous formaldehyde concentrations greater than 100 mg/l (HOCH) will have profound effects on bacteria unless they are acclimated to formaldehyde. Formaldehyde toxicity to algae is very low (~5 mg/l), which may be an important factor in pond treatment. Pearson et al subjected anaerobic bacteria to continuous and shock formaldehyde loadings and found that shock loadings have less of an effect on microbial activity until the formaldehyde concentration reaches 400 mg/l [33].

TABLE 3.19. FORMALDEHYDE TOXICITY (CONTINUOUS LOADING)

Investigator	Year	Type of Organism	Formaldehyde Toxicity or Inhibitory Concentration, mg/l	
Kiernan	1983	Anaerobic	40, 10 % decrease in gas production	
		Aerobic	>100 total inhibition	
Pearson	1980	Anaerobic	200, 50% inhibition	
Yang	1979	Anaerobic	100-400 inhibition	
			500	Toxic
		Acclimated Anaerobic	5700	
Gellmum & Heukelekim	1950	Anaerobic Act. Sludge	135-175	Toxic
		Acclimated	1750	Toxic
Bringham & Kuhn	1976	Algae	.3-.5	Toxic
Helms	1976	Algae	6-20	Toxic

TABLE 3.20. EFFECT OF 300 mg/L FORMALDEHYDE IN RV BLACK WATER ON SEPTIC TANK GASIFICATION AND LIQUIFICATION RATES (PEARSON, 1980)

Method	Relative Activity (1)	Formaldehyde Removal
Gasification Rate, Shock Loading	37%	59%
Gasification Rate, Cont. Loading	46%	84%
Liquification Rate, Shock Loading	~100%	~100%
Liquification Rate, Cont. Loading	68%	.85%
Recommended Design Value	40%	70%

(1.) Gasification or liquification rate in presence of 300 mg/l of formaldehyde relative to rate in absence of formaldehyde.

Relative activity and formaldehyde removal percentages for a formaldehyde concentration of 300 mg/l as found by Pearson [33] are listed in Table 3.20.

RV black water wastes average around 250 mg/l of formaldehyde and are a threat to biological systems. Although dilution by gray and rinse water can be expected, it can not be relied upon. Dilution of RV wastes with rest room wastewater is a possible treatment strategy to reduce formaldehyde concentrations. Another possibility is to use a separate acclimated biological system to treat RV dump station wastes.

3.10.5 RV Dump Station Effects on Treatment Systems

The effects of RV dump station wastes on rest area wastewater treatment systems vary with the treatment system used.

3.10.5.1 Septic Tank/Drainfield. RV dump stations will cause more frequent pumpouts of septic tanks sludge and scum and may mean an increase in drainfield size. Pearson et al have developed design procedures for septic tanks and drainfields receiving RV wastes, and they are shown in Appendix I [33]. Pearson used first order kinetics to predict sludge accumulation in the septic tank and arrived at pumpout intervals for tanks as follows:

Pumpout interval, months	Septic tank detention, days
3	1.7
6	3.3
12	6.2

Kiernan [34] suggested tanks be sized for a one-year accumulation of sludge and that the drainfield size be doubled to treat RV wastes. Pearson sizes the drainfield by maximum daily flowrate (see Appendix I).

Formaldehyde concentration reductions in septic tank/drainfield systems appear to be large and final tank and drainfield concentrations are 5 to 10 mg/L HOCH [33,34]. Formaldehyde reactions in the tank and drainfield plus settling of formaldehyde to the bottom of the tank are two processes which occur. It appears that formaldehyde concentrations cannot be reduced below 5 mg/l [34].

3.10.5.2 Pond Systems. The results of the study in Washington suggest that evaporative pond system performance is not affected by RV dump stations [34]. A dilution ratio of 5:1 of rest room wastewater to RV wastes was recommended to dilute RV wastes so formaldehyde concentrations will be lowered, thus reducing the chance of killing algae in the pond. A mixed culture of several species of bacteria and algae in pond environments makes the pond system less sensitive to formaldehyde than a pure test algae [34].

3.10.5.3 Extended Aeration Package Plants. The state of Louisiana provides RV dump stations at all its rest areas, and all rest areas use package plants. RV wastes are sent directly to the plants for treatment and no plant operational problems have been reported [22]. RV wastes can cause increased utilization of oxygen in the plant due to heavier organic loadings, but no other major effects have been observed [34].

3.10.5.4 Other Systems. Municipal treatment systems will not be affected by RV dump stations because of the large dilution that will occur at the plant. Whether formaldehyde will cause problems in land application systems or evapotranspiration beds is unknown.

3.10.6 Operation and Maintenance

The operational and maintenance costs of RV dump stations are high because of vandalism and improper use of the dump facilities [34]. Potential dumping of toxic wastes into stations is an eminent danger and potentially a costly aspect of RV dump stations. Use of maintenance and state patrol personnel to protect the stations against illegal dumping will distract them from their regular duties. The stations will require regular attention to keep them clean and operating properly.

In Washington the annual cost of an RV dump station is between $128,000 and $215,000 dollars a year (1983) [34]. The costs of the dump stations are paid for by a one dollar add on license fee for RV owners. This fee has marginally paid for the dump stations but results of a mail survey of RV owners in Washington suggest a larger fee can be charged.

3.10.7 Various State Policies Towards RV Dump Stations

Experience with dump stations varies widely among states. Some states are phasing out dump stations at rest areas because of high maintenance costs and problems with treatment system operation. These states are California, Montana, and Idaho. Other states, such as Louisiana and Washington, have not reported problems as a result of RV wastes [22]. Arizona policy prefers septic tanks for treating RV wastes. Nevada and Oregon use separate treatment systems for RV wastes. In general, there is no consensus among the states on the desirability of providing RV dump stations.

3.11 WATER SAVING TOILETS

3.11.1 Problem Statement

Water saving toilets have been installed in rest areas in Colorado, California, New Mexico, and Texas. Water saving toilets often are installed to reduce hydraulic loadings to failing wastewater treatment systems as well as to conserve water. The use of water saving toilets will increase the concentration of all parameters in the wastewater.

3.11.2 Microphor Toilets

Microphor "Microflush " toilets ideally use two quarts of water for flushing purposes as opposed to the 4 to 6 gallons used in conventional toilets. The toilets can use less water because the flushing action is assisted by pressurized air. Ideal water and air pressures to maintain a system for the toilets are 20 to 60 and 60 to 65 psi, respectively. Microphor toilets have been installed at numerous rest areas and some preliminary results have been noted.

3.11.3 Low Flush Toilet Effects on Treatment Systems

Hutter [36] found that the use of low flush toilets at the Deer Trail Rest Area in Colorado resulted in a wastewater production rate of 1.26 gallons per rest room user. The treatment system at the rest area consisted of an extended aeration package plant and a settling pond. After low flush toilets were installed the package plant received low flows and congestion of pipes and odors in the plant resulted. The problems were only temporary, with the odors disappearing after bacteria populations increased and congestion problems ending after the toilets were set to use more than two quarts of flush water.

Several rest areas in California also are using the Microphor toilets. Experience in California is that 0.9 gallon per flush is necessary for acceptable performance of the system

[34]. Pearson's method, presented in Appendix I, also can be used to calculate tank pumpout intervals for septic tanks which receive water saving toilet wastewater. Pearson has suggested the following pumpout schedule for septic tanks that receive wastes from water saving toilets:

Pumpout interval, months	Septic tank detention (days) for given restroom waste flow in gallons/veh		
	5.5	2	1
6	1.5	1.5	1.5
12	1.5	1.5	2.5
60	1.5	2.5	4.9

3.11.4 Operation and Maintenance

Presently fourteen pairs of rest areas in Texas have had Microphor toilets installed. Eight pairs of rest areas have installation underway, and nine pairs are planned to receive the toilets in the future. The Texas Department of Highways and Public Transportation identified one major problem in using the water saving toilets: the units often are installed incorrectly and do not function properly. Therefore, it is suggested that the contractors who install the toilets be closely monitored by proper state personnel.

The main maintenance problem with the toilets is that if water gets into the air pressure system it can cause corrosion of various components of the system [37]. Another possible problem is the need to maintain constant water pressure (+3 pounds psi) for the proper functioning of the toilet. Other maintenance includes component lubrication and adjustment of bowl flush water flow.

CHAPTER 4. TREATMENT SYSTEM COSTS

4.1 INTRODUCTION

Costs of water and wastewater systems should include capital, operation, and maintenance costs. In order to compare costs of various systems the annual costs of the systems over the lifetime of the system should be computed and compared. Although systems with low initial capital costs may look attractive at first, operation and maintenance costs must also be included in costs comparisons in order to determine long term costs. This chapter deals exclusively with wastewater treatment system costs as reported in the literature; water system costs are not covered because water system costs are less variable.

4.2 ANNUAL COSTS

The annual costs of a treatment system can be calculated from the following formula [38]:

$$AC = Ci + O\&M + i(C - S)/[(1 + i)^n - 1] + CRi/[(1 + i)^x - 1] \quad \text{(Eq. 4.1)}$$

where

AC	=	Annual cost of system
C	=	Initial capital cost of system
O&M	=	Annual operation andmaintenance cost of system
S	=	Salvage price of system after n years of use
CR	=	Cost of major repairs after x years of operation
n	=	Design life of system
i	=	Fixed interest rate

The third term on the right hand side of Eq 4.1 is a depreciation term while the last term represents costs of major repairs to the system. The salvage price, S, is often zero and a value of zero will result in a higher annual cost. The interest rate, i, is assumed to be fixed over time.

In most applications of annual cost equations the cost of major repairs, CR, is not addressed and the salvage cost is assumed to be zero. The major repair costs may be easier to predict for septic tank/ drainfields or package plant waste treatment systems because they have been used at rest areas frequently, but a literature search did not find appropriate repair costs to use in Eq 4.1. At rest areas it may be prudent to use the CR term to account for expansion of waste treatment systems. As an example, if a rest area uses a modular package plant the CR cost may account for the purchase of an additional aeration tank after x years, or if a pond system an additional pond after x years.

4.3 COST INDEXING

Cost indexing is recommended when comparing costs from various engineering reports. Construction cost indexes for municipal treatment plants are reported in the *Engineering News Record (ENR)* construction cost index and in the *EPA Sewer Construction and Sewage Treatment Plant* construction cost index. Metcalf & Eddy [2] recommend the use of the following equations for cost indexing:

Current Cost = (current value of index/value of index at time of report) X cost cited in report (Eq. 4.2)

Future Cost = (projected future value of index/current value of index) X current cost (Eq. 4.3)

Future cost indexing should be done every 3 to 5 years because of variability of the projected indexes.

Most available indexes do not apply to systems used at rest areas but apply to municipal plants. An EPA manual entitled, "Innovative and Alternative Technology Assessment Manual" [39], contains cost estimates for evaporative ponds, evapotranspiration beds, and septic tanks but these estimates are based on 1976 labor and land costs. The manual also contains cost curves for extended aeration plants, facultative lagoons, and irrigation systems, but only the package plant cost curves cover flows typical of rest areas. The construction costs presented in the manual must be converted to capital costs in 1986 dollars via ENR or EPA cost indexes (if the indexes are available).

Cost indexing is difficult to apply to rest area wastewater treatment systems. Cost indexes themselves are highly variable for different geographic locations and will probably not be available for many rest area sites. Cost indexes and cost curves are difficult to obtain for small wastewater systems and the accuracy of extrapolating existing cost data for low flow ranges is not known. Thus, local contractors and suppliers are the best sources for accurate and current capital cost figures.

4.4 REST AREA WASTEWATER TREATMENT RELATIVE COST COMPARISONS

Cost data for rest area wastewater treatment systems are scarce. The cost data that are available usually deal with capital costs and ignore operation and maintenance costs. However, some data have been reported for both types of costs.

Sylvester and Seabloom [8] reviewed capital and operating costs for a 500-man military camp (average wastewater flow = 36,500 gal/day). The data showed that an oxidation pond was the least expensive system at the camp. Other system costs relative to the oxidation pond system are presented in Table 4.1. An aerated lagoon was the next most cost effective system. Pfeffer [19] compared 1973 U.S. EPA installation cost figures for lagoon versus package plant systems and reported that package plants were roughly twice as costly to install as lagoons.

Strong compared the Monogram Oil Recirculating System to an extended aeration package plant (EAPP) and found the former to be more cost effective [30]. Operation and installation costs for these systems are shown in Tables 4.2 and 4.3 [30]. Strong found that 80 percent of the operating costs for the EAPP were labor costs while the Monogram system labor cost was 15 percent of the total operating cost [30]. The operating cost for the EAPP was three times that for the Monogram system. Installation costs (Table 4.3) were similar for the two systems and are not useful in comparing costs of the system.

Parker [31] compared a water recycle-EAPP with an Aqua Sans Mineral Oil System in 1977. Using a 20-year design life and an 8 percent discount rate he found the Aqua Sans system to be roughly 1.25 times more expensive than the water recycle-EAPP system. The 1977 dollar savings for a 10,000 and 20,000 gpd plant were ~$53,000 and ~$114,000 dollars, respectively. These values are not too meaningful because they are difficult to index to present costs.

In 1982 Erickson [20] compared the additional costs that land treatment systems added to existing lagoon treatment systems. He suggested that spray irrigation (barriered landscape water renovation system) was approximately four times more expensive than the overland flow - evapotranspiration system (OF-ET). He also found that seepage beds were equal to OF-ET systems in terms of added costs to existing lagoon treatment. Rodman [27] found that impact rotary sprayers were approximately one-half as expensive as pop-up sprinklers.

TABLE 4.1. TREATMENT SYSTEM COSTS RELATIVE TO OXIDATION POND FOR A 500- PERSON MILITARY CAMP (SYLVESTER AND SEABLOOM, 1972)

Type of System	Capital Costs	Operating Costs	Total
Extended Aeration Package Plant	6.88	1.55	2.64
Oxidation Ditch	3.75	1.29	1.76
Bio-disc	11.25	.61	2.75
Aerated Lagoon	1.63	1.38	1.44
Trickling Filter	6.63	2.48	3.33

4.5 SUMMARY

Annual water and wastewater treatment costs should include both capital and operation costs. Cost indexing, although desirable, is a difficult task for rest area treatment systems because cost indexes are not available for small systems, with the exception of extended aeration package plants. Relative cost comparisons from two studies shows that package plants are approximately twice as expensive as pond systems. Relative cost comparisons for package plants versus mineral oil recirculating are inconclusive so that the choice between using either of these systems depends more on other considerations. Based on one study it appears that overland flow-evapotranspiration systems may be less expensive than spray irrigation. The best way to estimate costs for rest area wastewater treatment systems is to consult local contractors and suppliers for capital cost estimates and to review operating and maintenance records at existing rest areas to estimate operation and maintenance costs for proposed rest area wastewater treatment systems.

TABLE 4.2. OPERATIONS COST SUMMARY FOR YEARS 1980 AND 1989 (JULY 1, 1979, TO JUNE 30, 1980)

Cost Item	10,000-GPD Monogram Plant	6500-GPD Extended Aeration Plant	15,000-GPDExtended Aeration Plant
Utilities (Electrical & Water)	$ 793.80	$ 531.61	$ 4,510.58
Filters & Plant Materials Plumbing & Electrical	2,805.71	137.56	1,016.55
Supplies & Fixtures	1,089.60	288.00	1,583.31
Labor	2,022.68	8,193.31	34,788.00
Recirculating Oil	6,112.95	N. A.	N. A.
Pumpouts	785.00	N. A.	N. A.
TOTAL	$13,609.74	$9,150.48	$41,898.44

TABLE 4.3. COMPARATIVE COST SUMMARY (RECIRCULATING OIL VERSUS EXTENDED AERATION)

Monogram Magic Flush Recirculating Oil Systemand Auxiliary Gray Water System (Project 8.1217113 - 1978)		
Monogram Plant		$41,200
Domestic Waste Disposal Package and 6-inch Ductile Iron Connections		16,210
Gray Water Waste Disposal System and 4-inch Ductile Iron Connections		13,110
Brick Masonry Pier and 4 foot Manhole		3,650
Access Road to Black Waste Tank		3,200
	Total	$77,370
Conventional Extended Aeration-Activated Sludge TreatmentSystem - 10,000-Gallons Per Day Capacity (Quotation for Project 8.1217113 - 1978)		
Secondary Sewage Treatment Plant & and 6-inch Ductile Iron Connections		$33,150
Chlorination Sewage Treatment Package		25,457
4-inch P.V.C. Discharge Pipe		17,045
Brick Masonry Pier, 4-foot Manholes and 8-inch Wall Manhole		7,100
Metal Posts and Gate Enclosure		6,192
	Total	$88,944
Conventional Extended Aeration-Activated Sludge Treatment System -6500-Gallons Per Day Capacity (Project 6.803175 - 1971)		
Secondary Sewage Treatment Plant and 6-inch Ductile Iron Connections		$14,720
Chlorination Sewage Treatment Packate		7,000
4-inch & 6-inch Cast Iron Connections and Cleanouts		6,469
Brick Masonry Pier and Manhole		999
Metal Posts and Gate Enclosure		877
	Total	$30,065
Conventional Extended Aeration-Activated Sludge Treatment System - 15,000 - Gallons Per Day Capacity (Project 8.1760206 - 1976)		
Secondary Sewage Treatment Plant and Cast Iron Sewer Pipes		$68,780
Tertiary Sewage Treatment Plant		62,700
Chlorination Sewage Treatment Package		23,100
4-inch and 6-inch PVC Service Pipe and Connections		5,074
8-inch Wall and 4 foot Manhole		1,490
Metal Posts and Gate Enclosure		4,291
	Total	$165,435

CHAPTER 5. CONCLUSIONS AND RECOMMENDATIONS

This report evaluates state-of-the-art water and wastewater systems at highway rest areas. Conclusions and recommendations for rest area water and wastewater systems drawn from this report follow.

WATER TREATMENT SYSTEMS

1. Water usage data for Texas highway rest areas are nonexistent. Estimates of water usage can be calculated and used for design if the following information is available: average daily highway traffic (ADT), percentage of ADT stopping at the rest area, percentage of those entering the rest area that use the rest rooms, and water use per person or per vehicle.
2. Water meters can provide valuable flow records for design and modification of future and existing water and wastewater systems at rest areas. Water meters should be installed in all new and existing rest areas. Meters should be installed in locations that permit measurement of rest room water use separately from outside water uses, such as lawn sprinkling. Water meters should be calibrated for a particular flow range and one meter should be installed on the inlet side of the hydropneumatic tank for more accurate flow measurements. Meters should be sealed and should be able to measure instantaneous flow as well as the total accumulated flow volume.
3. The design of water supply systems at rest areas requires knowledge of peak instantaneous, peak hourly, and peak daily water demands. Water requirements can be calculated using flow data at the rest area site or from an analogous site. If flow data are not available, indirect methods can be employed. Hunter's fixture method (Ch. 2) should be used to calculate peak instantaneous demands at rest areas. Equation 2.1 should be used to calculate peak hourly water demands for existing rest areas while the Zaltzman method (Appendix A) should be used for proposed rest areas. The peak daily demand should be determined from the Zaltzman method.
4. Pumps for hydropneumatic tanks should be sized based on the peak hourly water demand. However, peak instantaneous demand should be used to size pumps when a rest area is expected to experience frequent peak instantaneous demands (i.e., rest areas with high commuter traffic). Two pumps should be installed for alternate use, and for pump servicing, to provide continuous water supply.
5. Water storage facilities are required if the pump on the well cannot supply the maximum hourly (or instantaneous) water demand.
6. The total number of fixtures required at a rest area can be calculated using Eq 2.4, and distribution of toilets and urinals can be determined by the method presented in Appendix B.
7. When water saving toilets are installed at a rest area one gallon per flush (4-second flush time) should be used to estimate fixture water demands.
8. Water use for recreational vehicles will be roughly 9 gallons per vehicle.
9. Spigots to lavatories should be spring loaded for automatic shutoff.
10. Municipal water supplies should be the first choice for rest area water supplies. These sources are usually of high quality, require minimal maintenance, and do not need storage facilities.
11. If well water, surface water, or storage tanks are used, disinfection with chlorine is required. The choice of chlorination systems depends on past experience, maintenance records, and operation costs.
12. Softening is required for waters with a hardness of 300 mg/l or greater. Ion- exchange (Zeolite) is the preferred treatment method.

WASTEWATER TREATMENT SYSTEMS

1. Geologic and soil studies should be conducted at the proposed rest area site in addition to percolation tests. These studies will help determine the probable success of soil absorption, pond, land treatment, or evapotranspiration bed systems at rest areas. The geologic and soil studies should be a part of the rest area site feasibility study in order to combine waste treatment concerns with more routine highway and building construction concerns.
2. Rest area wastewater is similar to domestic wastewater but has higher carbonaceous oxygen demand and higher ammonium concentrations. Rest area wastewaters should not present any major treatment difficulties.
3. For proposed rest areas, evaporative ponds should be considered first. Evaporative ponds will require roughly three acres of land and should have an annual excess of evaporation over rainfall of 10 inches. Overflow ponds can be used where evaporation ponds are unsuitable; a minimum of three ponds should be used. Clay or synthetic pond liners are required by federal law and must be installed. West Texas rest areas are prime candidates for evaporative ponds whereas overflow ponds are probably more suitable in east Texas (but evaporative ponds should be evaluated first).
4. The use of overland flow or evapotranspiration beds for final disposal of septic tank effluent should be considered where drainfields have failed. Seasonal use of these systems during rest area high use periods is an option that should be fully explored. Study results have shown that four to six acres of land will be necessary for overland flow systems and that ET bed systems will require one-half (Texas guidelines) to three (by

Rugen's successive approximation method) acres of land.

5. Extended aeration plants are more desirable where less than three acres of land are available for the rest area wastewater treatment system. Higher maintenance and operating costs should be expected with package plants.
6. Spray irrigation is likely to require more land than overflow land treatment systems because buffer strips of trees or bushes are required for spray irrigation. Spray irrigation of package plant or lagoon effluent will require two to ten acres of land, depending on the rest area soil characteristics. Downward pointing spray heads should be used to reduce aerosols.
7. Water saving toilets or water recycle systems should be considered if water shortages or high wastewater flows are a concern at the rest area. Water saving toilets are low cost systems that may solve water shortage problems with a minimum of change to the wastewater treatment system. Water recycle systems are favored if low flush toilets are already installed or are not desirable based on local experiences with water saving toilets. Mineral oil systems are not recommended because they require a separate grey water system and have high overall costs and numerous maintenance problems.
8. Recreational vehicle (RV) dump stations are not recommended at rest areas because they necessitate changes in the operation of the waste water treatment system and they are prone to vandalism. Wastewater production from RVs will be approximately 16 to 21 gallons per vehicle. If RV stations are to be installed at rest areas, plans should be made for diluting RV wastes before they enter pond or package plant systems. If land adsorption systems are used at the rest area, a separate RV wastewater treatment system should be used. RV wastes and water saving toilet wastes are similar in that both will increase the organic concentrations of wastewaters flowing to the treatment system. Therefore, design of these systems must take the increased concentration of wastewater constituents into account.

RECOMMENDATIONS FOR INDIVIDUAL WASTEWATER SYSTEMS

Septic Tank/Drainfield Systems

1. Septic tanks should be sized based on a tradeoff between using peak daily flows at a higher tank cost and using the USPHS or similar formula at a lower tank cost. Traffic use data are important in determining the frequency of peak flows. The tank should have a riser to the ground surface for scum and sludge measuring and for pumping purposes.
2. Geologic and soil surveys should be performed at the rest area site, in addition to percolation tests for design purposes. The ability of a particular soil to treat septic tank effluent depends on the infiltration capacity and the chemical and physical properties of the soil.
3. Drainfield infiltration surface area must be calculated using the *peak* daily wastewater flowrate.
4. Alternate dosing and resting periods should be used in applying effluent to the drainfield. Resting times need to be determined locally for the site using soil moisture tests over various times of resting. It can be expected that sands will require about a month of rest and silts and clays several months or more.
5. Percolation rates should be correlated to unsaturated hydraulic conductivity (k) via Bouma's crust test and this conductivity should be used in determining drainfield surface areas. Laak's LTAR rates (Table 3.5) could be used in place of the crust test if Bouma's test is too expensive.
6. Narrow trenches are recommended for the drainfield system; bottom area and sidewall area can be used to design the field if the trenches are set up in series. If trenches are in parallel, bottom area alone should be used for design.
7. Temporary rehabilitation of drainfields can be accomplished by using hydrogen peroxide. A long term solution to the problem depends on determining whether the problem is due to volumetric or time loading.

Pond Systems

1. Evaporative ponds should be used at rest areas where the annual evaporation rate exceeds the annual rainfall rate and there are three or more acres of land available for the ponds. West Texas rest areas are prime candidates for evaporative ponds.
2. Three or more ponds in series should be utilized, especially for overflow ponds producing an effluent.
3. Evaporative pond surface areas should be based on excess yearly evaporation rates as related to inflows (see Appendix F). BOD loading rates can be used for designing overflow ponds.
4. Pond depths should be between 2 and 6 feet with 3 to 4 being standard.
5. Consideration should be given to using ponds to help failed septic systems during high rest area use periods of the year.
6. Aerobic ponds are not recommended for use at rest areas because of higher maintenance and operation costs.
7. Pond liners should be used in all pond systems to reduce pond seepage and meet federal and state regulations.

Extended Aeration Package Plant Systems

1. Extended aeration package plants should be considered for use if

   (1) less than three acres of land are available at the rest area,

   (2) groundwater contamination is likely if pond or septic tank/drainfield systems are used, or

(3) discharge to a stream is the only means of final effluent disposal.
2. Modular package plants are recommended for use at rest areas so that design capacity is more closely achieved throughout the life of the treatment plant. Sufficient space should be provided at the site for addition of modular units as needed.
3. Consideration should be given to using intermittent activated sludge treatment systems at rest areas. Intermittent systems offer greater flexibility and are more adaptable to changing loading conditions.
4. Sludge wasting from the EAPP via drying beds, small ponds, holding tanks, or truck hauling is a necessary element of design.
5. BOD tests should be carefully used in ascertaining plant performance because of the presence of nitrifiers in the aeration tank. The oxygen demand due to nitrifiers must be accounted for in analysis.
6. Recommended design criteria for an EAPP are given in Table 3.10 [13].

Land Treatment Systems

1. Land requirements for land treatment systems should be determined from a comparison of hydraulic and nitrogen loadings, with the larger land area requirement being chosen.
2. Wastewater application rates should be determined from water balances made from climatic monthly summaries. Information needs and sources of information for designing a land treatment system are shown in Table 3.15.
3. High nitrogen uptake crops should be planted in spray or overland flow-evapotranspiration fields. Canary grass and other plants should be investigated to find maximum plant uptake rates.
4. Spray irrigation systems should have fixed distribution systems and have buffer strip areas to both hide the system and trap aerosols.
5. Overland flow systems can be used on impermeable soils, such as clays and clayey loams. Overland flow-evapotranspiration fields can be added to failed septic tank/drainfield systems and operated during summertime high flows to give the drainfield a chance to aerate.
6. Land requirements for spray irrigation and overland flow range from 2 to 10 acres.

Evapotranspiration Bed Systems

1. Evapotranspiration beds can be used to rehabilitate failed septic tank/drainfield systems. It appears that ET beds can accept septic tank effluent without any problems and, if a liner is used, not be a threat to groundwater.
2. Grasses should be used as ET bed cover crops.
3. Effluent levels in ET beds should be higher than 10 inches below the ground surface.
4. Rugen's method for calculating bed area should be followed until a better formula or method is found.
5. Additional studies on ET beds need to be carried out to determine treatment ef-ficiencies and the danger ET beds pose to groundwaters.

Reuse-Recycle Systems

1. The use of the water reuse system is favored over the mineral oil system. Water reuse systems have fewer operational and maintenance costs than mineral oil systems and do not require a separate grey water system.
2. Reuse/recycle systems should be used only if water conservation or zero discharge requirements must be met.

Recreational Vehicle Dump Stations

1. Recreational vehicle dump stations should not be built at rest areas unless it is absolutely necessary. The dump stations are likely to be vandalized and are likely to require safety measures for the wastewater treatment systems.
2. If built, RV dump stations should be connected to a holding tank so that the wastes can be diluted by rest area wastewater before treatment by package plant, pond, or land treatment systems.
3. Separate RV waste treatment systems should be provided for rest areas which utilize septic tank/drainfield or recycle/reuse waste treatment systems.
4. Additional data are needed on RV traffic flows (use of rest area and dump facility), wastewater production, and seasonal dump station usage.
5. The method presented in Appendix I can be used to determine septic tank and drainfield sizes and to determine septic tank pumpout schedules.
6. Service fees should be charged to RV owners to pay for RV dump stations.

Low Flush Toilets

1. Low flush toilets reduce wastewater flows but increase the concentration of constituents of wastewater.
2. Existing treatment system operation will require adjustments when low flush toilets are used. For example, septic tanks will need to be pumped out more often if low flush toilets are used at the rest area.
3. Maintenance problems with low flush toilets in Texas rest areas are associated mostly with improper installation of toilets and water condensing in the air pressure system. It is recommended that state personnel oversee the contractor to ensure proper installation of the toilets.

COSTS OF TREATMENT SYSTEMS

1. Annual costs should be determined using Eq 4.1.
2. Cost indexing is advised for all costs if possible but is probably possible only for extended aeration package plants.
3. In most cases costs can be estimated using local information from contractors and suppliers and using operating expenditures at existing rest areas.

REFERENCES

[1] Folks, Nancy, "Manual for Safety Rest Area Water Supply Systems," FHWA-RD-77-113, September 1977.

[2] Metcalf & Eddy, Inc., *Wastewater Engineering: Treatment, Disposal, Reuse*, McGraw-Hill Series in Water Resources and Environmental Engineering, McGraw-Hill Book Company, New York, NY, 1979.

[3] Hunter, Roy, "Methods of Estimating Loads in Plumbing Systems," Building Materials and Structures, Report BMS65, National Bureau of Standards, U.S. Department of Commerce, December 1940.

[4] Johnson, R. L., "Water Systems for Interstate Safety Rest Areas: Quantity and Quality Aspects," Presented at 48th Annual Meeting of the Committee on Roadside Development, Highway Research Record #280, pp 39-52, 1969.

[5] "Rest Area Design Charts," Federal Highway Administration, U.S. Department of Transportation, FHWA Technical Advisory Report No. T 5140.8, August 10, 1979.

[6] Zaltzman, R., et al, "Establishment of Roadside Rest Area Water Supply, Waste Water Carriage, and Solid Waste Disposal Requirements," unpublished final report, prepared for Federal Highway Administration by West Virginia University, April 1975.

[7] Conversation with Bill Wilson, Architectural Engineer, Texas State Department of Highways and Public Transportation, May 30, 1986.

[8] Sylvester, Robert, and Seabloom, Robert, "Rest Area Wastewater Disposal," report prepared for the Washington State Highway Commission, Department of Highways, by the Department of Civil Engineering, University of Washington, Seattle, WA, January 1972.

[9] Cantor, Larry W., and Knox, Robert C., *Septic Tank System Effect on Ground-water Quality*, Lewis Publishers Inc., 121 S. Main Street, Chelsea, MI, January 1985.

[10] Kreissel, James F, "Management of Small Waste Flows," U.S. Environmental Protection Agency Report No. 600/2-78-173, September 1978.

[11] "Manual of Septic Tank Practices," U.S. Department of Health, Education, and Welfare, U.S Public Health Service, Publication No. 526, 1967.

[12] "Construction Standards for Private Sewage Facilities," Texas Department of Health, 1100 West 49th St., Austin, TX 78756, November 1977.

[13] Hughes, Gregory, and Averett, Daniel E., "Wastewater Treatment Systems for Safety Rest Areas," Final Report for the Federal Highway Administration, Report No. FHWA-RD-77-107, September 1977.

[14] Winneberger, John H. Timothy, *Septic Tank Systems, A Consultant Tool Kit, Vol II, The Septic Tank*, Butterworth Publishers, Boston, 1984.

[15] Winneberger, John H. Timothy, *Septic Tank Systems, A Consultant Tool Kit, Vol I, Subsurface Disposal of Septic Tank Effluent*, Butterworth Publishers, Boston, 1984.

[16] Ryan, Henry, "Notes on the Design of Sewage Disposal Works, with Special Reference to Small Installations," Private Publication, New York, 1928.

[17] Frederick, Joseph C., "Soil Percolation Rates," *Public Works*, 83(7) : 46-48, 89-90, 1952.

[18] Bouma, J., et al., "Field Measurement of Unsaturated Hydraulic Conductivity By Infiltration Through Artificial Crusts," *Soil Science of America Proceedings*, Vol. 35, pp 362-364, 1971.

[19] Pfeffer, John T., "Rest Area Wastewater Treatment and Disposal," Report prepared for the Federal Highway Administration by the Department of Civil Engineering, University of Illinois, Urbana-Champaign, November 1974.

[20] Erickson, A. E, et al, "Sewage Treatment Systems at Freeway Rest Areas, Final Report," FHWA-MI-RD-81, Michigan State University, East Lansing, December 1982.

[21] Jenkins, Stephen R., et al, "Transpiration and Evaporation of Sewage Effluent," FHWA/AL-76/81, Engineering Experimental Station, Auburn University, Alabama, December 1976.

[22] Tom Straughan, visits with transportation departments of the states of Louisiana, Oregon, Washington, and California, Dec-March 1986, sponsored by Department of Civil Engineering, The University of Texas at Austin.

[23] Palaez, Harry Mathew, "Wastewater Characteristics and Wastewater Treatment Systems at Texas State Parks," Masters Thesis, Department of Civil Engineering, The University of Texas at Austin, December 1979.

[24] Goronszy, M.C., "Intermittent Operation of the Extended Aeration Process for Small Systems," JWPCF, Vol. 51, No. 2, pp 274-287, February 1978.

[25] Pfeffer, J. T., "Design Criteria for Extended Aeration," *Kansas University Bulletin of Engineering and Architecture*, No. 51, 1963, pp 15-22.

[26] Mahloch, J. L., Averett, D. E., and Headsteam, M., "Laboratory and Pilot Plant Evaluation of Intermittent Loadings on Small Extended Aeration Biological Systems," Environmental Effects Laboratory, U.S. Army Engineer Waterway Experimental Station, Vicksburg, MS, September 1977.

[27] Rodman, Roy S., "Rest Area Sewage Disposal System," American Association of State Highway Transportation Officials (AASHTO) Proceedings for November 18-20, 1974, meeting.

[28] Erickson, A. E., et al, "On-site Land Treatment Systems for Freeway Rest Areas," Transportation Research Record 822, Texas State Department of Highways and Public Transportation, Austin, Texas, 1982.

[29] Rugen, Mark A., et al, *Evapotranspiration, A Method of Disposing of Septic Tank Effluent*, report prepared for the Edwards Underground Water District by Raba & Assoc., San Antonio, TX, November 1977.

[30] Strong, M. P., "Evaluation of MONOGRAM Recirculating Oil Sewage Disposal System," Final Report, Study Project 170-5(11), Planning & Research Branch, Div. of Highways, North Carolina Department of Transportation, March 1982.

[31] Parker, Clinton E., "Water Reuse at Highway Rest Areas: Evaluation Phase," FHWA/VA-78/R22, Virginia Highway and Transportation Council, Charlottesville, VA, December 1977.

[32] Parker, Clinton E., "Water Use at Highway Rest Areas: Follow-up of Implementation," FHWA/VA-79/R43, Virginia Highway and Transportation Council, Charlottesville, VA, April 1979.

[33] Pearson, Frank, et al, "Recreational Vehicles Disposal in Roadside Rest Septic Tank Systems," FHWA/CA/UC-80/01, prepared by the Sanitary Engineering Research Laboratory, University of California, Berkley, June 1980.

[34] Kiernan, K., et al, "Recreational Vehicle Stations at Highway Rest Areas (Final Report)," FHWA/WA-60.1, Department of Civil Engineering, University of Washington, Seattle, WA, June 1983.

[35] Grottkau, William A., et al, "A Design Method for Roadside Rest Area Septic Tank Leach Field Systems," FHWA/CA-81/13, California State Department of Transportation, Sacramento, CA, July 1981.

[36] Hutter, Werner, "Low Flush Toilets at Deer Trail Rest Area," FHWA-CO-79-10, prepared by the Colorado State Department of Highways, Denver, CO, October 1977.

[37] Microphor maintenance conference, sponsored by the Texas State Department of Highways and Public Transportation, San Antonio, TX, May 29,1986.

[38] Ewing, Ben, Professor emeritus of Civil Engineering and Institute for Environmental Studies (BSCE 1944, MSCE 1949, The University of Texas at Austin; Ph.D. 1959, University of California at Berkley), University of Illinois at Champaign Urbana, lecture notes from CE 342, Water Pollution, The University of Texas at Austin, Spring 1986.

[39] "Innovative and Alternative Technology Assessment Manual," EPA 430/9-78-009, Office of Research and Development (MERL), Cincinnati, OH, February 1980.

[40] Laak, R., *Wastewater Engineering Design for Unsewered Areas* (Ann Arbor, MI: Ann Arbor Science Publishers, Inc., 1980).

[41] Winneberger, J. T., "Correlation of Three Techniques for Determining Soil Permeability," *Journal of Environmental Health* 37(2): 108-118 (1974).

[42] "Process Design Manual for Land Treatment of Municipal Wastewater," EPA 625/1-81-013, Center for Environmental Research Information, Cincinnati, OH, October 1981.

APPENDIX A. CALCULATION OF DEMANDS BY THE ZALTZMAN METHOD [6]

The Zaltzman approach attempts to avoid estimating daily peaking factors used to determine rest area peak water demands and wastewater production. The method is predictive and is necessary only if flow data are not available or cannot be monitored at an existing analogous rest area.

1. Highway traffic data must be collected for a full year at the roadway site where the rest area will be located. The six peak three-day weekend data are then selected (a three-day weekend consists of Friday, Saturday, and Sunday). The six peak three-day weekend highway traffic data (daily traffic) are then averaged to obtain a design daily traffic. Example calculations using the Zaltzman method follow.
2. The following traffic data were collected for the six peak three-day weekends of the year :

Friday	Saturday	Sunday	Totals
11,364	13,426	12,978	37,768
11,027	13,142	13,264	37,433
10,642	12,976	12,718	36,336
9,267	13,179	12,653	35,099
10,117	12,349	12,144	34,610
9,870	11,957	11,643	33,470
			214,716

3. Calculate the design daily traffic (HIWAY 24) for the six peak three-day weekends.
   HIWAY 24 = 214,716 vehicles/18 days
   HIWAY 24 = 11,929 veh/day
4. Calculate the design average 24-hr rest area traffic (REST)
   REST = 0.09 x HIWAY 24
   REST = 0.09 x 11,929 veh/day
   REST = 1074 veh/day
   Note: The 9 percent figure is the percentage of mainline traffic stopping at the rest area and is a figure Zaltzman obtained from the data he collected.
5. Calculate the design average 8-hr rest area traffic (REST 8). This value is the average rest area traffic between 8:00 am and 4:00 pm and is used to calculate WATER 8.
   REST 8 = 0.67 x REST 24
   REST 8 = 0.67 x 1074 veh/day
   REST 8 = 720 veh/day
   Note: The 0.67 value is the percentage of daily traffic that occurs during the 8-hour period stipulated above according to Zaltzman's data.
6. Calculate the peak 1-hr rest area traffic (PK VOL 1).
   PK VOL 1 = 0.15 x REST 24
   PK VOL 1 = 0.15 x 1074 veh/day
   PK VOL 1 = 161 veh/day
   Note: The value of 0.15 is the ratio of peak hourly to daily traffic.
7. Calculate the daily water requirement (WATER 24). This value is the peak daily demand and is the amount of water that must be available continuously at the rest area.
   WATER 24 = 6.7 gal/veh x REST 24
   WATER 24 = 6.7 gal/veh x 1074 veh/day
   WATER 24 = 7196 gal/day
8. Calculate the 8-hr water demand (WATER 8).
   WATER 8 = 0.67 x WATER 24
   WATER 8 = 0.67 x 7196 gal/day
   WATER 8 = 4822 gal/8 hr
9. Calculate the peak hourly water demand (WATER 1).
   WATER 1 = 6.7 gal/veh x PK VOL 1
   WATER 1 = 6.7 gal/veh x 161 veh/hr
   WATER 1 = 1079 gal/hr
   This is the amount of water needed to meet the peak hourly demand at the rest area.
10. Calculate the design wastewater production (WASTE 24).
   WASTE 24 = 5.5 gal/veh x REST 24
   WASTE 24 = 5.5 gal/veh x 1074 veh/day
   WASTE 24 = 5907 gal/day
   This is the peak daily wastewater production and can be used in designing the wastewater treatment facilities. For clarity and simplicity this figure will be rounded to 6000; this also gives a more conservative figure for design.
11. Now BOD_5 and suspended solids (SS) can be calculated. It is assumed that a value of 165 mg/l and 190 mg/l for each constituent is representative of rest area waste.
   BOD_5(lbs/day) = WASTE 24 x BOD_5 x 8.34 x 10^6
   BOD_5 = 6000 gal/day x 165 mg/l x 8.34 x 10^{-6}
   BOD_5 = 8.26 lb/day
12. Calculate the suspended solids (SS) loading in lb/day.
   SS (lbs/day) = WASTE 24 x SS x 8.34 x 10^{-6}
   SS = 6000 gal/day x 190 mg/1 x 10^{-6}
   SS = 9.5 lb/day

The engineer is now able to determine the water demands at the proposed rest area and design supply and treatment systems accordingly. The values for percent average daily traffic stopping, peak hourly/daily traffic, gal/veh water used and wastewater produced, and the percentage 8-hr stopping to daily stopping ratio will probably vary for different areas of Texas and if possible should be determined. If data are not available for the proposed rest area region Zaltzman's values can be used. Many of the rest area rest rooms in Texas will have Microphor toilets so that the values used in this example will grossly overestimate water use; a more realistic value for water use per vehicle would be 2.5 gallons.

The values assumed for organic (BOD_5) loading and suspended solids (SS) loading are acceptable values determined by several studies done at rest areas (Sylvester and Seabloom, Pfeiffer, Etzel, and others) and should be used. If recreation vehicle dumping stations are provided these values are not acceptable for use and should be determined for the waste in question.

APPENDIX B. DISTRIBUTION OF TOILETS AND URINALS IN REST ROOMS [5]

Assumptions (from usage data).

1. 60% of rest room users are male
2. 40% of rest room users are female
3. 67% of men use urinals
4. 67% of women urinate in rest rooms
5. 1 minute cycle time for men's urinal
6. 2 minute cycle time for women's urinal
7. * 3.23 minute cycle time for defecation
   * This is an adjusted value to yield an average cycle time of 2 minutes/fixture.
8. Users per hour per activity

Men's urination: 60 per hour per fixture (60/1 = 60)
Women's urination: 30 per hour per fixture (60 / 2 = 30)
Defecation: 18.57 per hour per fixture (60/3.23 = 18.57)
Average user: 30 per hour per fixture (60/2 = 30)

Design example: 10 total water closets & urinals

Total users per hour =	10 x 30 = 300	
Men's urination -	.6 x 2/3 x 300 =	120 per hour
Women's urination -	.4 x 2/3 x 300 =	80 per hour
Women's defecation -	.4 x 1/3 x 300 =	40 per hour
Men's defecation -	.6 x 1/3 x 300 =	60 per hour
Total		300 per hour

Toilets required in women's rest rooms

$$\frac{80}{30} = 2.67 \text{ toilets}$$

$$\frac{40}{18.57} = 2.15 \text{ toilets}$$

$$2.67 + 2.15 = 4.85 \text{ toilets}$$

$\therefore$ **Use 5.**

Toilets required in men's rest room

$$\frac{60}{18.57} = 3.23 \text{ toilets}$$

$\therefore$ **Use 3.**

Urinals required in men's rest room

$$\frac{120}{60} = 2 \text{ urinals}$$

$\therefore$ **Use 2.**

Results

48.2% of total are women's toilets
32.3% are men's toilets
20% are urinals

These values were used to distribute toilets and urinals in the rest room.

APPENDIX C. TECHNIQUES FOR BEST PERCOLATION TEST RESULTS AND ASSESSMENT OF SOIL PERMEABILITY [15] (ADAPTED FROM WINNEBERGER)

Always remove the upper soil layers of no concern to expose the soil to be tested. The test hole is to be at the bottom of a hole large enough for a technician to work within. That is most easily accomplished with a backhoe. The percolation test hole is to be hand dug to about a couple of inches deeper than water fillings of the test are to be.

DIGGING THE TEST HOLE

At the bottom of a larger hole, dig a test hole smaller in diameter than the finished test hole is to be. A bucket auger 3-1/4 in. in diameter serves quite well for test holes that are to be 4 in. or so in diameter. A posthole digger will serve if the hole is to be 12 in. in diameter. Take care when digging the smaller hole not to crush the soil sidewalls.

Insert the blade of a 2-in. rigid putty knife into the top side of the hole opposite you, holding the blade with its cutting edge vertical. Pull the blade away so as to break loose a chunk of soil. Next, insert the vertical blade into the soil at a place perhaps an inch away from the place where the chunk was just removed. Break loose another chunk of soil. Continue working around the hole until back at the start. Then the blade is used likewise to remove the next ring of soil below.

As one works down into the hole, soil having fallen to the bottom must be removed carefully by hand or with a small dipper, taking care not to brush against the freshly exposed sidewalls of the hole.

Depending on the size of the hole, several wider rings may need to be removed before the desired diameter is reached. The bottom should be almost flat. Also, select the smallest hole diameter local practices permit.

The objective in hand-digging the test hole is to have a hole with sidewall soils in as nearly an undisturbed state as possible. Breaking soil away best meets that objective. Never cut the soil in a shearing, chisel-like manner. Ideally, most of the hole sidewall will not have had contact with the putty knife, and no crushing forces will have been suffered by the soil. The bottom of the hole needs less care.

When the hole is finished, it will have ragged, irregular sidewalls of soils with pores and small roots (if present) in view. It will look much like a clod of soil if one breaks the clod and exposes a fresh soil face.

PAPER BASKET LINER

Cut the bottom out of a large paper bag and cut the bag open along a side. Lay the opened paper bag onto grass, onto a soft garment, or otherwise support the paper such that a pointed object can be used to punch holes in the paper about as big in size as an ordinary pencil.

Punch holes in the paper about 2 or 3 in. apart and in rows spaced 3 or 4 in. apart. The intention is to perforate the paper so water can easily pass through. An excessive number of holes is neither needed nor desirable.

Roll the perforated paper to form a tube with the shorter dimension being the axis. Place the tube into the prepared percolation test hole. Then open the tube until it softly fits the sidewalls of the hole. The size of the bag should have been chosen so that the side walls of the hole are covered with considerable overlapping of the paper upon itself. The length of the paper tube may have to be cut so that the tube rests on the almost flat bottom of the hole and protrudes a couple of inches, or so, above the top of the hole. Lastly, roll the top of the paper back to form a stiffened collar and to hold the overlap in place.

Fold the bottom of the bag which had been first cut out, over itself once. Then fold it sideways such that the first fold lays upon itself. Holding the folded bag bottom at the corner of the folds, cut a quarter circle at the outer edge such that a full circle is had when the bag bottom is unfolded. Choose a radius of the quarter circle about an inch or so larger than the radius of the circular paper tube in the test hole. Then place the circular bag bottom at the test hole bottom. Unfold it within to get it inside. The bag bottom with a larger diameter than the paper tube lining the hole should be pushed into place at its outer edges to fit well.

Lastly, place gravel over the bag bottom at the bottom of the hole so as to hold the paper in place when water is added later. Only an inch or so of gravel is needed. Sometimes a flat rock serves when gravel is not at hand. A 2-in. layer of fine gravel or coarse sand could be used instead of using the bottom of the paper bag at the hole bottom.

When a large paper bag is not on hand and the test holes are to be only 6 in., or so, in diameter, newspapers can be used instead of a paper bag to form a paper basket inside the hole. A double thickness is needed.

FLOAT GAUGE

Difficulty in measuring water level changes with other methods favors the use of float gauges for measurements of water levels inside test holes. Such gauges provide needed accuracy, and they are much easier to use than tape measures, yardsticks, and the like. Float gauges are easily devised.

A metal rod, 1/2 in. in diameter and about 18 in. long, has a sharpened end. The rod is driven into the ground beside small test holes, or into the outer edge of the inside bottom of a wide test hole. The rod is most conveniently driven at an angle about 30 to 45 degrees from vertical and to one side of the hole center. A clamp is next connected to the rod. The clamp should be able to connect in any angle in any plane (a right angle, swivel type clamp).

Next a clear, rigid, plastic tube about 1/2 in. in diameter and 14 in. long is clamped vertically over the test hole. Graduations on the plastic tube can be provided by an adhesive, plastic tape printed in inches and graduated in tenths of an inch. A styrofoam ball about 2 in. in diameter serves as a float. A hollow brass tube, about 3/16 in. in diameter (from hobby or hardware shops), about 24 in. long, is pushed into the float ball. The float rod is inserted into the vertical plastic tube from below. Then the graduated tube is adjusted up or down in its clamp and fixed so that the upper end of the float rod will be within the graduations when the hole is either empty or filled with water.

ADDING WATER TO THE HOLE

Water supplies, usually, must be taken to the percolation test site. Convenience is important, and clean water is essential. Use clean 5-gal plastic jugs, each with a screw-capped opening and a separate small hole for releasing air locks. They can be hand carried a distance, being quite portable. A few jugs can constitute a sizeable water supply, and they can be conveyed to the site by ordinary automobiles. They are superior to commonly used, dirty, heavy, 55-gal oil drums, which must be mounted on a truck and cannot be carried to a place inaccessible by truck.

With the paper basket in place, water can be carefully poured from the jug into the hole. Where for some reason a paper basket is not in the hole, a flexible hose should be connected to the jug and water eased into the hole through the hose. Pinching the hose will cause the water flow to be gentle and only fast enough to fill the hole faster than water seeps away. The soils under no circumstances should be exposed to a direct flow of water which might churn them. Break the flow on a rock or by cupping the hand over the hose end.

An expensive but most manageable hose has a 5/8-in. inside diameter and a 1/8-in.-thick wall, is called Amber Latex tubing, and is available from chemical supply houses. A rubber stopper fits into the water jug's screw-capped opening, and the hose is slipped onto a piece of 1/2-in. copper plumbing tubing which has been fitted through a hole in the rubber stopper of the jug. There must be a hole to prevent airlocking.

When performing a USPHS test, it is often convenient to water fill the test hole to the mark at which presoaking begins. Measure the drop in water level during the first 5 min. Then fill the hole with water for routine presoaking. The 5-min.-presoaking water drop, as a coarse rule of thumb, tends to be about the drop that will be measured during the last 1/2 hr. of the 4-hr. test period on the following day. Not always, but often enough, that rule of thumb helps to guide the technician in his plans for time and water amounts needed for the test on the following day.

THE MINI-PERC

When preparing a percolation test program, it is helpful to know about how fast water supplies will dwindle. Water supplies must generally be brought to the site, and the amount of water needed is related to the rate at which it will be used.

Sometimes it is helpful in viewing soils in profile to be able to predict chances that percolation tests will pass locally set limitation on acceptable rates. In addition to making experienced visual judgments which often are about equal to percolation tests, the writer has found that a small bottle of water with an eye dropper can be used as follows for subjective judgments.

Break a clod of the subject soil open to expose a face of fresh soil. From a height of about 1 in., cause one drop of water to fall onto the horizontally held soil face. The water drop will spread, wet approximately a circle, and infiltrate. Before infiltration, the water surface will glisten. At the exact time of complete infiltration, the glistening will suddenly give way to a dullness. When the soil is rapidly permeable, the wet circle will seem scarcely more than the drop in diameter, and only a fraction of a second will pass before infiltration is complete. When a slowly permeable soil is in hand, the water drop will wet a much larger circle, and the time required for infiltration will be on the order of several seconds. If the soil is about as permeable as a rock, the water drop will spread for quite some time over the open face of the clod, and infiltration will not be a sharply defined event.

The mini-perc likely would lend itself to quantitative measurements, but the writer has neither performed the developmental studies needed nor made plans to do so. He has performed the mini-perc at many sites where he also performed percolation tests. Thereby, a subjective experience has been gained which helps to predict the outcome of percolation tests to come.

A NOTE

The question is often asked, "Why such care in the percolation test, considering the soil destructive actions of a backhoe digging a disposal field? After all, the percolation test seems to be a freshwater simulation of a disposal trench."

The question is answered by the word "empiricism." The test is empirical. A short-term, fresh-water test cannot simulate the long-term, wastewater infiltration of a disposal field. And it would be technical nonsense to use a soil-destructive percolation test to guide soil-destructive construction practices. The goal should be percolation tests performed to obtain correct measurements, and the effort should be directed towards improvement of construction practices to reduce soil damage. The biggest step in the latter direction would be to dig soils for disposal fields of a septic-tank system during drier seasons of the year.

RYAN'S PERCOLATION TEST PROCEDURE IS RECOMMENDED

Henry Ryan's test is simpler in procedure than the USPHS test and most other varieties of tests. Ryan's test, being more oriented in logic, does not suffer the strange behaviors the illogical USPHS test suffers. Still, more exact wording is needed for Ryan's test, inasmuch as Ryan's simple wording has escaped understanding in many minds.

GENERAL

It is assumed that the technician understands careful techniques of working with soils, discussed above, and uses them in performing the test procedures to follow.

PREPARING THE HOLE

Remove soils overlaying the stratum of concern such that a technician will have space needed to perform the test within that stratum.

After having brushed loose soil to one side, hand-dig the test hole to either 12 in. square or about 15 in. round. The bottom of the test hole should be 7 or 8 in. below the working surface of the exposed soil stratum.

Place a perforated paper basket into the hole with the basket bottom held in place with an inch or two of clean gravel or another clean weight.

Fit the hole with a float gauge such that when 6 in of water are over the soil bottom of the hole the technician will be able to ascertain that exact water level with the gauge. Where soils are to be presoaked overnight and it would be undersirable to leave the float gauge in place, make whatever measurements or preparations are needed to fit the float gauge properly at the time of the test.

PRESOAKING

The purpose of presoaking is to cause soil colloids with a shrink-swell potential to swell. If such colloids are present in quantities enough to result in extensive cracking of dry soil, perform the test only when such cracks are swelled shut. When doubt exists, or when a clear understanding of the pertinent aspects of soil science is not had, presoak.

Presoaking is accomplished by water filling the hole once, or twice, and leaving the hole overnight. Where large cracks in the ground are had, extensive presoaking or a season of rains may be needed to shut the cracks.

Water fill the hold to exactly 6 in. deep over the soil bottom. (Do not regard gravel or the like holding the paper basket in place as the soil bottom.) Record the time required for the water level to recede to 5 in. deep. As soon as possible, and not letting more than a minute or two pass, refill to 6 in. deep. Again record time needed for water to recede to 5 in. deep. Continue this process until successive time measurements agree within 10 percent. The last measurement is the percolation rate, min/in.

SOILS WITHOUT MUCH COHESION

Some soils have little cohesive strength, and a percolation test hole in them will not tolerate water fillings without collapse of sidewalls or migration of soils into the hole.

COLLAPSING SIDEWALLS

Collapse can most times be prevented by the paper basket described. It is surprising how much support paper can offer. As a matter of record, the paper basket was devised to prevent collapsing of test holes in a soil especially prone to do so. Finding a reduction in damage from water fillings was a happy benefit.

Unaware of the use of paper baskets for support of sidewalls of percolation test holes, some technicians place a length of 4-in diameter, perforated, plastic pipe into a test hole. Then they fill the annular space between the pipe and the sidewalls of the hole with gravel. The test is performed inside of the pipe.

In some places authorities prescribe pipe and gravel as routine. Unfortunately, many people do not realize that water levels in such gravel-assisted holes recede faster than in the same-sized holes without gravel and no corrections of overly liberal results are made.

Rather than undergo the mathematical considerations to account for presence of gravel, a technician might prefer to construct a hole about 2 in. or so larger than the finished hole was to have been. Then he might fit a wire basket, or the like, of the intended hole dimensions inside of the larger hole. Small gravel could then be placed between the basket and soil wall of the hole. Although gravels and soils differ in porosity, it would be sufficiently accurate for most purposes to assume porosity of the gravel the same as that of the soil, and therefore the inside dimensions of the basket are essentially those of the originally intended test hole.

SOIL MIGRATION

Soils with large amounts of silts and few clays are prone to migrate into the bottoms of water-filled holes. Whereas collapse of a sidewall is obvious, soil migration is not. It is observed when a technician returns the next day after presoaking and he finds the test hole is not as deep as it was the day before. When silts are found to migrate, a special construction technique will be needed for both a percolation test hole and the disposal field to follow (Chapter 14, Ref 15).

A square hole could serve, but a round hole might be easier to manage. Say the originally intended test-hole size was to have been about 15 in. round. Dig the hole to about 21 in. round. Construct three sheet metal rings, 19 in., 17 in., and 15 in. in diameter and longer than the hole is deep. Perforate only the 15 in. ring with holes about 1/8 in. in diameter, spaced about 2 in. apart, and in rows about 2 in. apart. Place the rings concentrically within the hole. Fill the annular space, and finally fine gravel in the annular space of the inside ring. Carefully lift out the two outer rings. Perform the test within the remaining, perforated, inside ring.

The intention of the foregoing is to provide a fine sand facing the soils to keep silt from migrating to the inside of the hole when water is added.

SOIL PERMEABILITY

The empiric percolation test is related to soil permeability, but it does not measure it, as such. The objectives of the percolation test are decisions on acceptability of the soil for septic-tank practices and design sizes of disposal fields. Once septic-tank effluents have infiltrated a soil through a man-made device, the disposal field, and factors affecting performance, such as the ability of the soil to conduct the effluents away, soil permeability and hydraulic gradient, generally are ignored.

Unlike percolation tests which yield values related to specific procedures followed, measurements of soil permeability properly performed yield same value for same soil samples. In his text, Professor Laak presented design criteria for disposal fields related to direct measurements of soil permeability [40]. Laak described a falling-head permeameter and a pit-bailing test for assessment of soil permeability.

In California, persons qualified either to perform or to judge the value of measurements of soil permeability are neither common nor much involved with septic-tank practices. Where regulations specify a percolation test, local authorities are acquainted with meanings of results which they must process. A pit-bailing test is not a common field practice. However sound it may be, local authorities are not prepared to interpret its results and have refused pit-bailing tests, and the use of his falling-head permeameter tests. Many western soils are so stone filled that valid samples for permeameter tests can be collected. Clients are more willing to support required percolation-test programs than measurements of permeability which are not required by authorities.

Winneberger has correlated the USPHS percolation test to measurements of soil permeability [41]. Thus, a required percolation-test program can be performed to satisfy local demands, and data can be translated by correlation to k, Darcy's coefficient of permeability. That value can be used to evaluate the assimilative capacity of a site.

At a correlation coefficient of 0.75, paired data from 34 sites yielded (corrected from earlier publication)

$$\log k = -4.76 + 1.55 \log p$$

where Darcy's coefficient of permeability, k, has units of cm / sec; and the USPHS percolation rate, p, has units of in./hr. The USPHS test must be performed in a 4-in.-diameter hole, and techniques of hole digging and water filling described herein must be followed for the formula to be accurate.

To provide a reader with a scale of reference, Table 3.5 is presented. The coefficients of permeability and descriptions of porous media are from A. Casagrande and R. E. Fadum, well known names in soil mechanics. The corresponding percolation rates are from the formula of correlation just described.

APPENDIX D. MEASUREMENT OF HYDRAULIC CONDUCTIVITY FOR UNSATURATED SOILS USING THE CRUST TEST [18]

The percolation test is valid only for measurement of hydraulic conductivities of saturated soils. Bouma et al [18] have devised an in situ test to measure the hydraulic conductivity of an unsaturated soil, which is called the crust method.

THEORY

The hydraulic conductivity of a soil is a function of its moisture content. At low moisture contents, negative pressures which can be measured are created in the soil. Low conductivities are associated with high negative soil pressures and low moisture content. In the crust test, crusts of varying hydraulic resistance are placed on top of a soil column and the infiltration rate for each crust is measured. The crust has a lower hydraulic conductivity than the soil column so that the soil column is not saturated even though the crust is subject to a small positive head.

The infiltration rate into the soil is defined as

$$I = (Q/t)/A = q/A \qquad \text{(Eq. D.1)}$$

where

= infiltration rate (L/T)

q = total flow rate through burette (L^3/T)

A = cross sectional area of the column (L^2)

Darcy's law for flow through soil states

$$q = kiA \qquad \text{(Eq. D.2)}$$

where

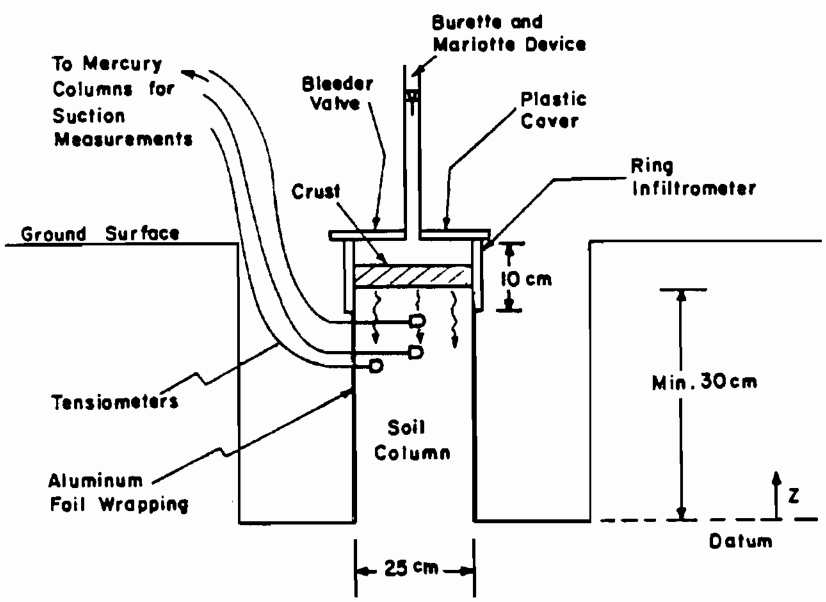


Fig D.1. Bouma crust test soil column (1975).

= total flowrate (L^3/T)

k = hydraulic conductivity (L/t)

i = hydraulic gradient (Dh/DL)

A = cross sectional area (L^2)

The hydraulic gradient, i, for flow through soil is defined as

$$i = Dh/Dz = D(hp + z)/Dz \qquad \text{(Eq. D.3)}$$

where

hp = pressure head

z = elevation above arbitrary datum

If hp is constant over the soil column elevation, then

$$i = Dz/Dz = 1 = \text{unity}$$

Combining Equation D.2 and D.3 yields

$$q = kiA = kA, \text{ or } k = q/A \qquad \text{(Eq. D.4)}$$

but Equation D.4 is equivalent to the infiltration rate I. Thus, if the soil suction hp is constant throughout the vertical soil column then the flowrate measured from the burette is equivalent to the unsaturated hydraulic conductivity (k).

PREPARATION OF SOIL COLUMN

Bouma's crust test column is shown in Fig. D.1. A 25-cm-diameter cylindrical soil column 30 cm in height is carved out of the soil in question. A 10-cm-thick steel ring infiltrometer 25 cm of diameter is fitted to the top of the soil column. The crust, which is either a porous ceramic plate or an artificially prepared clay barrier, is placed on top of the soil column within the ring infiltrometer.

A plastic cover, with a rubber gasket glued to it, is bolted to the infiltrometer. A burette is inserted into an intake port in the cover. The burette has a Marionette device within to maintain a constant pressure of 3 mm. One tensionmeter is placed just below the crust in the cross-section center; two others are placed 3 cm below the crust, in the center and periphery of the cross-sectional area. Tensionmeter tubes are connected to calibrated mercury columns to measure soil suction. Aluminum foil is wrapped around the soil column below the ring infiltrometer to the bottom of the column.

An improved crust test column is shown in Fig. D.2. The ring infiltrometer extends into the base of the column as shown and is made of PVC. Gauge-type tensionmeters replace the mercury-type tube and are placed at a minimum of three depths in the soil column.

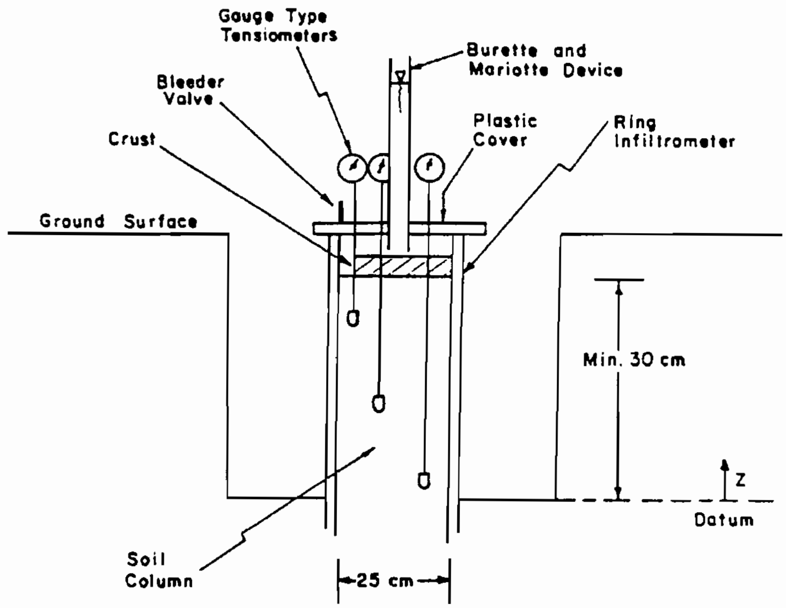


Fig D.2. Improved crust test column.

TEST PROCEDURE

A series of tests using progressively higher permeable crusts is carried out on soil that is initially low in moisture content. For each test the crust is placed on the soil and water is introduced via the burette. The space between the cover and crust is filled with water and air is expelled out of a bleeder valve on the cover. Water is continually added until the infiltration becomes steady, and the rate of water movement in the burette is recorded as soon as the tensionmeters read the same negative pressure head (suction head).

The infiltration rate, when steady for a period of 4 hours, is taken as the unsaturated hydraulic conductivity at the sub crust level when the suction gradient is zero (i.e., all tensionmeters read the same negative pressure head). If the suction gradient is not zero then the unsaturated hydraulic conductivity is

$$K = v/i$$

where

v = infiltration rate

i = hydraulic gradient below the crust

Each crust test for each soil type is plotted as a single point and a series of crust tests for a soil can be plotted as shown in Fig. D.3. Fig. D.3 is a plot of Bouma's results for four soil types in Wisconsin.

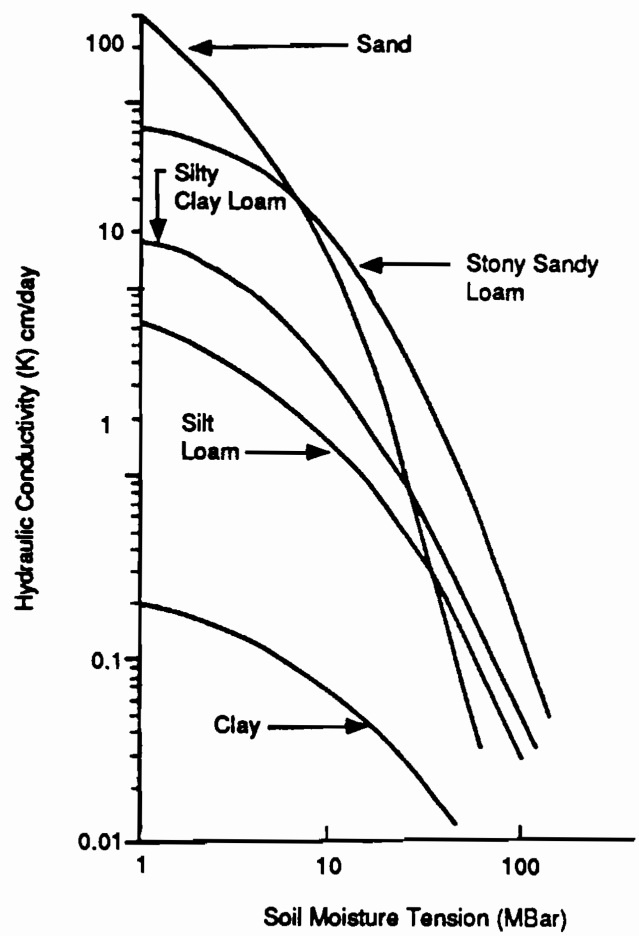


Fig D.3. Hydraulic conductivity (K) versus soil moisture tension relationship for a sand (B^3 of Plainfield Sand), a stony sand loan glacial till, a silt loam (A^2 of Plano Silt Loam), a silty clay loam (B^2 of Plano Silt Loam), and a clay (B^3 of Ontonagon Silty Clay) [18].

APPENDIX E. DEVICES FOR MEASURING SCUM AND SLUDGE IN SEPTIC TANKS [14] (ADAPTED FROM WINNEBERGER)

After experiencing failure with methods advised in the *Manual of Septic-Tank Practice* [11], people often request advice on methods for measuring scum and sludge in septic tanks. It seems a worthwhile digression herein to present methods known to work.

The USPHS *Manual of Septic-Tank Practice* presents a drawing of a stick with turkish toweling wrapped around the bottom 3 ft. of the stick. The homeowner is advised to lower the stick into the septic tank and remove it carefully. The sludge level is supposed to be distinguished by particles adhering to the toweling. Winneberger and others acquainted with the contents of septic tanks have had poor results with the turkish towel method.

When USPHS studies of septic tanks were performed, fieldmen did not use turkish towels wrapped around sticks for measuring sludge levels. Rather, another device was described. It "was a jointed (3/4") copper tube with a water tight window at the bottom, below which was fixed a flashlight bulb....The inspector observed the light bulb through the tube as the tube was lowered into the tank....The light blanked out at the sludge level, recorded as the top of sludge." Reproductibility by different operators was about 1/8-in.

During the 2-yr studies, the writer constructed devices for measuring scum and sludge (Fig. E.1). The scum measuring device had a 1-in. x 2-in. wooden handle, long enough to comfortably reach scum. Because of risers over tanks to ground level, it was found that a 5-ft-long handle was about right. At the bottom of the handle, a 1/4-in. x 5-in. x 10-in. exterior plyboard "flopper" was hinged on one side of the handle and at about the middle of the flopper. A steel angle brace on the opposite side of the hinge side was fixed such that the flopper could be moved to about right angles with the handle. A nylon string fixed at each end of the flopper was used to control it.

To measure scum, the flopper was pulled up vertically alongside the handle. The flopper was then shoved through the scum mat. Then the other side of the string was pulled until the flopper was stopped by the angle brace, at right angles to the handle. The device was pulled upwards until the bottom side of the scum mat was felt. The thickness of the mat was noted to the nearest inch mark on the ruler attached to the handle.

Scum mats are not flat near the inlet end of a septic tank; rather, they are mounded there. So, measurements were made away from the inlet device. Sometimes a cross stick nailed on a handle was laid onto the top of the scum mat and against the ruler. That made it easier to determine just where the top level of an irregular mat was.

The sludge measuring device was constructed heavier than the USPHS device. The bottom end of the device, drawn in Fig. E.1, was sealed by epoxy resins for watertightness and firmness. The "Christmas Tree" flashlight socket was sealed at the wire end by a silicone sealant and the wires were laid against the bottom end of the pipe (not loose as drawn) and held in place by the silicone sealant. The flashlight bulb was made watertight by seating it through a greased O-ring, which fitted between the bulb's underside and the outer edge of the socket. It was found convenient to fit the upper end of wires with jacks for disassembly convenience.

The sludge measuring device should be of sufficient length and diameter to permit viewing the sludge level and, perhaps, the bottom of the tank. The sludge thickness may be measured by noting where the scum mat or other reference point is on the pipe (a ruler taped thereon might help), just when sludge is reached; then measuring to the tank's bottom, and adding the distance between the light bulb and the plastic adapter bottom. It helps much to have a second person's assistance.

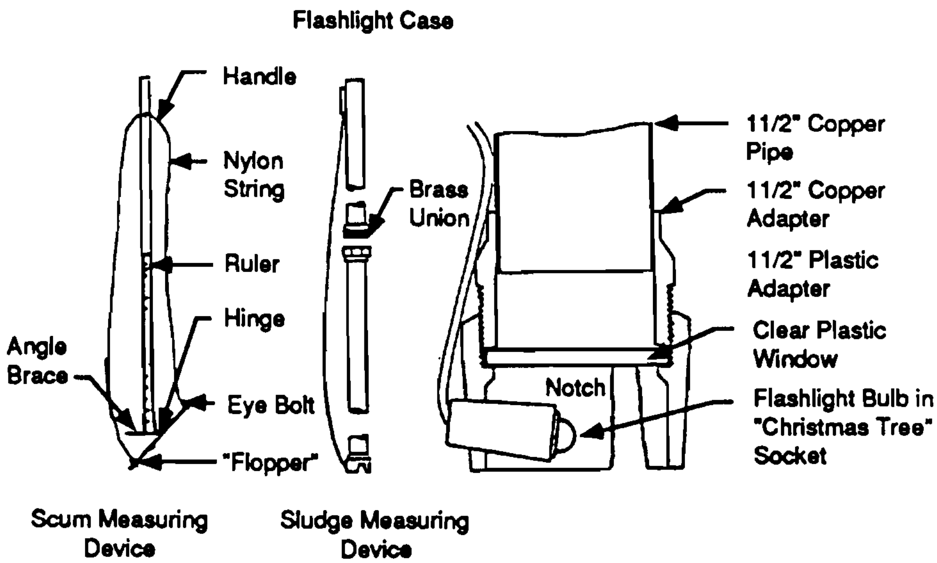


Fig E.1. Devices for measuring the thickness of a scum mat and the depth of sludge in septic tanks.

The bottom side of the plastic adapter should be notched as much as it is believed possible without destroying the strength of the sides remaining. It is necessary that water not be trapped within, such as to depress the sludge's entry. Slow, careful lowering of the device is important.

Separating the bulb from the clear (or light-transmitting frosted) plastic window by a variety of distances was not tried, but about 1/2 in. seemed far enough away. Sludge blanked out the light easily within that 1/2 in. of separation.

APPENDIX F. EVAPORATIVE POND DESIGN CALCULATIONS

Pond surface area and volume and the rate of solids build-up for an evaporative pond at the San Marcos rest area were calculated as follows.

Given: Estimated average daily traffic (ADT) =14,000veh/day

Percentage of ADT stopping at rest area = 7

Wastewater production = 5.5 gal/veh (Zaltzman value)

Estimated evaporation rate = 55 in/yr

Estimated rainfall rate = 32 in/yr

BOD_5 = 200 mg/L

Suspended solid (SS) = 200 mg/L

Pond depth = 4 ft

CALCULATION OF POND(S) SURFACE AREA AND VOLUME

1. Evaporation rate excess

   E (excess) = 55 - 32 = 23 in./yr.

   23 in./yr X 1 ft/12 inches X 43,650 sq ft/acre = 83,500 cu ft/acre/yr

2. Compute yearly wastewater flowrate

   Q (daily average) = 14,000 X 0.07 X 5.5 gal/veh = 5,400 gal/day

   5,400 gal/day X 365 days/yr = 1,967,000 gal/yr

   Q = 1,967,000 gal/day X .1335 cu ft/gal = 262,800 cu ft/yr

3. Compute surface area

   SA = 262,800 cu ft/yr + 83,500 cu ft/acre/yr = 3.1 acres

4. Compute volume of ponds

   VOL = 3.1 acres X 43,560 cu ft/acre X 4 ft = 540,144 cu ft

   = 4,040,000 gallons

CALCULATION OF POND SOLIDS DEPOSITION RATE

1. Calculation of mass loadings to pond

   SS (lb/day) = 200 mg/L X 5400 gal/day X 3.785L/gal X 2.2 lb/kg X 1 kg/10^6 mg = 9 lb/day

   BOD_5 (lb/day) = 200 mg/L X 5400 gal/day = 9 lb/day

   Assume half of the total biomass (algae + bacteria) settles to the bottom of the pond:

   BOD_5 (lb/day) = 4.5 lb/day

   Total solids to pond = 4.5 + 9 = 13.5 lbs/day

2. Calculation of fixed solids settling to pond bottom

   Assume suspended solids volatile fraction is 65%, assume BOD volatile fraction is 80%:

   SS volatile = 9 X 0.65 = 5.85 lb/day = 5.9 lb/day

   BOD_5 volatile = 4.5 X 0.8 = 3.6 lb/day

   Total volatile solids = 3.6 + 5.9 = 9.5 lb/day

   Total fixed solids = Total solids - Total volatile solids = 13.5 - 9.5 = 4 lb/day

3. Calculation of mass solids per cu ft of wastewater

   Assume sludge is 10 % solids (mass solids/mass sludge), specific gravity of sludge is 1.1, and the density of water is 62.4 lb/cu ft:

   Mass solids/cu ft sludge = 0.1 lb solids/lb sludge X 62.4 lb/cu ft X 1.1 = 6.86 lb solids/cu ft

4. Calculation of deposition rate

   Assume that fixed solids equals volatile solids in the sludge after digestion (i.e. in the long run) so

   Total solids = 4 lb/day X 2 = 8 lb/day

   Vol solids accumulated per day = 8 lb/day + 6.86 lb solids/ cu ft = 1.17 cu ft/day

   Yearly rate = 1.17 cu ft/day X 365 days/yr = 425 cu ft/yr

   Yearly deposition rate = 425 cu ft/yr + 43,560 sq ft/acre = .01 ft/yr

5. Calculation of time required for 1 ft of deposition of solids

   Time for 1 ft deposition = 1 ft + .01 ft/yr = 100 yrs

Comments: Approximately 3.1 acres are needed for the total surface of the pond system; three one-acre ponds would be suitable. The deposition rate calculated is based on a surface area of one acre (one pond) for a conservative design. The rate calculated shows that ponds can operate many years before any cleaning will be required and, in fact, may never need cleaning.

APPENDIX G. AREA CALCULATIONS FOR EVAPORTRANSPIRATION BEDS USING A SUCCESSIVE APPROXIMATION METHOD [29]

The size of an evapotranspiration bed is controlled by (1) the anticipated loading rate and (2) the minimum daily pan evaporation for the location. The successive approximation method calculates the monthly effluent level below grade for a chosen area and effluent loading rate. These monthly levels will give an approximation or an average effluent level to use over the year. By varying the area of the bed in the calculations the effect on the bed effluent level can be observed.

To use the successive approximation approach monthly pan evaporation data for a ten-year period-of-record for the region near the site are necessary. In addition, the soil porosity should be known or estimated. Pan evaporation data are available from the United States Weather Service and porosity values can be approximated using data from the U. S. Soil Conservation Service or determined in soil laboratories.

In general, the effluent bed level should be kept greater than 10 inches below grade for proper functioning of the ET beds.

The method outlined by Rugen et al has the following steps:

1. Choose a reasonable bed area and depth for the ET beds as a first guess.
2. Find the minimum monthly pan evaporation over the period-of-record for each month (Fig. G.1) and divide this value by the number of days in the month to obtain a daily evaporation rate for each month of the year in inches per day . Convert to gallons per square foot per day using the following conversion:

   $gal/ft^2/day = in./day \times 1\ ft/day \times 7.48\ gallons/ft^3$

3. Find the evapotranspiration rate (ET) using Fig. G.2 for a bare tank at a given effluent level. To start the calculations assume an effluent level of 6 inches above the tank bottom.

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Adj. annual
1914	2.72	2.35	3.93	4.89	4.37	7.55	9.80	7.72	8.03	4.40	2.65	2.09	60.50
1915	2.26	2.81	3.37	3.36	6.00	8.99	11.33	8.47	5.29	5.46	4.07	2.51	63.92
1916	2.36	3.79[1]	7.21	6.64	7.32	9.92	7.21	7.69	6.26	5.00	3.44	2.82	69.66
1917	2.61	3.99	5.60	7.68	7.56	9.88	9.89	9.81	6.93	6.17	3.63	2.92	76.67
1918	3.41	2.54	5.83	5.76	6.94	8.16	10.36	10.03	7.88	4.57	2.47	1.89	69.84
1919	1.79	2.48	3.04	5.50	5.52	5.81	5.64	7.06	5.16	3.73	2.77	2.06	50.56
1920	1.31	2.56[1]	4.37	7.23	6.25	6.05	7.14	10.50	7.18	5.21	2.80	2.99	60.12
1921	2.46	3.36	3.78	4.58	6.51	7.56	9.29	9.94	6.16	5.37	3.60	3.57	66.74
1922	2.82	3.30	4.59	4.34	5.39	5.95	10.01	8.90	7.56	6.17	2.77	2.99	65.83
1923	3.25	1.62	4.52	3.91	7.52	9.36	8.96	9.97	5.45	4.84	2.24	1.79	62.36
1924	2.07	2.78[1]	3.73	4.96	5.48	7.45	8.80	9.97	7.39	5.21	4.82	2.56	65.22
1925	2.69	4.43	6.12	8.06	8.38	10.16	11.27	9.98	6.07	4.88	2.55	2.31	76.90
1926	1.71	3.80	3.54	3.54	5.59	7.19	7.22	8.82	7.31	6.00	4.19	2.19	61.10
1927	2.04	2.50	3.80	5.97	7.99	6.13	7.82	10.85	7.37	5.21	4.47	2.37	66.52
1928	2.91	2.57[1]	4.93	6.05	6.10	7.49	9.99	9.98	4.76	4.96	2.50	2.08	64.32
1929	2.22	2.71	3.84	4.65	6.01	7.09	6.71	8.78	6.83	4.43	2.64	2.54	58.45
1930	1.66	2.72	3.69	4.86	4.86	6.76	8.84	9.82	7.96	3.95	2.52	2.19	60.53
Daily Avg.	.08	.10	.14	.18	.20	.26	.29	.29	.22	.16	.11	.08	

Notes: 1. February months with 29 days.

2. Equipment type: Bureau of Plant Industry 6-ft diameter pan.

Fig G.1. Pan evaporation data[2] for San Antonio.

4. Calculate the total quantity of water evapotranspired using the following equation:

   Q_{ET} = (ET)(no. of days in the month)(assumed area of bed)

   where

   Q_{ET} = Total evapotranspiration from the system (gal/month)

   ET = Evapotranspiration rate calculated in step 3
5. Calculate the monthly inflow rate to the ET beds:

   Qinflow(gal/month) = (Q_{inflow}, gal/day)(days in the month)
6. Calculate the effluent in gallons which is not evaporated in the system over the month:

   D system = Q_{inflow} - Q_{ET}
7. Calculate E_L, the effluent level, using the following formula:

   $E_L = D - (P)(D\ system)$

   where

   E_L = effluent level below grade (inches

   D = total depth of evapobed (inches)

   P = 100%/porosity of capillary media in %

   D system = effluent not evapotranspired (ft^3)

   A = surface area of evapobed (ft^2)

   To use the equation the value in step 6 must be converted from gallons to feet squared and the second term on the right side of the equation multiplied by 1/12 to convert to inches.
8. Use calculated E_L value to do calculations in steps 3 - 7 for the next month and proceed through the remainder of the months of the year and plot the effluent level vs. the month of the year. If necessary, repeat the process for year 2, 3, etc. until the effluent level stabilizes.

Now that the effluent level for that area has been calculated a choice can be made if the areas used will produce an effluent level that is acceptable. An example of the successive approximation method follows:

Assume: Rest area site near San Antonio

Daily flowrate Q (gal/day) @ 5,000

Pan Evaporation rates as given in Fig. G.1

Bed depths of 30 inches

Initial Bed(s) area (ft^2) @ 40,000

Calculations - Calculations will be completed to fill in the chart given in Fig. G.3.

1. *Calculate the minimum Daily Pan Evaporation for July. From Fig. G.1*

   min evaporation (in./mo) = 5.64 in/mo

   5.64 in/mo x 1 ft/12 in. x 1 mo/31 days x 7.48 gal/ft^2 = .113 gal/ft^2/day

   This value is entered in column three of Fig. G.3.
2. *Calculate the ET rate*

   Use the evaporation rate above in conjunction with Fig. G.2 to find the ET rate. Assume an effluent level of 6 inches above the bed bottom so that the effluent level is 30 - 6 = 24 inches (below grade).

   From Fig. G.2

   ET (gal/ft^2/day) = .022

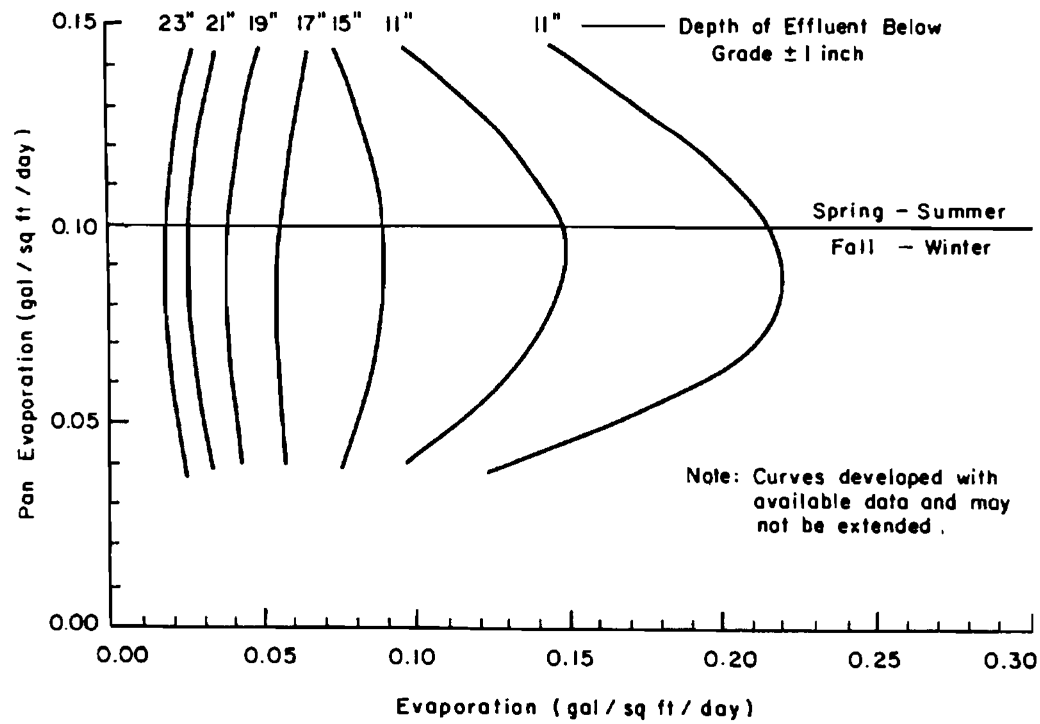


Fig G.2. Relationship between pan evaporation and evapotranspiration, bare tanks.

Enter this in the fourth column of Fig. G.3.

3. *Calculate the total Evaporation Q_{ET} (gal/mo)*

Q_{ET} = ET (no. of days in mo)(area of bed)(ft^2)
Q_{ET} = 0.022 (31)(40,000)
= 27,280 gal/mo

Enter this value in column 5 of Fig. G.3.

4. *Calculate the monthy effluent Inflow Q_{inflow}*

Q_{inflow} = (daily loading gal/day)(no. of days in mo)
= (5000)(31 days)
= 155,000 gallons

Enter this value into column 6 of Fig. G.3.

5. *Calculate the change in value stored in the system (D system)*

D system = $Q_{inflow} - Q_{ET}$ = 127,720

Enter this value into column 7 of Fig. G.3.

6. *Calculate the effluent level at the end of July*

E_L = D - P(D system)/A
= 30" - [(100%/50%) x (127,720 gal/40,000 ft^2) x (1 ft^3/7.48 gal) x (12 in./1 ft)
E_L = 30" - 10.25" = 19.75"

Enter this value in the last column in Fig. G.3.

The effluent level calculated for July is now used to find the ET rate from Fig. G.2 using the minimum pan evaporation rate for August. The values for August are calculated following the above steps and the procedure is repeated for each month of the year. Extrapolations for bed effluent levels in between those shown in Fig. G.2 have been made to complete Fig. G.3.

If a bed area of 30,000 ft^2 is used the effluent level stabilizes around 18 inches below grade (Fig. G.4). Thus, the effect of choosing different bed surface areas can be compared graphically (Fig. G.5).

A factor beta, b, can be calculated by dividing the surface area of the ET bed by the effluent loading rate for each different surface area used. Then b can be plotted vs. the bed effluent level. An example of this plot is shown in Fig. G.6. Now a suitable surface area can be selected by choosing an effluent level, finding the corresponding beta term, and then multiplying beta by the expected loading rate (Q_{inflow}).

The analysis presented here for a rest area wastewater flow suggests a surface area of 25,000 to 30,000 square feet for the ET system. This value is roughly .6 to .7 acre. The volume of sand needed will be roughly 75,000 ft^3 (30,000 x 2.5) and the excavated material (if low in permeability) will need to be hauled off site. If beds are made 30 x 40, then approximately 25 ET beds will be needed for the example given.

Month	No. Days	Pan Evap., (gal/sq ft/d)	ET Rate, (gal/sq ft/d)	Qet Total (gal/mo.)	Q in, (gal/mo.)	Sys. Gallons.	E level Inches
Jul.	31	.113	.022	27,280	155,000	127,720	19.8
Aug.	31	.141	.038	47,120	155,000	107,880	21.3
Sept.	30	.100	.025	30,000	150,000	120,000	20.4
Oct.	31	.075	.033	40,920	155,000	114,080	20.8
Nov.	30	.047	.031	37,200	150,000	112,800	21.0
Dec.	31	.036	.035	43,400	155,000	111,600	21.0
Jan.	31	.026	.040	49,600	155,000	105,400	21.5
Feb.	29	.035	.035	40,600	145,000	104,400	21.6
Mar.	31	.061	.027	33,480	155,000	121,520	20.3
Apr.	30	.070	.030	36,000	150,000	114,000	20.9
May	31	.088	.025	31,000	155,000	124,000	20.0
Jun.	30	.121	.035	42,000	150,000	108,000	21.3
Jul.	31	.113	.028	34,720	155,000	120,280	20.4

Comments: Each bed is 30" in total depth.
Estimations: Area = 40,000. ft^2, porposity = 50%, loading = 5000 gal/d

Fig G.3. Sample work sheet for effluent level calculations.

Month	No. Days	Pan Evap., (gal/sq ft/d)	ET Rate, (gal/sq ft/d)	Qet Total (gal/mo.)	Q in, (gal/mo.)	Sys. Gallons.	E level Inches
Jul.	31	.113	.022	20,460	155,000	134,540	15.6
Aug.	31	.141	.070	65,100	155,000	89,900	20.4
Sept.	30	.100	.034	30,600	150,000	119,400	17.2
Oct.	31	.075	.052	48,360	155,000	106,640	18.6
Nov.	30	.047	.045	40,500	150,000	109,500	18.3
Dec.	31	.036	.050	46,500	155,000	108,500	18.4
Jan.	31	.026	.055	51,150	155,000	103,850	18.9
Feb.	29	.035	.038	33,060	145,000	111,940	18.0
Mar.	31	.061	.047	43,710	155,000	111,290	18.1
Apr.	30	.070	.047	42,300	150,000	107,700	18.5
May	31	.088	.045	41,580	155,000	113,150	17.9
Jun.	30	.121	.050	45,000	150,000	105,000	18.8
Jul.	31	.113	.040	37,200	155,000	117,800	17.4

Comments: Each bed is 30" in total depth.
Estimations: Area = 30,000. ft^2, porposity = 50%, loading = 5000 gal/d

Fig G.4. Sample work sheet for effluent level calculations.

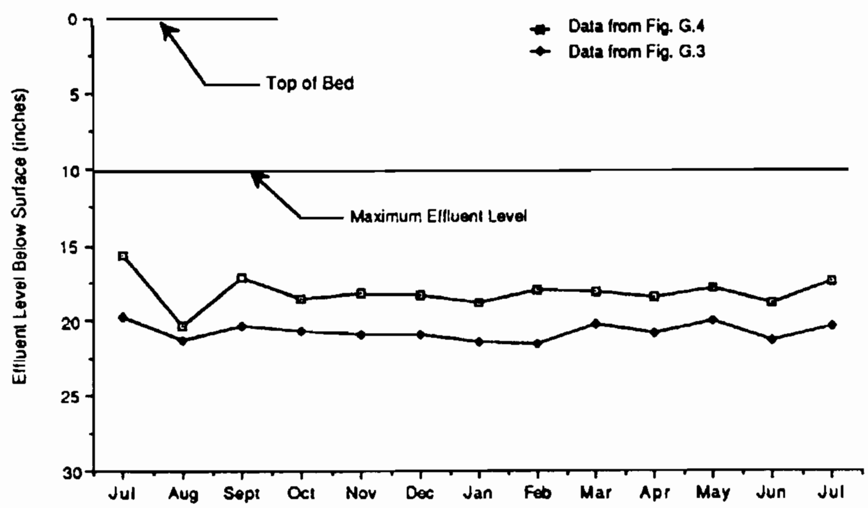


Fig G.5. Graph of effluent level versus time in months.

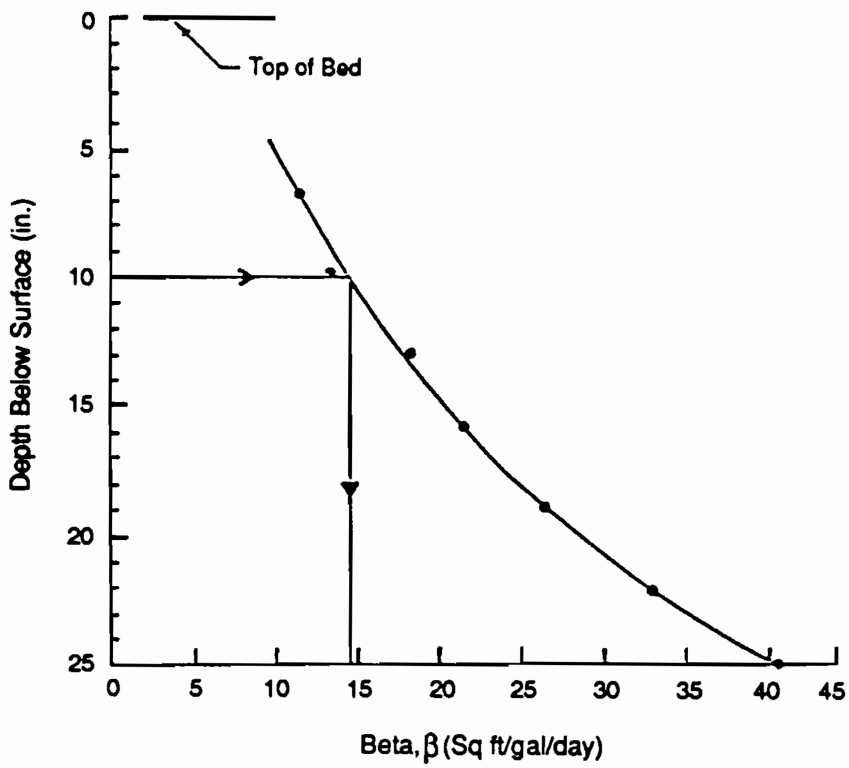


Fig G.6. Beta calculations for San Antonio.

APPENDIX H. RECYCLE RATIO CALCULATION BY PARKER'S METHOD [32]

Recycle ratio = recycle water/total water use
= Δaccumulated flush vol/(Δaccumulated flush vol + Δpotable water use)

Example: Input data

DATE	FLUSH VOLUME USED (gal)	ΔVOL USED	POTABLE VOLUME USED (gal)	ΔVOL USED
3/17	8,245		721	
		16,493		924
3/23	24,738		1,645	
		19,342		1,169
3/30	44,080		2,814	

Calculation of recycle ratio:

Recycle ratio, 3/17 - 3/23 = 16,493 / (16,493 + 924) = .97

3/23 - 3/30 = 19,349 / (19349 + 1169) = .94

Note: The figure used for potable water use should be the amount of water used for rest room purposes only. If outside potable water use is included then the re-cycle ratio will be underestimated.

APPENDIX I. RECOMMENDED DESIGN PROCEDURES FOR HIGHWAY RESTROOM AND RV DUMP STATION TANK-LEACH FIELD SYSTEM [33]

RECOMMENDED DESIGN PROCEDURES FOR HIGHWAY RESTROOM AND RV DUMP STATION SEPTIC TANK-LEACH FIELD SYSTEMS

Restroom and RV Dump Station Septic Tank Detention Time

Sludge accumulation from a household of four of 98 gallons by the end of one year and 188 gallons by the end of four years were used to derive $v = 0.092$ gal/person-day for the rate of sludge accumulation, and $k = 0.0018$ day^{-1} for the first order rate constant for volume reduction due to biodegradation. This value does not lead to under-estimation of sludge accumulation.

In view of the wide range in flow and strength of highway rest area waste it is prudent to design rest area waste disposal facilities for vehicle loadings and waste flows and strengths determined individually at the site in question. Site-specific data may be used in the selection of septic tank detention based on actual waste strength, and preservative concentration and the desired pumpout interval.

The domestic equivalent pollutant load per vehicle using rest area (restroom or RV dump) facilities is:

$$P = C_r Q_r / CQ \qquad \text{(I-1)}$$

where:

- P = domestic equivalent, person-days per vehicle;
- C_r = rest area waste strength parameter value, mg/L;
- C = domestic waste strength parameter value, mg/L;
- Q_r = rest area waste flow, gallons per vehicle; and
- Q = domestic waste flow, gallons per person-day.

The rest area sludge accumulation rate is:

$$v_r = vP \qquad \text{(I-2)}$$

where:

- v_r = rest area sludge accumulation rate, gallons per vehicle; and
- v = domestic sludge accumulation rate, gallons per person-day.

$$V_t = v_r n / ki[1-\exp(-kit)] \qquad \text{(I-3)}$$

where:

- V_t = volume of sludge after time t, gallons;
- n = vehicles per day using rest area facilities;
- k = rate constant for sludge volume reduction due to biodegradation, day^{-1};
- i = coefficient of inhibition of sludge volume reduction due to preservative; and
- t = time from previous desludging of septic tank, days.

Substituting Eqs I-1 and I-2 in Eq I-3 and dividing by the rest area waste flow per day, Q_n, yields:

$$d = C_r v / kiCQ[1-\exp(-kit)] \qquad \text{(I-4)}$$

where:

- d = septic tank detention, days.

COD or suspended solids may be used to evaluate C_r and C. Values for domestic waste of $C = 200$ mg/L for SS and 500 mg/L for COD with $Q = 60$ gal/person-day were observed. Using $v = 0.092$ gal/person-day, $k = 0.0018$ day^{-1} and $i = 0$ for restroom waste or $i = 0.4$ for RV waste, the recommended septic tank detention times may be calculated and are presented in Tables I-1 and I-2.

TABLE I-1. RECOMMENDED HIGHWAY RESTROOM SEPTIC TANK DETENTION IN DAYS

Wastewater strength (use either column)		Design pumpout interval, months		
COD, mg/L	SS, mg/L	6	12	60
500	200	1.5	1.5	1.5
1,250	500	1.5	1.5	2.1
2,500	1,000	1.5	2.1	4.1
3,750	1,500	1.8	3.1	6.2
5,000	2,000	2.4	4.1	8.2

TABLE I-2. RECOMMENDED RV DUMP STATION SEPTIC TANK DETENTION IN DAYS

Wastewater strength (use either column)		Design pumpout interval, months			
COD, mg/L	SS, mg/L	1	3	6	12
1,250	500	1.5	1.5	1.5	1.5
2,500	1,000	1.5	1.5	1.5	2.5
5,000	2,000	1.5	1.5	2.6	4.9
12,500	5,000	1.5	3.4	6.6	12.3
25,000	10,000	2.3	6.8	13.1	24.7

In caution it should be noted that the detention times in Tables I-1 and I-2 contain no safety factors, and are in fact the calculated time from pumpout that sludge will spill from the septic tank to the leach field, rapidly causing failure of the leach field. An essential maintenance task is to check sludge accumulation after perhaps half the design pumpout interval, then more frequently as sludge fills the tank.

Septic Tank and Leach Field Design

In the remainder of this report FHWA recommended design procedures for septic tank-leach field systems (10) are streamlined in the form of a design table and nomographs for rapid design with minimal errors.

Calculation of restroom and RV dump station waste flow is based on the respective equations:

Rest area wastewater flow, in gallons per day
x Percentage of vehicles travelling that enter rest area/100%
x Persons per vehicle
x Gallons of wastewater per person (I-5)

RV wastewater flow, in gallons per day
= Vehicles travelling per day
x RVs travelling as a percentage of total vehicles travelling/100%
x Percentage of RVs travelling that enter rest area/100%
x Percentage of RVs entering rest area that use dump facilities/100%
x RV wastewater discharged per dump, in gallons (I-6)

The septic tank design equation is:

$$QT = V = 7.48\,LBD \tag{I-7}$$

where:

Q = wastewater flow, gal/day;
T = time of detention in septic tank, days;
V = volume of septic tank, gal;
L = length of septic tank, ft;
B = breadth of septic tank, ft; and
D = depth of water in septic tank, ft.

Design constraints are: $T > 1.5$, $V \geq 1500$, $2B \leq L \leq 3B$, $L \geq 12$, $B \geq 4$ and $2.5 \leq D \leq 5$. The latter three constraints were used to develop the L' scale of Fig. 53 based on $L' = 0.0928\,V^{0.5}$ assuming $L' = 2.5B$ and $D = 3.6$.

Several equations are used for the design of leach fields, considered here in the same order as used in design. The relationship between porosity and unit weight of gravel used to fill the leach field trenches based on a specific gravity of 2.65 for gravel particles is:

$$E = 1 - 0.00605U \tag{I-8}$$

where:

U = unit weight of gravel in leach field trenches, lb/ft^3; and
E = porosity of gravel in leach field trenches.

Then the volume of leach field gravel required for one day's flow capacity in the voids is:

$$V = Q' / 7.48E \tag{I-9}$$

where:

V = volume of leach field gravel, ft^3; and
Q′ = maximum daily flow, gal/day.

Next, the equation relating percolation test result to allowable sewage infiltration rate is solved:

$$S = 5\,P^{-0.5} \tag{I-10}$$

where:

S = sewage application rate to trench walls, gal/ft^2-day; and
P = percolation rate of clean water into soil, min/in.

The soil is suitable for a leach field if $5 > P > 30$, with seasonal high groundwater at least 4 ft below the trench bottom. The value of S from Eq I-10 is then used to compute the required trench sidewall area:

$$A = Q'/S \tag{I-11}$$

where:

A = adsorption area of leach field trench walls below pipe invert, ft^2.

From the volume of leach field gravel and the adsorption area of the trench walls, the trench width can be computed:

$$W = \frac{24V}{A} = \frac{24Q'}{7.48EA} = \frac{24SA}{7.48EA} = 3.208\,\frac{S}{E} \tag{I-12}$$

where:

W = width of leach field trenches, in.

Given the adsorption area and the depth of gravel in the trenches, the length of the trench may be computed:

$$F = 6A/G \tag{I-13}$$

where:

F = leach field drainage pipe length, ft; and
G = gravel depth below pipe invert, in.

A dosing siphon is needed if F exceeds 500 ft. The capacity of the dosing siphon chamber is 60-70% of the interior volume of the drain pipe. Since separate drain pipe systems are needed for each 1000 ft of drainage pipe (or part thereof) it can readily be calculated that 4-in. drainage pipe has adequate hydraulic capacity to carry waste to the entire 1000 ft of its length. For 4-in. drain pipe 60-70% of the interior volume is 0.052-0.061 ft^3/ft. This procedure uses 0.06 ft^3/ft.

A series of parallel closed-loop drainage pipes is recommended, spaced at least 6 ft apart, laid level, and arrayed bearing in mind the desirability of keeping the maximum flow path as short as possible, but in any case not *exceeding*

100 ft. Selection of the number of parallel lines and their length based on a spacing of 6 ft can be made by the equation:

$$F = 12(N-1) + NY = 2X + NY \qquad (I\text{-}14)$$

where:

N = number of parallel leach field lines;

X = leach field width, ft; and

Y = leach field length, ft.

Table I-3 outlines the design procedure developed above in Eqs I-5 through I-14, allowing nomographic solution of rest area and RV dump station waste flows in Fig I-1, design of septic tank dimensions in Fig I-2, and design of the leach field (including the dosing siphon chamber if needed) in Fig I-3.

Table I-3 suggests the use of average wastewater flow for the peak month for sizing septic tank-leach field systems. This value can be developed from traffic counts for the peak month as indicated, this being perhaps the most readily available data on traffic density. Sizing of waste disposal facilities on the basis of peak month average waste flow may in some cases provide a small factor of safety against complete filling of a septic tank with sludge over the design pumpout period.

TABLE I-3. HIGHWAY REST AREA AND RV DUMP STATION SEPTIC TANK LEACH FIELD DESIGN EXAMPLE AND WORKSHEET

Phase	No.	Step	Symbol	Variable	Scale	Example	Design	Unit
Estimate	1	Enter	ADT	Vehicles travelling per day (Average for peak month)	d	1,500		veh/day
wastewater	2	Enter	--	Percentage of vehicles travelling that enter rest area	a	20		--
flow from	3	Read	--	Vehicles entering rest area per day	e	300		--
restroom	4	Enter	--	Persons per vehicle	b	2.5		--
(Fig I-1)	5	Read	--	Persons using rest area facilities per day	f	750		--
	6	Enter	--	Gallons of wastewater per person	c	2.0		--
	7	Read	Q_1	Rest area wastewater flow, in gallons per day	g	1,500		gal/day
Estimate	8	Enter	ADT	Vehicles travelling per day (Average for peak month)	k	1,500		veh/day
recreation	9	Enter	--	RVs travelling as a percentage of total vehicles travelling	h	40		--
vehicle	10	Read	--	RVs travelling per day	l	600		--
wastewater	11	Enter	--	Percentage of RVs travelling that enter rest area	i	20		--
flow	12	Read	--	RVs entering rest area per day	m	120		--
(Fig I-1)	13	Enter	--	Percentage of RVs entering rest area that use dump facilities	j	20		--
	14	Read	--	RVs using dump facilities per day	n	24		--
	15	Enter	--	Gallons of RV wastewater discharged per dump	p	21		--
	16	Read	Q_2	RV wastewater flow, in gallons per day	o	500		gal/day
Design	17	Enter	Q_1	Rest area wastewater flow, in gallons per day (Peak month average)	A	1,500		gal/day
septic	18	Enter	T_1	Time of detention of rest area wastewater in septic tank, in days	B	2		days
tank	19	Read	V_1	Volume of septic tank for restroom wastewater, in gallons	C	3,000		gal
volume								
(Fig I-2)	20	Enter	Q_2	RV wastewater flow, in gallons per day (Peak month average)	A	500		gal/day
	21	Enter	T_2	Time of detention of RV wastewater in septic tank, in days	B	6		days
	22	Read	V_2	Volume of septic tank for RV wastewater, in gallons	C	3,000		gal
	23	Compute	$Q = Q_1 + Q_2$	Rest area plus RV wastewater flow in gallons per day (Peak month average) . .		2,000		gal/day
	24	Compute	$V = V_1 + V_2$	Volume of septic tank for rest area plus RV wastewater, in gallons		6,000		gal
	25	Enter	Q	Rest area plus RV wastewater flow, in gallons per day (Peak month average) .	A	2,000		gal/day
	26	Enter	V	Volume of septic tank for rest area plus RV wastewater, in gallons	C	6,000		gal
	27	Read	T	Time of detention of rest area plus RV wastewater in septic tank, in days . .	B	3		days
Design	28	Enter	V	Volume of septic tank for rest area plus RV wastewater, in gallons	C	6,000		gal
septic	29	Read	L'	Approximate length of septic tank; in feet	D	23.6		ft
tank	30	Select	L	Length of septic tank, in feet.	E	25		ft
dimensions	31	Intercept	--	Turning line.	H	--		--
(Fig I-2)	32	Select	D	Depth of water in septic tank, in feet	F	4		ft
	33	Read	1.2D	Depth of septic tank, in feet	G	4.8		ft
	34	Read	B	Breadth of septic tank, in feet	E	8		ft
Design	35a	Enter or	E	Porosity of gravel in leach field trenches	I	--		--
leach	35b	Enter	U	Unit weight of gravel in leach field trenches, in pounds per cubic foot . . .	J	95		lb/ft^3
field	36	Enter	P	Percolation rate of clean water into soil, in minutes per inch	O	12		min/in.
dimentions	37	Read	S	Sewage application rate to trench walls, in gallons per square foot per day .	P	1.44		gal/ft^2-day
(Fig I-3)	38	Read	W	Width of leach field trenches, in inches (Minimum 12 inches)	K	Use 12		in.
	39	Enter	P	Percolation rate of clean water into soil, in minutes per inch	O	12		min/in.
	40	Enter	Q'	Peak wastewater flow rate, in gallons per day	L	2,000		gal/day
	41	Read	A	Adsorption area of leach field trench walls below pipe invert, in square feet	M	1,400		ft^2
	42	Enter	G	Gravel depth below pipe invert, in inches (Minimum 6 inches)	N	12		in.
	43	Read	F	Leach field drainage pipe length, in feet (Maximum 1,000 feet per field) . .	Q	700		ft
	44	Read	C	Capacity of dosing siphon chamber to water level, in cubic feet (if F > 500 ft)	R	42		ft^3
	45	Select	N	Number of leach field lines	S	10		--
	46	Read	Y	Leach field length, in feet	T	59		ft
	47	Compute	X	Leach field width, in feet = (no. of leach field lines -1) x line spacing (ft)	--	9 x 6 = 54		ft

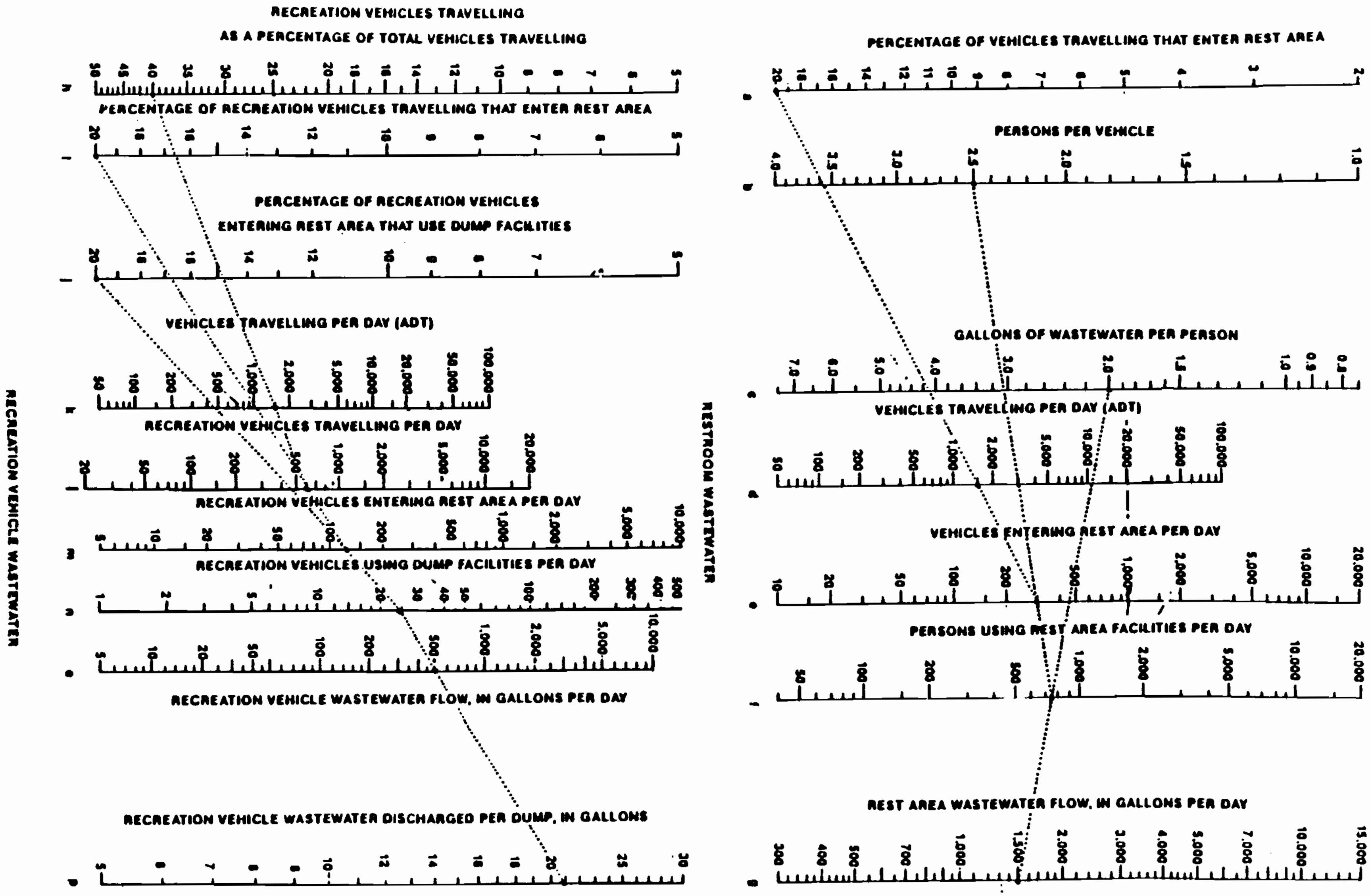


Fig I-1. Nomograph for computing restroom and RV wastewater flow [33].

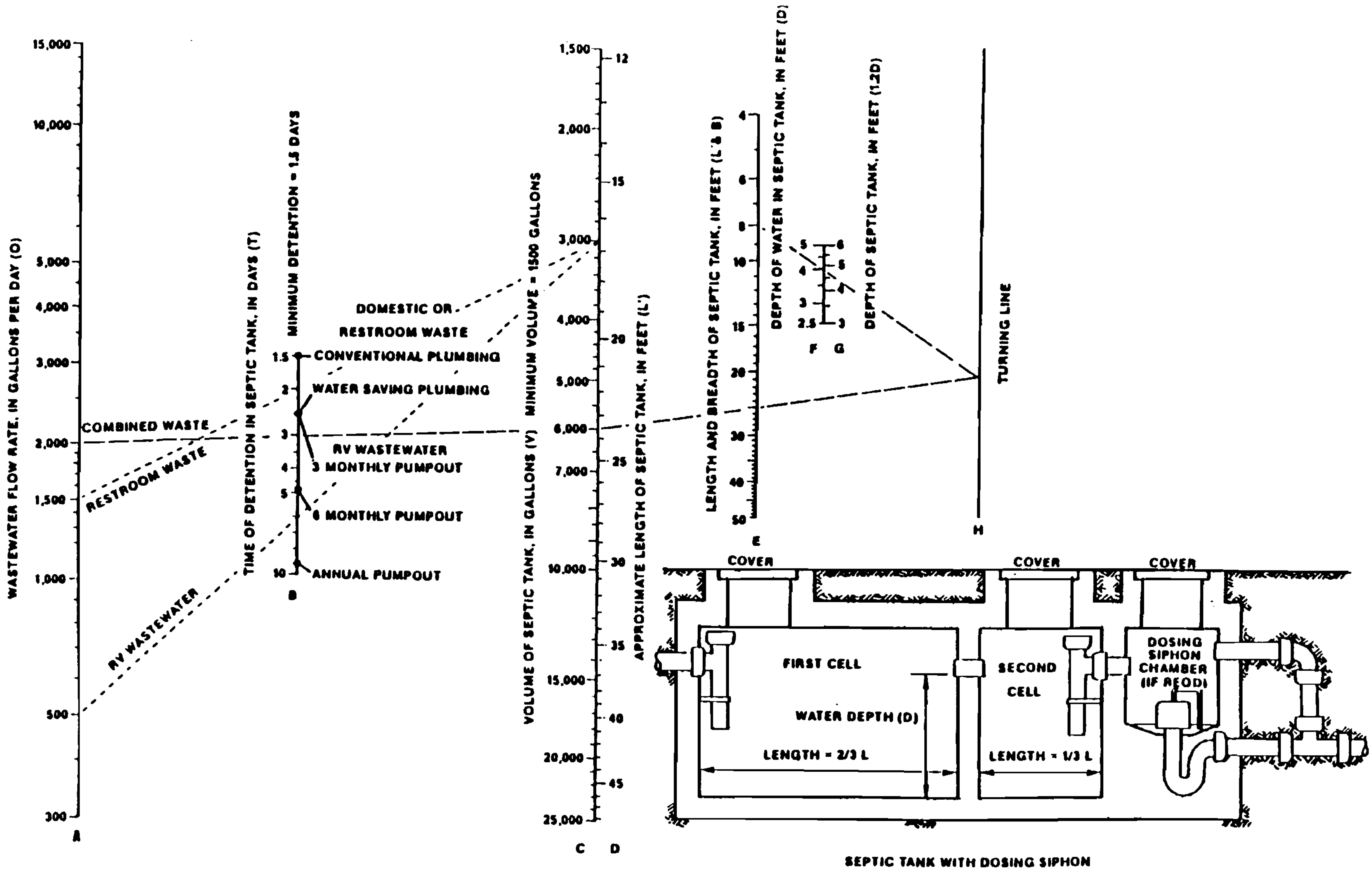


Fig I-2. Nomograph for sizing septic tank [33].

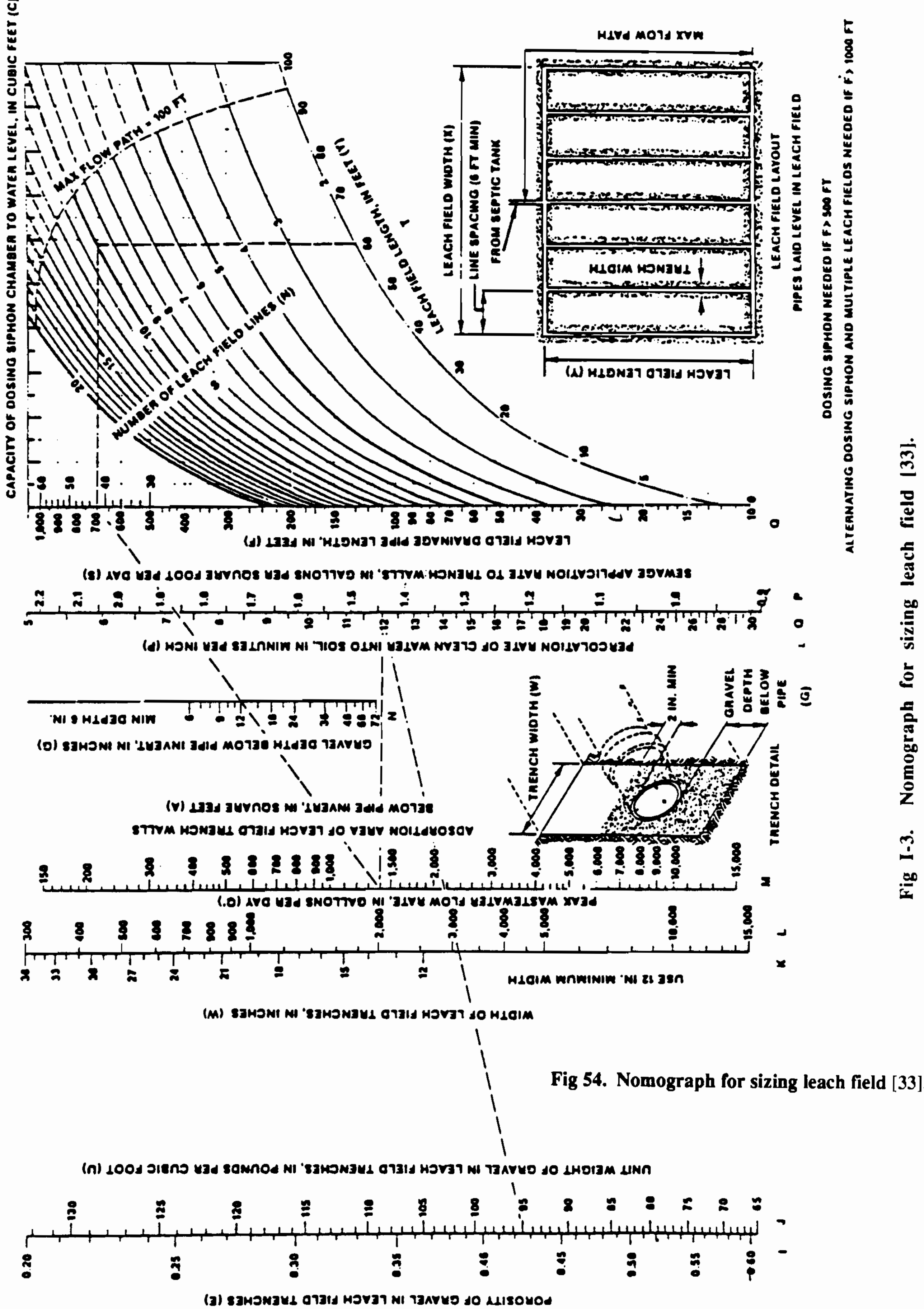


Fig 54. Nomograph for sizing leach field [33].