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WASTEWATER TREATMENT SYSTEMS FOR SAFETY REST AREAS

September 1977

Final Report

Prepared for

FEDERAL HIGHWAY ADMINISTRATION
Offices of Research & Development
Washington, D.C. 20590

FOREWORD

This report presents design guidelines for rest-area wastewater treatment systems that are capable of complying with the requirements of PL 92-500, Federal Water Pollution Control Act Amendments of 1972. Each system is described in detail. This report will be of interest to engineers involved in the design of rest areas.

The final report presents the results of a two-phase study conducted for the Federal Highway Administration, Office of Research, Washington, D.C., at the U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, Environmental Effects Laboratory during the period of June 1974 to July 1977.

Acknowledgment is given to Mr. A. J. Green, Jr., and to Dr. John Harrison for supervision of the study. Special thanks is given to Dr. A. Schindale of Mississippi State University and Mr. E. A. Disque, Consultant to FHWA on Land Use Planning and Design, who were the contributing authors to the final report.

Sufficient copies are being distributed by FHWA Bulletin to provide a minimum of one copy to each FHWA Regional office, FHWA Division office, and each State highway agency. Direct distribution is being made to the Division offices.

Metric equivalents are not provided within the text of this report as this research was initiated before this requirement became operational. Appendix D contains appropriate conversion factors.

Charles F. Scheffey
Director, Office of Research
Federal Highway Administration

NOTICE

NOTICE

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16. Abstract

Design guidelines for rest-area wastewater treatment systems that are capable of complying with the requirements of PL 92-500 are presented. Also presented is guidance for determining wastewater flow and constituent concentration.

Each wastewater treatment system presented (septic tanks, lagoons, extended aeration package plants, trickling filters, rotating biological filters, land treatment systems, intermittent sand filter systems, and disinfection) includes a process description, a discussion of the performance, flexibility, reliability, and operational procedures, preliminary design formulations, and references and selected bibliography. Also included is a flow chart showing a general design procedure for new and existing rest areas with emphasis on water supply needs and wastewater treatment systems.

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PREFACE

The study described in this report is the second phase of a two-phase investigation funded by the Department of Transportation, Federal Highway Administration (FHWA), under Intra-Government Purchase Order No. 4-1-0188. Mr. Byron N. Lord was the FHWA project manager for the Environmental Design and Control Division of the Office of Research.

The study was conducted during the period June 1974 - July 1977 by the Environmental Effects Laboratory (EEL) of the U. S. Army Engineer Waterways Experiment Station (WES), Vicksburg, Mississippi. Messrs. G. W. Hughes, D. E. Averett, and N. R. Francingues, Jr., of EEL were principal authors. The investigation was accomplished under the direct supervision of Mr. A. J. Green, Jr., Chief, Environmental Engineering Division, and under the general supervision of Dr. John Harrison, Chief, EEL.

Contributing authors were Dr. A. Shindala of Mississippi State University and Mr. E. A. Disque, Consultant to FHWA on Land Use Planning and Design.

Directors of the WES during the conduct of this study and preparation of the report were COL G. H. Hilt, CE, and COL J. L. Cannon, CE. Technical Director was Mr. F. R. Brown.

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SAFETY REST AREA SEWAGE TREATMENT MANUAL

1. INTRODUCTION

1-1. BACKGROUND

The advent of the interstate highway system has resulted in an increase in travel by a more mobile American public. Today more motorists are traveling longer distances at a greater frequency than in the past.

To accommodate the highway travelers, the concept of providing safety roadside rest areas* on the Nation's highways was introduced in the Federal Aid Highway Act of 1938 administered at that time by the Secretary of Agriculture. At the present time Federal responsibility rests with the Department of Transportation Federal Highway Administration (DOT FHWA). Originally, a few rest areas were planned and constructed on the major highway networks. These areas became increasingly popular with the motorist, and people began demanding more facilities.

With the request for more rest areas also came the demand for more conveniences. Most major rest areas now furnish drinking water fountains, telephones, solid waste containers, picnic facilities, travel information, and modern rest rooms equipped with flush toilets.

One of the major problems recognized in the construction of a rest area is provision of adequate wastewater treatment and disposal facilities. Many rest-area facilities are located in remote areas and do not lend themselves to connection with municipal wastewater treatment systems. In addition, wastewaters produced at such facilities are subject to high seasonal as well as daily variations in flow and composition. This, in most cases, makes design and operation of waste-treatment systems different from conventional municipal designs. Finally, the difficulty of providing properly trained operating personnel presents a major problem at sophisticated waste-treatment systems that require frequent attention of a skilled operator.

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* Hereafter referred to simply as "rest areas."

In the past, wastewater-treatment and disposal facilities at rest areas received limited attention, and, in many instances, a minimum level of treatment was provided. However, the enactment of the Federal Water Pollution Control Act (FWPCA) Amendments of 1972 Public Law 92-500 and the increasing public concern for environmental quality have resulted in a search for more efficient methods of wastewater-treatment and disposal practices at all rest areas.

The FHWA has clearly stated that efforts will be made to minimize the environmental impact of the highway system. In recognition of this policy, Public Law 92-500, and as part of the ongoing Federally Coordinated Programs of Research and Development in Highway Transportation (FCP), the FHWA has, as one of its objectives, the development of an effective rest-area waste-treatment technology to comply with the 1977 requirements of the FWPCA 1972 Amendments, Public Law (PL) 92-500.

Presently, there are nearly 7700 rest areas being operated and maintained by State highway departments on interstate, primary, and secondary highways. However not all rest areas provide wastewater treatment and disposal facilities. Although it is recognized that many of these facilities are meeting (or are capable of meeting) the 1977 standards, a great many of them will have to be upgraded, modified, or redesigned in order to comply. Therefore, FHWA-funded research is designed to assist the State highway departments by providing information and guidelines to be used in designing or upgrading rest-area wastewater-treatment and disposal systems to comply with future requirements.

1-2. PURPOSE

The objective of this study was to develop recommendations and guidelines for bringing rest-area sanitary wastewater-treatment and disposal systems into compliance with the 1977 requirements of the FWPC Amendments of 1972, PL 92-500. The specific objective of this report is to present final results of the second phase of the study.

1-3. SCOPE

This report presents the requirements of Public Law 92-500 as well as state water-quality standards and effluent limitations. Water usage, wastewater production, and wastewater characteristics at rest areas are discussed with emphasis placed on estimating these parameters for designing new rest areas. The majority of this report presents design guidelines for rest area wastewater treatment facilities. Facilities discussed include septic tank--adsorption field systems, lagoons, extended aeration package plants, rotating biological filters, land treatment, and plastic media trickling filters. A glossary of the terms used is included as a help to the reader.

1-4. COMPANION STUDIES

Companion studies being conducted by Ultra-Systems* relate to cost-effective rest-area components and drinking water sources and treatment. Most State highway departments have been surveyed, and data have been collected which have been prepared on components generally common to Interstate Highway System rest areas. The surveys are generally limited to rest areas having flush toilets and potable water systems in service for one year or more. Criteria and design specifications for rest area components tend to be site specific and vary significantly from state to state. The potable water systems survey contains data relating to water sources, supply and distribution systems, and water-treatment methods and materials. Both surveys report the costs of services, practices, and materials as they affect the operation and maintenance of rest areas.

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* Ultra-Systems, Inc., Irvin, CA, has prepared Report Nos. FHWA-RD-76-62, "Cost-Effective Rest Area Components," FHWA-RD-76-63, "Handbook on Components for Safety Rest Areas," and FHWA-RD-76-103, "Safety Rest Area Water Supply Systems."

2. RESULTS OF PHASE I STUDY

2.1. BACKGROUND

The following sections briefly discuss the results of the Phase I study. For a complete discussion of the Phase I study the reader is referred to the Federal Highway Administration Report FHWA-RD-76-64, "Safety Rest Area Sewage Treatment Methods, State of the Practice, Current Technology, Interim Design Criteria, and Regulations," published 1 December 1975.

2-2. DESIGN

In order to design any wastewater-treatment system the flow and the concentration of the various constituents in the wastewater must be known. In the design of a municipal wastewater-treatment system, total flow is estimated by assuming an equivalent per capita volume for wastewater production. Concentrations of pollutants are estimated from data collected on similar wastewaters in similar situations. In this manner, the total flow and constituent concentrations can be determined and the wastewater-treatment system can be designed accordingly.

The design of a treatment system for rest-area-generated wastewater also requires that the flow and constituent concentrations be known. In a rest-area situation the flow is generated not by a stationary population, as in the case of a municipality, but by a percentage of the traffic using the roadway where the rest area is located. To obtain flow estimates in this situation, it is necessary to determine the number of vehicles using the roadway. An estimate of the percentage of those vehicles that enter the rest area, the average number of occupants per vehicle, and the amount of wastewater produced per user are also needed.

2-3. PUBLISHED LITERATURE

Previously, obtaining the concentrations of rest-area wastewater constituents had been an "art" rather than a science. Until 1971 there were few rest-area water usage, wastewater production, and wastewater characteristics studies. Rest areas were designed using the concentrations of an average domestic wastewater (Table 2-1).

Since 1971 four important studies have been performed on rest-area wastewater and wastewater-treatment facilities.[2-5] Among the parameters studied were the number of vehicles entering a rest area, the number of occupants per vehicle, the wastewater produced either per person or per vehicle, and the concentration of the wastewater constituents.

Sylvester and Seabloom studied wastewater characteristics at four rest areas in Washington.[2] In comparing the characteristics of the rest-area wastewater with those of domestic wastes, the investigators concluded that rest-area wastewater:

a. Had essentially no grease or scum materials.

b. Was high in nitrogen, indicating a preponderance of urine.

c. Contained suspended solids (SS) and 5-day biochemical oxygen demand (BOD_5) between a weak and average domestic sewage.

d. Had a chemical oxygen demand (COD) of a strong sewage due to paper content.

e. Had a phosphate content corresponding to weak sewage.

f. Evidenced settleable solids much greater than domestic sewage due to high paper content.

Also from this study, Sylvester and Seabloom determined that the amount of wastewater produced per user varied from site to site, and there was a large fluctuation in the flow rate from hour to hour and from day to day. An assumed flow of 3.5 gal/cap/day (13.2 ℓ/cap/day) and 0.0048 lb (2.2g) of BOD/capita were used for sizing different types of wastewater-treatment systems applicable in Washington.

In a similar study, Etzel et al. sampled seven rest areas in Indiana.[3] Analysis of influent and effluent samples led Etzel to conclude that "the plants are for the most part substantially underloaded (hydraulically) and accordingly BOD loadings are low."

As a result of their findings, Etzel et al. proposed a design flow of 5.0 gal/cap/day (18.9 ℓ/cap/day) and a BOD loading of 0.007 to 0.01 lb (3.18 to 4.54g) of BOD/capita. They also recommended that comprehensive traffic data be collected before a rest-area facility is designed.

Table 2-1. Typical Composition of Domestic Wastewater.[1]
(All values except settleable solids are expressed in mg/ℓ)

Constituent	Concentration Strong	Medium	Weak
Solids, total	1200	700	350
Dissolved, total	850	500	250
Fixed	525	300	145
Volatile	325	200	105
Suspended, total	350	200	100
Fixed	75	50	30
Volatile	275	150	70
Settleable solids (mℓ/liter)	20	10	5
Biochemical oxygen demand, 5-day, 20°C (BOD_5-20°)	300	200	100
Total organic carbon (TOC)[a]	300	200	100
Chemical oxygen demand (COD)	1000	500	250
Nitrogen (total as N)	85	40	20
Organic	35	15	8
Free ammonia	50	25	12
Nitrites	0	0	0
Nitrates	0	0	0
Phosphorus (total as P)	20	10	6
Organic	5	3	2
Inorganic	15	7	4
Chlorides[b]	100	50	30
Alkalinity (as $CaCO_3$)	200	100	50
Grease	150	100	50

[a]Values reported in Reference 1; however TOC cannot be equal to BOD_5 unless all of the organic carbon is completely biodegradable.
[b]Value should be increased by amount in carriage water.

Pfeffer conducted a study on rest-area characteristics in Illinois and Iowa.[4] Pfeffer, in comparing the results of his study with those of the previous two, concluded that "the range of average BOD_5 for the various rest areas is from 110 to 204 mg/ℓ (average 150 mg/ℓ). The suspended solids range from 56 to 230 mg/ℓ with an average of 149 mg/ℓ. These data suggested that rest-area wastewater is comparable with normal municipal waste."

Pfeffer also conducted a mail survey to determine design assumptions in various states. He recommended that 12 percent of the highway traffic be assumed as entering a rest area. He also recommended that an occupancy of 3.1 persons per vehicle and a wastewater production of 5 gal/cap/day (18.9 ℓ/cap/day) could be used as design values for rest-area treatment facilities. This hydraulic flow rate yields an organic loading of 0.0063 lb (2.86 g) of BOD/capita.

Zaltzman et al., at West Virginia University (WVU), conducted an extensive study of rest areas for the FHWA.[5] Various rest-area parameters were monitored in Florida, Tennessee, New Hampshire, Colorado, and Iowa.

The most important parameter, according to Zaltzman, was the accurate forecasting of average daily traffic (ADT) and the percentage of ADT stopping at the rest area. After monitoring traffic approaching and entering rest areas, regression models were developed to predict the ADT entering the rest areas surveyed.

Zaltzman et al. also sampled the rest-area wastewater. The results are shown in Table 2-2. In analyzing the data, Zaltzman said the wastewater produced corresponds to a weak to medium strength domestic wastewater with respect to BOD, COD, SS, and pH. Nitrogen and phosphorous concentrations often exceed those of strong domestic wastewater.

There were several basic assumptions made by WVU in the interpretation of their data. The first was that a weekend consisted of three days: Friday, Saturday, and Sunday instead of the normal weekend days of Saturday and Sunday. This was felt to be a better representation of the sustained peak flow conditions that exist at rest areas with the majority of usage occurring over this newly defined weekend. The second

Table 2-2. Wastewater Strengths at Various Rest Areas.[5]

State	Strength	Parameter			
		BOD mg/ℓ	COD mg/ℓ	SS mg/ℓ	pH
Colorado	High	156	507	504	8.3
	Low	23	145	72	7.8
	Mean	78	203	208	8.0
	Standard deviation	45	103	118	0.14
Florida	High	300	440	530	8.6
	Low	140	216	28	6.8
	Mean	181	342	186	7.4
	Standard deviation	43	60	111	0.55
Iowa	High	561	787	652	8.5
	Low	59	140	38	7.1
	Mean	210	383	224	7.9
	Standard deviation	137	209	153	0.35
New Hampshire	High	330	480	624	8.4
	Low	90	197	1	6.4
	Mean	203	330	208	7.2
	Standard deviation	62	82	165	0.65
Tennessee	High	233	883	310	8.7
	Low	63	160	16	7.1
	Mean	158	362	124	7.7
	Standard deviation	52	174	72	0.45
All areas	Average	166	344	190	7.6

major assumption was that ADT was not a good basis from which to derive wastewater production. Instead, the average of the six peak three-day weekends occurring throughout a year (18 days) on the average of the three peak usage months of a year (90 days) gave a better indication of rest-area usage and resulting wastewater production.

The following discussion is a summary of the results of the WVU regression analysis of the data.

Prediction of the wastewater production rates from the traffic data known is accomplished by multiplying the average of the six peak three-day weekends or the average of the three peak months (called HIWAY 24) by 9 percent. This value is the design number of vehicles entering the rest area and is called REST 24. Multiplying REST 24 by the values given in Table 2-3 (values given were developed from one rest area in each state listed) or by 5.50 gal/vehicle (20.8 ℓ/vehicle) (average weighted value for all rest areas surveyed) yields a design wastewater production rate in gallons per day. This may also be accomplished by multiplying HIWAY 24 by 0.495 (0.09 × 5.50). Rest area water usage and wastewater production may also be arrived at by using actual traffic counts and by simultaneously monitoring the water usage and wastewater production.

Table 2-3. Rest-Area Design Values, Gal/Vehicle[5] from the WVU Study of 1975.

Location of Study Rest Area	Water Usage			Wastewater Production		
	Peak	Average	Minimum	Peak	Average	Minimum
Florida	5.0	4.5	4.0	5.0	4.25	3.5
Tennessee	7.0	4.5	2.5	7.0	4.5	2.5
New Hampshire	7.0	6.5	5.5	6.0	5.75	4.25
Colorado	5.5	4.25	2.25	5.0	4.25	3.0
Iowa	5.5	4.25	2.25	5.5	4.25	2.25

The results of these studies and the data collected by the WES have been incorporated in this report to develop design criteria (see

Section 14) for predicting rest-area wastewater flows and characteristics.

Zaltzman[5] and Pfeffer[4] have shown that in recent years rest areas have been sized as a function of the predicted 20-year ADT for that roadway upon which they are to be located. The predicted 20-year ADT is recommended by the FHWA as well as the American Association of State Highway and Transportation Officials (AASHO) for the design basis for highway systems. As a result, the rest-area wastewater-treatment system was designed based on a predicted 20-year ADT. Various factors were applied to that ADT, such as percent stopping, seasonal correctional factors, number of occupants per vehicle, percent of occupants utilizing the rest rooms, and assumed water usage per person utilizing the rest rooms. Eventually from these manipulations and assumptions, wastewater production rates and wastewater characteristics were determined and a design was formulated.

Because there were so many assumptions associated with predicting wastewater production rates at rest areas a definition of the problem and development of design criteria were needed. WVU began to formulate the design criteria correlating water usage and wastewater production to vehicles rather than per capita. Similarly, by defining the percent of vehicular traffic entering a rest area, water usage and wastewater production were defined as a function of roadway traffic.

2-4. SURVEY OF REST AREAS

To obtain information on rest area site selection criteria, facilities provided at the rest area, water supply sources, rest area water supply treatment, wastewater characteristics and types and sizes of wastewater treatment plants provided so that a base, state-of-the-practice, could be determined to develop guidelines from, it was determined that the WES should contact State highway departments in each of the nine FHWA regions. Because this report deals with wastewater treatment it was decided to concentrate efforts on rest areas providing comfort facilities with flush toilets as opposed to rest areas which provide only picnic facilities and places to safely stop along the highway.

While, at present, there are nearly 7700 rest areas (Table 2-4)

Table 2-4. Number of Rest Areas by State and Highway System, Summer 1975.[6]

State	Rural Highways: Interstate Nontoll	Rural Highways: Remainder of Federal-Aid Primary System	Rural Highways: All Other Highways Except Local Roads	Urban Highways: Urban Extensions of Federal-Aid Primary Systems	Urban Highways: All Other Highways Except Local Roads	Total
Alabama	8	229	40	--	--	277
Alaska	--	35	5	--	--	40
Arizona	41	99	44	5	1	190
Arkansas	20	64	17	--	--	101
California	51	40	26	2	--	119
Colorado	24	25	7	2	--	58
Connecticut	6	24	27	5	6	68
Delaware	--	13	1	1	--	15
Florida	12	169	39	17	--	237
Georgia	27	177	36	9	--	249
Idaho	28	27	6	9	--	70
Illinois	14	188	4	--	--	206
Indiana	28	30	33	3	3	97
Iowa	48	156	--	41	--	245
Kansas	32	107	12	--	--	151
Kentucky	20	69	34	1	--	124
Louisiana	15	52	13	5	--	85
Maine	11	74	56	2	--	143
Maryland	6	55	4	3	--	68
Massachusetts	12	113	13	61	14	213
Michigan	44	163	28	13	--	248
Minnesota	17	102	37	63	7	226
Mississippi	23	88	3	6	--	120
Missouri	28	64	--	2	--	99
Montana	39	66	5	--	--	110
Nebraska	24	101	12	3	--	140
Nevada	13	28	7	1	--	49
New Hampshire	7	144	117	1	--	269
New Jersey	16	4	2	13	--	35
New Mexico	37	47	17	--	--	101
New York	47	221	33	20	--	321
North Carolina	35	63	27	--	--	125
North Dakota	27	29	--	--	--	56
Ohio	63	155	46	2	1	267
Oklahoma	27	120	29	1	--	177
Oregon	36	35	8	1	--	80
Pennsylvania	57	42	2	--	--	101
Rhode Island	3	11	8	6	6	34
South Carolina	43	93	17	3	--	156
South Dakota	18	54	5	2	--	79
Tennessee	48	284	2	22	--	356
Texas	149	640	283	9	1	1082
Utah	12	12	--	3	--	27
Vermont	28	29	14	--	--	71
Virginia	10	44	4	1	--	59
Washington	26	22	5	1	--	54
West Virginia	7	76	10	6	--	99
Wisconsin	19	206	66	7	2	300
Wyoming	53	46	2	--	--	101
Puerto Rico	--	1	--	--	--	1
Totals	1359	4736	1206	352	41	7694

throughout the United States, only 16 percent of these have toilet facilities. Privies are used in 23 percent of the rest areas but the remainder of the areas (61 percent) provide no rest room facilities. However, it should be noted that 60 percent of the rest areas along the Interstate Highway system provide flush toilets.[6]

The WES team visited states in each of the nine FHWA regions (Figure 2-1). The following factors served as the basis for selection of the states included in the Phase I Study.

1. A sufficient number of states would be chosen to generate an adequate data base.

2. The data base would be representative of each region and would yield a cross section of conditions across the country.

3. States considered must have a well-developed rest-area program. Puerto Rico, Delaware, the District of Columbia, Hawaii, and Alaska were eliminated from consideration because they contain no Interstate Highway system rest areas.

The 21 states selected are shown by the crosshatched areas in Figure 2-1.

Each state visit included an onsite survey of at least one rest area and a meeting to obtain data for the information summary. Meetings were held with FHWA Division Office personnel and State highway department personnel responsible for design, construction, and maintenance of rest areas. In some cases, the meetings were also attended by members of the State health agency and pollution control or regulatory agency responsible for issuing permits for (and, in some instances, the testing of) sewage-treatment facilities at rest areas.

2-5. DATA ANALYSIS

One of the main objectives of the field visits was to determine the types of wastewater-treatment systems in use at rest areas in order to evaluate their ability to meet the 1977 requirements of PL 92-500. (Requirements of PL 92-500 are discussed in Section 3 of this report.) The data were plotted on maps of geographical location, climate, soil type, geologic formation, soil moisture content, precipitation,

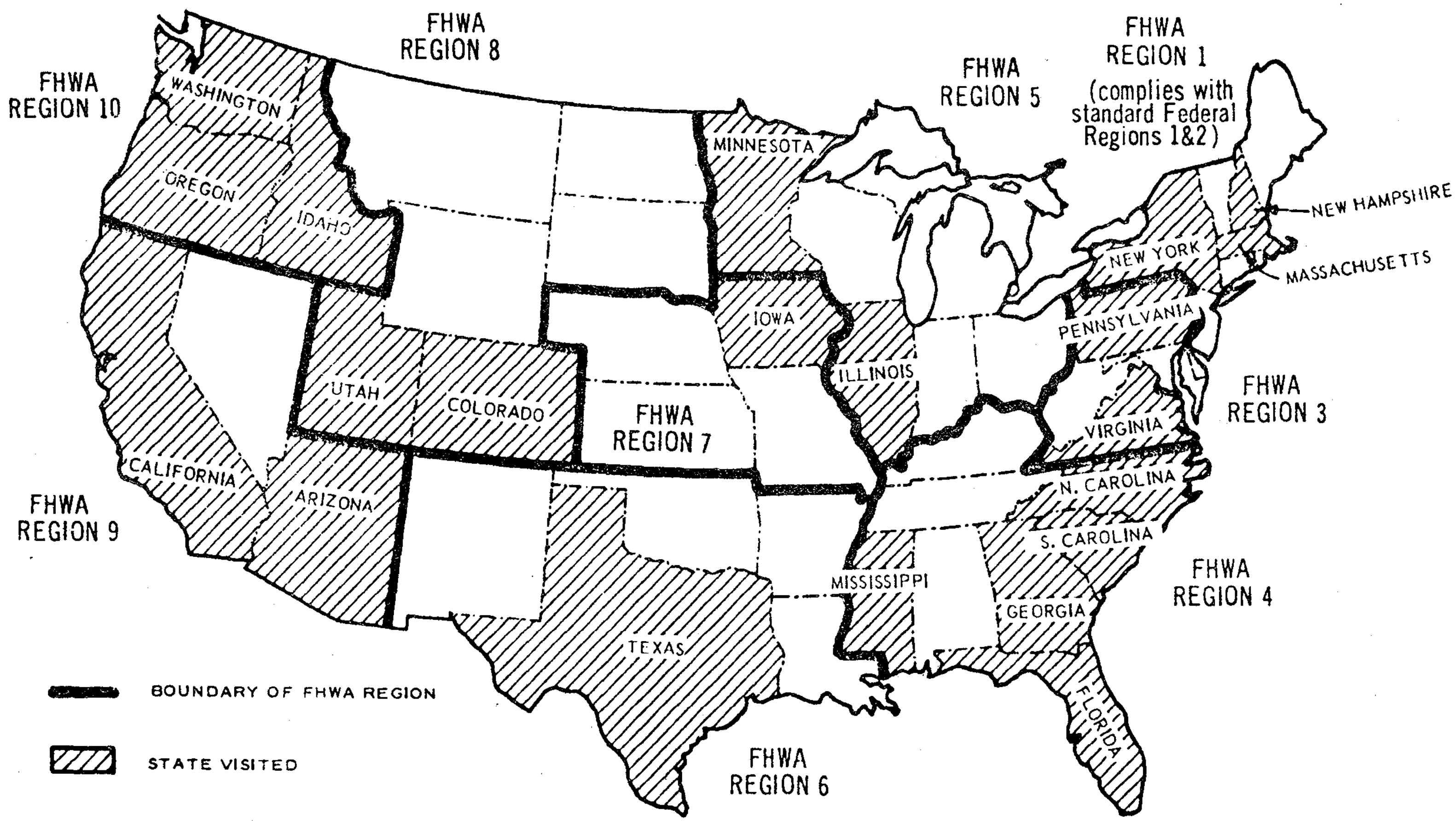

Figure 2-1. States visited by the WES Survey Team.

evaporation, and normal annual temperature. The only relationship of treatment method to any of those parameters was the predominant use of evaporative lagoons in regions of low precipitation and high evaporation in the Midwest and the West. The reader is referred to Report No. FHWA-RD-76-64, Appendix B for a detailed account of each FHWA region.

The principal types of rest-area wastewater-treatment and disposal systems are: septic tanks followed by (either) leach fields or sand filters; extended aeration activated sludge package plants; chemical vault holding tanks; discharge into a municipal wastewater-treatment system; and use of facultative, aerobic, or totally evaporative lagoons.

Table 2-5 shows the results of the data gathered by the WES on the

Table 2-5. Wastewater Treatment Types by FHWA Region Based on WES Survey Information Collected in Summer 1974.

Type of Wastewater Treatment	FHWA Region, No. Each Type									Total of Facilities Reported
	1	3	4	5	6	7	8	9	10	
Septic tank-leach field	11	10	16	11	24	--	11	67	30	180
Lagoon (aerobic, facultative, or evaporative)	--	2	10[a]	2	--	1	12	4	1	22
Extended aeration package plant[b]	--	23	72	--	10	--	6	5	--	116
Chemical vault holding tank	--	--	--	1	--	--	3	15	18	37
Discharge to a municipality	1	13	1	2	1	--	4	3	--	25
Recirculation-incineration	1	--	--	--	1	--	1	--	1	4
Physical-chemical	--	6	--	--	--	--	--	--	1	7
Septic tank-sand filter	1	26	4	--	--	--	--	--	1	31
Total of facilities for FHWA Region	14	80	93	16	36	1	37	94	51	422

Note: This does not represent all facilities, only those reported. In some cases it does not represent all facilities in state interviewed.

[a] Used in conjunction with EA plants, does not show in totals.

[b] Some EA plants are supplemented with leach fields, sand filters, Or spray irrigation.

number of each type of wastewater-treatment system being used in each FHWA region. In Region 9, there were 10 instances where facultative ponds were used as the tertiary treatment method after the employment of extended aeration activated sludge package plants. In Regions 3, 4, 6, and 10, spray irrigation was used as a tertiary treatment either after the use of lagoons or after the use of package plants.

While it is apparent from Table 2-5 that discharge to a municipality is not the most common disposal method used throughout the FHWA regions, it has been stated by most of the states visited that it is the most desirable method. Discharge to a municipality is desirable for four reasons: (1) a National Pollutant Discharge Elimination System (NPDES) permit is not required; (2) the State highway department is relieved of the responsibility of designing and constructing onsite wastewater-treatment and disposal systems, but not of designing and constructing a transport system; (3) operation and maintenance costs are reduced; and (4) the obligation of the state to provide a means of wastewater treatment and disposal of wastewaters generated at rest areas is fulfilled.

2.6. REFERENCES

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

6. "Interstate Safety Rest Area Status Report," Response to FHWA Memorandum of 9-13-73.

3. REQUIREMENTS OF PUBLIC LAW 92-500

3-1. INTRODUCTION

The 1972 Amendments to the Federal Water Pollution Control Act (FWPCA), PL 92-500, set forth effluent limitations for publicly owned wastewater-treatment systems. In particular, Section 301(b) requires that by 1 July 1977 all point sources from publicly owned treatment works produce an effluent that reflects the application of secondary treatment and that does not violate applicable water-quality standards for the receiving stream.

To emphasize application of this act to highway rest areas, FHWA regulations[1] published 25 September 1974 included the following statement:

> "It is the policy of the Federal Highway Administration that on-site sewage treatment facilities shall be designed, constructed, and operated to meet the 1977 effluent limitations pursuant to PL 92-500 or State standards, whichever is more restrictive, and water quality standards for receiving the water."

The effluent limitations and water quality standards of PL 92-500 and state effluent limitations will be briefly discussed in this section. Detailed discussion of the regulations and permit procedures of the National Pollutant Discharge Elimination System (NPDES), established by Section 402 of PL 92-500, are included in Appendix A. Particular emphasis will be given to the permit requirements that are important in the design and operation of wastewater-treatment facilities.

PL 92-500 limitations that reflect secondary treatment are given in Table 3-1. These limits establish the minimum national standard for secondary treatment that must be achieved in all states. However, state water-quality standards and state effluent limitations or regulations may require a higher degree of treatment for wastewater discharges from rest areas than that defined as secondary.

The Environmental Protection Agency (EPA) has included in the September 2, 1976 copy of the Federal Register a proposed rule change to PL 92-500. This proposed rule change would allow the use of lagoons

Table 3-1. Secondary Treatment Requirements of PL 92-500.[2]

Parameter	30-Day Mean	7-Day Mean
Biochemical oxygen demand (5-day) (arithmetic mean)		
Influent $\geq$ 200 mg/ℓ	30 mg/ℓ	45 mg/ℓ
Influent $\leq$ 200 mg/ℓ	15% of influent	45 mg/ℓ
Suspended solids (arithmetic mean)		
Influent $\geq$ 200 mg/ℓ	30 mg/ℓ	45 mg/ℓ
Influent $\leq$ 200 mg/ℓ	15% of influent	45 mg/ℓ
pH of effluent	$\geq$ 6.0 , $\leq$ 9.0	$\geq$ 6.0 , $\leq$ 9.0

NOTES: (1) These requirements represent the minimum effluent standards that must be achieved by 1977 by publicly owned facilities.

(2) The pH limitation is applicable only where chemical addition is used for wastewater treatment and/or where industrial sources affect the pH of the discharge.

as the sole means of achieving secondary treatment if the 7-day and 30-day BOD requirements were met and the suspended solids value in the effluent "...is equal to the effluent concentration achieved 90 percent of the time within a state or appropriate contiguous geographical area by waste stabilization ponds that are achieving the levels of effluent quality established for biochemical oxygen demand."[3]

3-2. 1983 REQUIREMENTS OF PL 92-500

PL 92-500 also requires that publicly owned treatment works provide for the application of best practicable waste-treatment technology by 1 July 1983. A recent Environmental Protection Agency (EPA) report entitled "Alternative Waste Management Techniques for Best Practicable Waste Treatment" was published on 17 October 1975. Included in the EPA report is a chapter describing general criteria for "best practicable treatment." A supplement was also published in the Federal Register on 11 February 1976 and is included as Appendix C of this report. Regulations establishing numerical limitations for best practical treatment have not yet been promulgated.

3-3. STATE WATER-QUALITY STANDARDS AND EFFLUENT LIMITATIONS

Each state has adopted water-quality standards to protect or enhance the water quality of its lakes and streams. A summary of these standards is given in Appendix C of the Phase I report (FHWA-RD-76-64). PL 92-500 requires that all state water-quality standards be upgraded to meet standards established in Federal regulations. A few states have standards that are more restrictive than those of most states; all of the states' standards are not the same due to differences in climate, geography, and the major uses of state waters. Therefore, it is impossible to generalize the standards for all of the states. In the cases of an individual state, there are many details and specifics for water bodies or stream segments that can only be extracted directly from the state regulation; even then it may require an explanation, interpretation, or judgment from the state pollution control agency on a site-specific basis. Since many rest areas are located on small streams, water-quality standards often govern the degree of treatment required.

Table 3-2 lists states that have established effluent limitations either for all domestic wastewater discharges, or for certain receiving waters that are more stringent than the secondary requirements given in Table 3-1. Many states have similar unpublished criteria that are used for guidance in establishing wastewater-treatment requirements necessary to protect the water quality of the receiving streams. <u>Therefore, the State agency should always be consulted prior to planning wastewater-treatment systems at rest areas.</u>

Table 3-2. State Effluent Limitations for Rest Areas that Are More Stringent than PL 92-500.

State	Applicable Discharges	BOD_5 mg/ℓ	TSS mg/ℓ	Total Nitrogen mg/ℓ	Total Phosphorus mg/ℓ
Florida	All domestic wastewater discharges, 90% treatment				
	Advanced wastewater treatment required as deemed necessary by the Department of Pollution Control	5	5	1	1
Illinois	Dilution ratio of receiving stream to effluent less than 1:1	4	5		
	Dilution ratio of receiving stream to effluent less than 5:1	10	12		
	Discharges to Lake Michigan	4	5		1
	Discharges to Fox River Basin if waste load is greater than 1500 population equivalents				1
	Discharges from third-stage-treatment lagoons treating less than 2500 population equivalents exempt from dilution ratio requirements	30	30		
	All other discharges	30	30		
Michigan	Best practicable waste treatment for removal of phosphorus with goal of 1 mg/ℓ as phosphorus				1
Minnesota	Dilution ratio of receiving stream to effluent less than 10:1	5	5		
	Discharges to lakes or reservoirs				1
	All other discharges	25	30		
Missouri	Discharges to lakes and reservoirs	20	20		
	Discharges to "losing streams"	5	10		
	New discharges to wild and scenic rivers prohibited				
North Dakota	All discharges	25	30		

(Continued)

Table 3-2 (Concluded). State Effluent Limitations for Rest Areas that Are More Stringent than PL 92-500.

State	Applicable Discharges	BOD_5 mg/ℓ	TSS mg/ℓ	Total Nitrogen mg/ℓ	Total Phosphorus mg/ℓ
Ohio	Discharges to water-quality-limited receiving waters (most stringent requirement)	10	12	1.5 NH_3-N (Jul-Oct)	1.0
Oregon	Discharges to selected waters	5	5		
	Discharges to selected waters	10	10		
	Discharges to selected waters	20	20		
	Discharges to other waters must provide secondary treatment or meet water-quality standards	30	30		
South Dakota	Discharges to cold-water fisheries	10	10		
Utah	Compliance for all discharges by 30 Jun 1977	25	25		
	Compliance for all discharges by 30 Jun 1980	10	10		
Virginia	Discharges to Potamac River embayments from Jones Pt. to Rt. 301 Bridge[a]	3		1 (unoxi-dized N (Apr-Oct))	0.2
	Discharges to Aquia Creek, provide 100 days storage to eliminate discharge during low flow months or provide nutrient removal[a]				
	Discharges to Chickahominy Watershed above Walker's Dam[a]	6	5	0.2 (NH_3-N)	0.1
	Discharges to Rappahannock River above Salem Church	1	0	1	0.1

[a]For rest areas and other small discharges, a no-discharge system is encouraged.

3-4. REFERENCES

1. Federal Register, "Water Supply and Sewage Treatment at Safety Rest Areas," Vol 39, No. 187, September 1974, pp 34366-34400.

2. Federal Register, "Secondary Treatment Information," Vol 38, No. 159, August 1973, pp 22298-22299.

3. Federal Register, "Secondary Treatment Information," Vol 41, No. 172, September 1976, pp 37222-37223.

4. WATER USAGE, WASTEWATER PRODUCTION, AND WASTEWATER CHARACTERISTICS

4-1. GENERAL

At present the majority of the rest areas in use that are equipped with flush toilets employ wastewater-treatment systems that have been hydraulically overdesigned. To define the extent of the overdesigns the FHWA contracted with West Virginia University (WVU) for the collection of data on rest-area traffic, facility usage, water-usage rates, and wastewater flows. The collection and compilation of these data are complete and the results have been published. From the data collected by WVU (Zaltzman et al.) and from the data collected by the WES, design guidelines for use in predicting water-usage rates, wastewater production rates, and wastewater characteristics have been developed.

The following rest-area design guidelines are based on information collected from six rest areas. If more detailed information is available for a given state or for a given locale, or if the amount of wastewater generated at a rest area is already known, then that information should be used. However, if no data are available within a given state or region, then the formulations presented in this section may be helpful in determining the size of a rest-area wastewater-treatment system.

It must be pointed out that the methods currently in use for predicting water usage and wastewater production have resulted in the oversizing of water supply systems and wastewater-treatment systems. Thus it is felt that instead of assuming an occupancy of 3.1 persons per vehicle and a water usage and wastewater production of approximately 5 gal per person as has previously been used by many states for design purposes the following method will yield a more accurate prediction of what is expected at a rest area.

4-2. WATER USAGE

Total water usage at a rest area is the sum of the water used in rest-room fixtures (lavatories, toilets, urinals, sinks) and cleanup of rest rooms, water supplied to drinking fountains, water supplied for cleanup and refill at trailer dumping station, and, in some instances, water used for landscape irrigation and cleanup of service vehicles. Because a rest-area water supply is available for such divergent uses the water demand at rest areas varies considerably from site to site.

Another factor influencing rest-area water usage is climate. In some instances, as demonstrated by the data collected by WVU in Florida, there is very little variation in weather throughout the year, the demand for water remains high and is a function of rest-area traffic, which may be computed directly from highway traffic data. In other instances, as in the New Hampshire data, large seasonal fluctuations in traffic are evidenced and reflected in highly variable water demand. In this situation the use of average daily water demand as a design value may prove inadequate for periods of high sustained use. The design approach formulated by Zaltzman et al. based on the traffic data for the three peak months of the year enables the design engineer to size a water supply system that will prove adequate for the entire year. This approach will allow for designing a system that will handle heavy demands without being overdesigned.

Other factors influencing the water demand at a rest area are the site specific conditions at each location. Such factors may include the proximity of the rest area to large urban centers and recreational areas, speed limits, the size of the rest area, the volume of highway traffic, and proximity to two previous rest areas. Because these factors may influence water demand at a rest area, design of a water supply should reflect good engineering judgment when determining the final water supply requirements.

When designing the water supply, if the water-usage rate at a rest area or an analagous site is known then this rate should be used in sizing the water supply system. If the water usage rate at an analagous rest area is not known it may be possible to monitor the rest area to determine this value. Monitoring of the rest area is accomplished by counting the traffic entering the rest area and metering (with a water meter) the total water used in the rest area. Both studies must be performed simultaneously, in this manner the average water usage per vehicle entering the rest area may be determined. If, however, there is little or no water-usage information available and none can be obtained then the following method and values developed by Zaltzman et al.[1] may be used.

The following method is based on average design rest-area traffic (REST 24). REST 24 is determined by multiplying the average 24-hour traffic of the six peak three-day weekends (Friday, Saturday, and Sunday) of the year (18 days) or the three peak months of the year (90 days) by 9 percent to determine the number entering the rest area. The value of 9 percent is a weighted average arrived at from data obtained for FHWA by Zaltzman et al. If the design engineer knows that more or less of the roadway traffic will stop at his rest area then it is recommended that he should adjust the 9 percent accordingly. If, however, the design engineer does not know the expected percent stopping then he should use the 9 percent figure for design purposes. A note should be made that if a rapid growth is expected in highway traffic then this factor must also be included in obtaining REST 24; i.e., if the highway traffic is expected to increase 300 percent in the next 20 years than the amount of vehicles entering the rest area must also be expected to increase 300 percent. In such an instance it may be necessary to construct the rest area in phases.

Expected water usage is arrived at by multiplying the rest area traffic (REST 24) by the expected average water usage per vehicle (6.7 gallons/vehicle based on the data obtained for FHWA by Zaltzman et al.). This value is called WATER 24 and it is the design daily water usage for the rest area. This is the sustained water usage that may be used for designing the rest area water supply system.

Zaltzman et al. have shown that two thirds (67%) of the average daily water usage occurs between the hours of 8 A.M. and 4 P.M., an 8-hour period. Thus the design engineer may wish to calculate this value (called WATER 8) to use in designing his water supply system. WATER 8 may be calculated by multiplying WATER 24 by 0.67 or by multiplying REST 24 by 4.5 gallons per vehicle. This is the peak 8-hour sustained flow that must be available at the rest area.

The peak instantaneous water demand that will occur at a rest area is a function of the fixtures at that rest area. To predict this value Zaltzman et al. first determined the peak volume of traffic that might enter the rest area. This value was called PK VOL 1 and may be calculated by multiplying REST 24 by 0.16 (based on the data collected by Zaltzman et al.). Peak one hour water demand is then calculated by multiplying PK VOL 1 by 6.7 gallons/vehicle. This value is called WATER 1 and is the peak one-hour period of flow. WATER 1 may then be used by the design engineer to determine the maximum size of the water pumps that are required or both WATER 1 and WATER 8 may be used to design a water storage facility if it is determined that one is desirable or necessary.

When the location of a rest area is not fixed by distance or predetermined by other constraints, the location of an adequate water supply may become the determining factor in rest-area site selection. In such instances where an adequate source of potable water is available throughout a stretch of highway, rest-area site selection may become independent of water needs.

Once the total daily water demand for a rest area has been determined the water supply system can be located. If a supply of water is located at the rest area but it will not provide a sufficient supply to meet the design demand water usage, water conservation measures may have to be instituted. Flow restrictions or flow reduction fixtures and plumbing are among the conservation methods which may be used. Another alternative would be to pump water from some other site where an adequate supply of potable water exists. This decision must be based on the economics of a distribution system versus recycle, hauling, etc.

The reduced water demand (RWD) through the use of flow reduction fixtures may be computed by multiplying the daily water demand (WD), determined by the ratio of the nonstandard fixtures (NSF) to standard fixtures (SF). The formulation follows:

$$\text{RWD} = \text{WD} \times \frac{\text{NSF}}{\text{SF}}$$

When nonstandard fixtures are used then the reduction of water for each fixture must be taken into account. That is, if flow reduction fixtures are used only on the water closets, urinals, and lavatories but not on outside drinking fountains and hose bibs then total RWP must be determined by monitoring each separate facility (hose bibs, urinals, etc.) to determine the net flow reduction achieved for the entire facility.

Another method of reducing total water demand may be the use of a recycle system for flushing of water closets and urinals.[2] Again it must be pointed out that the effect of using such a system on the total water demand must be determined by monitoring each water closet and urinal. Therefore, the use of either nonstandard fixtures or recycle will reduce the total water demand of a rest area facility but determining how much the demand will be reduced can only be accomplished through field monitoring of the rest area. It should be pointed out that the use of NSF may increase the constituent concentrations in the wastewater (expressed as mg/ℓ) but will not reduce the total constituent loads to the wastewater-treatment plant (expressed as pounds per day).

4-3. WASTEWATER PRODUCTION RATES

Total wastewater production at a rest area is the sum of the water used for flushing of urinals and toilets, water used in lavatories, and water used for cleaning the rest rooms, lobby areas, etc. It may also include the water from drinking fountains that has not been consumed and the wastewater from the trailer dump stations including trailer wastes and washdown water. As with water usage, wastewater production will vary from site to site and is affected by many of the factors that affect water usage. However, in each instance wastewater production can be directly related to highway traffic.

Once a rest-area location has been established and an adequate supply of potable water located, the rest-area designer can estimate a daily volume of wastewater to be treated. If the wastewater production rate at a nearby rest area with similar characteristics is known then this value can be used for design. The designer may also wish to measure flows at a nearby rest area to determine a design value.

Wastewater flow may be determined by monitoring just the water used at the comfort building with water meters and equating this to wastewater production or by measuring the wastewater flow as it proceeds down the sewer line to the wastewater treatment facility. If, however, no data are available to the design engineer and none can be obtained then the following method may be used in determining rest area wastewater production.

Wastewater production at a rest area may be determined through use of the weighted average design value of 5.5 gal/veh (developed by Zaltzman et al.) multiplied by REST 24. This will yield the expected average daily wastewater flow for the rest area.

Zaltzman has pointed out that, at a rest area, two-thirds of the wastewater flow will be generated in the 8-hr period from 8 a.m. to 4 p.m. (see Figure 4-1). Thus, on an hourly basis, four times as much

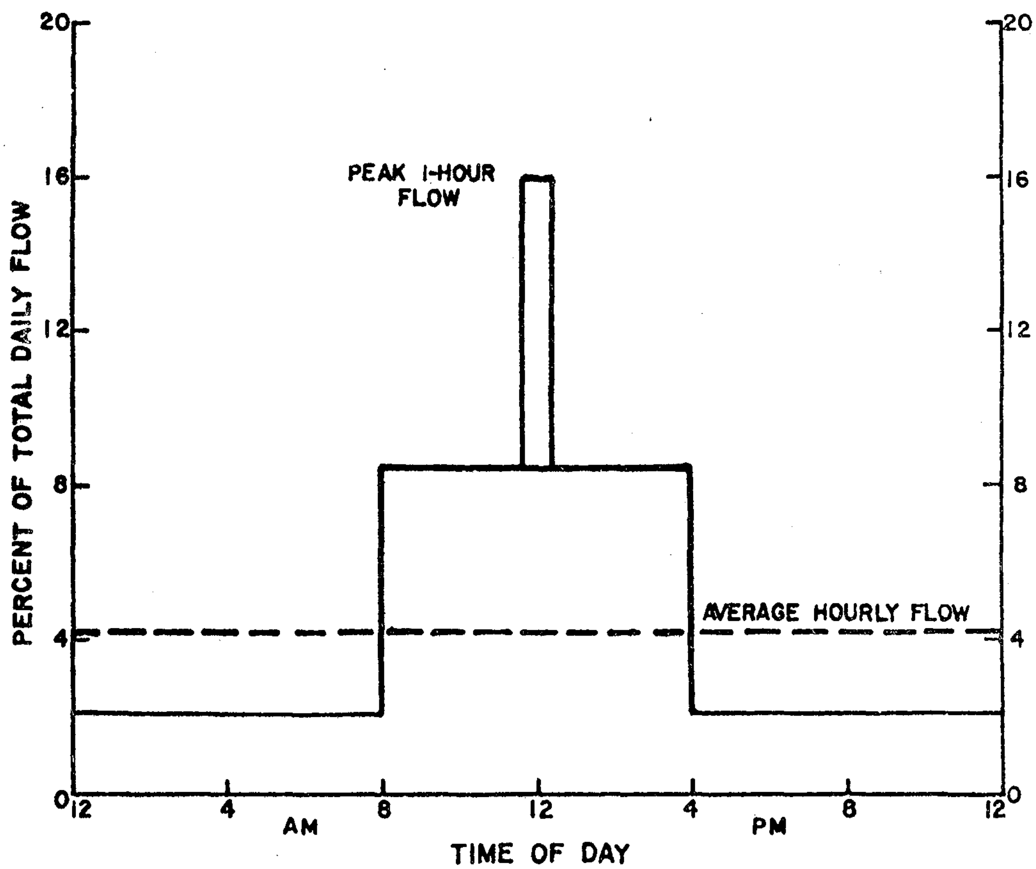


Figure 4-1. Hourly wastewater flow generation.

wastewater is produced between 8 a.m. and 4 p.m. than is produced between 4 p.m. and 8 a.m. Therefore, the designer of a rest-area wastewater-treatment system must design not only for the average daily wastewater production rate but must check the effects of seasonal daily and hourly flow variations on the operation characteristics of the plant.

4-4. WASTEWATER CHARACTERISTICS

In the design of a rest-area wastewater-treatment system not only must the expected wastewater production rate in gallons per day be determined but also the concentration of the wastewater constituents. in particular, the primary design parameters, BOD_5 and SS.

A summary of the rest-area wastewater characteristics found in the published literature and from the WES field study is given in Table 4-1. From these data it is seen that the average BOD_5 ranges from 78 to 210 mg/ℓ and the average SS concentration ranges from 124 to 224 mg/ℓ. As expected, analysis of the available data reveals that the wastewater characteristics will vary hourly, daily, seasonally, and from site to site in much the same way as the wastewater flows.

Rest-area wastewater-treatment systems have been designed using the constituent concentrations found in typical domestic wastewater of medium strength (Table 2-1). However, from the data shown in Table 4-1 it is seen that most rest-area wastewaters are characterized by a lower BOD_5 and a lower SS than domestic wastewater. A more representative basis on which to design a rest-area wastewater-treatment system would be to assume that the wastewater had an average BOD_5 ranging from 125 to 175 mg/ℓ and an average SS in the 125 to 200 mg/ℓ range.

Since many treatment systems are checked for both hydrualic loading rate and organic (BOD_5) loading rate, it is necessary to determine both flow in gallons per day and expected BOD_5 in pounds per day. To obtain pounds of BOD_5 per day, use the following equation or determine directly from Figure 4-2.

$$BOD_5(\text{lb/day}) = Q \times Ci \times 8.34 \times 10^{-6} \qquad (4\text{-}1\text{-}1)$$

Table 4-1. Wastewater Strengths at Various Rest Areas.

State	Strength	Parameter BOD$_5$ mg/ℓ	COD mg/ℓ	SS mg/ℓ	pH
Colorado[a]	High	156	507	504	8.3
	Low	23	145	72	7.8
	Mean	78	203	208	
	Standard deviation	45	103	118	
Florida[a]	High	300	440	530	8.6
	Low	140	216	28	6.8
	Mean	181	342	186	
	Standard deviation	43	60	111	
Iowa[a]	High	561	787	652	8.5
	Low	59	140	38	7.1
	Mean	210	383	224	
	Standard deviation	137	209	153	
New Hampshire[a]	High	330	480	684	8.4
	Low	90	197	1	6.4
	Mean	203	330	208	
	Standard deviation	62	82	165	
Tennessee[a]	High	223	883	310	8.7
	Low	63	160	16	7.1
	Mean	158	362	124	
	Standard deviation	52	174	72	
Mississippi[b]	High	432	979[c]	839	6.7
	Low	12	225	4	9.1
	Mean	124	563	140	
	Standard deviation	86		145	

[a] Data collected by Zaltzman et al.

[b] Data collected by WES.

[c] COD data for Mississippi collected for 7 days only; do not reflect the BOD$_5$ and SS data which were for 43 days.

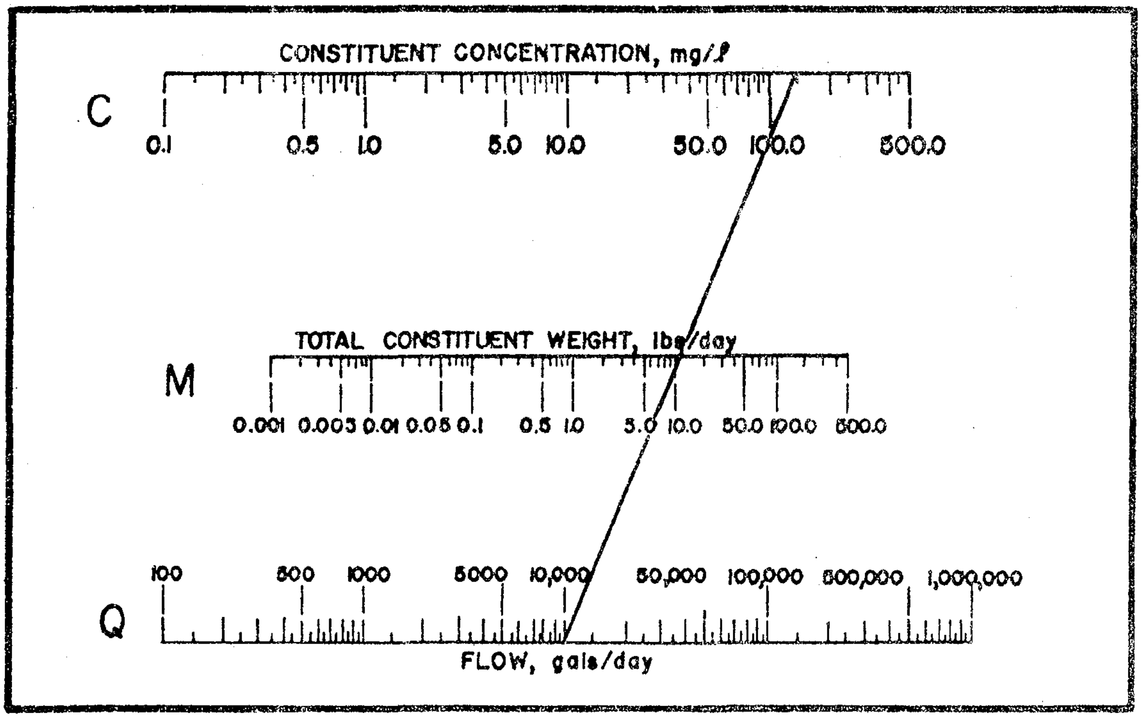


Figure 4-2. Nomograph for use in determining flow and BOD_5 per day.

where

BOD_5 = 5-day biochemical oxygen demand

Q = wastewater flow, gal/day

Ci = constituent concentration, BOD_5, mg/ℓ

8.34×10^{-6} = conversion factor

Or

$$BOD_5(\text{kg/day}) = BOD_5(\text{lb/day}) \times 0.45359$$

where

0.45359 = conversion factor

Total constituent weight (M) in pounds per day is determined in Figure 4-2 by drawing a straight line from the known constituent concentration (C) in mg/ℓ to the known flow (Q) in gallons per day. Where this line crosses the M scale this value is the total constituent weight in pounds per day. An example of this is shown in Figure 4-2 where constituent concentration (C) is 150 mg/ℓ and flow (Q) is 10,000 gallons per day. By connecting these points with a straight line it is seen that total constituent weight (M) is 12.5 pounds per day.

4-5. EXAMPLE CALCULATIONS

The design engineer is now able to select from three different methods of obtaining predicted water usage, wastewater production, and wastewater characteristics. These three methods are: (1) use the data that are available from an analagous rest area; (2) monitor an analagous rest area to obtain the necessary data; and (3) use the predictive method set forth previously in this section. To correctly use the predictive method, example calculations follow.

a. Traffic data must be collected for a full year on the roadway where the rest area is to be located. The data for the six peak three-day weekends or the three peak months is then selected. In this example the data for the six peak three-day weekends is used. Ranked in order, the six peak three-day weekends are:

	Friday	Saturday	Sunday	Total
1.	11,364	13,426	12,978	37,768
2.	11,027	13,142	13,264	37,433
3.	10,642	12,976	12,718	36,336
4.	9,267	13,179	12,653	35,099
5.	10,117	12,349	12,144	34,610
6.	9,870	11,957	11,643	33,470
			TOTAL	214,716

b. Calculate the design daily traffic (HIWAY 24) of the six peak three-day weekends.

$$\text{HIWAY 24} = \frac{214{,}716 \text{ vehicles}}{18 \text{ days}}$$

HIWAY 24 = 11,929 veh/day

c. Calculate the design average 24-hr rest area traffic (REST 24).

REST 24 = 0.09 × HIWAY 24

REST 24 = 0.09 × 11,929 veh/day

REST 24 = 1074 veh/day

d. Calculate the design average 8-hr rest area traffic (REST 8).

This value may be used for computing WATER 8 and it may be used for sizing of parking facilities.

REST 8 = 0.67 × REST 24

REST 8 = 0.67 × 1074 veh/day

REST 8 = 720 vehicles/8 hours

e. Calculate the peak 1-hr rest area traffic (PK VOL 1). This value may be used for computing WATER 1 and for sizing the parking facilities.

PK VOL 1 = 0.15 × REST 24

PK VOL 1 = 0.15 × 1074 veh/day

PK VOL 1 = 161 vehicles/1 hour

f. Water supply requirements may now be obtained from the rest area traffic; calculate the daily water requirements (WATER 24).

WATER 24 = 6.7 gal/veh × REST 24

WATER 24 = 6.7 gal/veh × 1074 veh/day

WATER 24 = 7196 gal/day

This is the amount of water that must be available continuously at the rest area.

g. Calculate the 8-hr water demand (WATER 8).

WATER 8 = 0.67 × WATER 24

WATER 8 = 0.67 × 7196 gal/day

WATER 8 = 4822 gal/8 hr

This is the amount of water that must be available throughout the peak 8-hr period of 8 a.m. to 4 p.m.

h. Calculate the peak 1-hr water demand (WATER 1)

WATER 1 = 6.5 × PK VOL 1

WATER 1 = 6.5 × 161 veh/hr

WATER 1 = 1047 gal/1 hr

This is the amount of water that must be available during the peak hour of the day.

i. The wastewater production rates must now be determined for the proposed rest area. Calculate the design wastewater production rate (WASTE 24).

WASTE 24 = 5.50 gal/veh × REST 24

WASTE 24 = 5.50 gal/veh × 1074 veh/day

WASTE 24 = 5907 gal/day

For the purpose of clarity and conservation WASTE 24 will be rounded off to 6000 gal/day. The value for WASTE 24 may now be used in designing the wastewater treatment facilities.

j. Calculate the 8-hr wastewater production (WASTE 8).

WASTE 8 = 0.67 × WASTE 24

WASTE 8 = 0.67 × 6000 gal/day

WASTE 8 = 4020 gal/8 hr

This is the peak 8-hr wastewater production rate for the proposed rest area. This flow should occur between 8 a.m. and 4 p.m.

k. The organic (BOD_5) and suspended solids (SS) loadings must now be computed. A concentration of 165 mg/ℓ BOD_5 and 190 mg/ℓ SS will be assumed for this example. Calculate the organic (BOD_5) loading in pounds per day.

BOD_5(lb/day) = WASTE 24 × BOD_5 × 8.34×10^{-6}

BOD_5(lb/day) = 6000 gal/day × 165 mg/ℓ × 10^{-6}

BOD_5(lb/day) = 8.26 lb/day

l. Calculate organic loading (BOD_5) in kilograms/day.

BOD_5(kilograms/day) = BOD_5(lb/day) × 0.45359

BOD_5(kilograms/day) = 8.26 lb/day × 0.45359

BOD_5(kilograms/day) = 3.75 kilograms/day

m. Calculate the solids (SS) loading in pounds/day.

SS(lb/day) = WASTE 24 × SS × 8.34×10^{-6}

SS(lb/day) = 6000 gal/day × 190 mg/ℓ × 8.34×10^{-6}

SS(lb/day) = 9.5 lb/day

n. Calculate the solids (SS) loading in kilograms/day.

SS(kilograms/day) = SS lb/day × 0.45359

SS(kilograms/day) = 9.5 lb/day × 0.45359

SS(kilograms/day) = 4.31 kilograms/day

A tabulation of the results of this example appears in Table 4-2. The

Table 4-2. Calculated Example Values.

Symbol	Values
HIWAY 24	11,929 veh/day
REST 24	1,074 veh/day
REST 8	720 veh/8 hr
PK VOL 1	161 veh/1 hr
WATER 24	7,196 gal/day
WATER 8	4,822 gal/8 hr
WATER 1	1,047 gal/1 hr
WASTE 24	6,000 gal/day
WASTE 8	4,000 gal/day
BOD_5	8.26 lb/day = 3.75 kg/day
SS	9.5 lb/day = 4.31 kg/day

calculated values in this table will be used in the remainder of the examples in this report.

4-6. SUMMARY

The design engineer is now able to determine water requirements at a proposed rest area. He is able to determine these requirements by monitoring nearby rest areas or by using the values proposed in this report based on work by Zaltzman. In a similar manner, he is able to predict daily flow of wastewater throughout the year. With these values and expected wastewater strengths he can determine the daily organic (BOD_5) and solids (SS) loads that he must design the wastewater treatment facility to remove.

With wastewater flow and daily loads of wastewater constituents determined, the design engineer is able to start designing the rest-area wastewater-treatment facility. Design guidelines for wastewater-treatment systems that may be used at rest areas are included in the following sections.

4-7. REFERENCES

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

2. Parker, C. E. et al., "Evaluation of a Water Reuse Concept for Highway Rest Areas," presented at 56th Annual Meeting of the Transportation Research Board, Washington, D. C., January 1977.

5. REST AREA DESIGN SITUATIONS

5-1. INTRODUCTION

Previous discussion of water usage, wastewater production, wastewater characteristics, and wastewater-treatment system design has been directed toward rest areas that receive fairly uniform usage, serve one-way traffic, have an abundant supply of potable water, and use standard fixtures. However, in many instances these situations do not exist; in these instances the design engineer must adjust his design to reflect the true situation. Therefore, modifications that must be made to the design procedure to reflect the true situation of a given rest area are outlined below.

5-2. SEASONAL USE REST AREAS

In many instances, particularly in northern latitudes, rest areas are operated on a seasonal basis; i.e. they are closed for a period of time each year. In such instances the design engineer must consider the fact that there will be a period of zero flow of wastewater to the wastewater treatment plant. Because biological treatment systems require a sustained supply of food (wastewater), prolonged periods of nonuse will eliminate the possible use of some treatment systems at rest areas. Those treatment systems which are particularly affected are extended aeration and the rotating biological filter systems. Lagoons and septic tanks, because they are able to function anaerobically, are better able to withstand periods of zero wastewater flow.

5-3. REST AREAS WITH INSUFFICIENT WATER SUPPLIES

In many instances rest areas have been built or the rest-area location preselected in an area that is unable to produce an adequate supply of potable water. In such instances various steps may be taken by the design engineer to reduce the water requirement of the rest area. Among the options available to the design engineer are recirculation of treated wastewater for flushing purposes, use of grey water from sinks and drinking fountains for flushing purposes, use of flow reduction fixtures, and use of waterless toilet systems.

5-4. JOINT USE REST AREAS

In some instances rest areas may be designed to accommodate visitors from both directions of travel, or may be used in conjunction with nearby park and camping facilities. In such cases the design engineer must anticipate the additional water demand and wastewater production based not only on roadway traffic, but also on anticipated use by park visitors. In designing such an area it is recommended that the design engineer consult with the local park service personnel to obtain predictive use data.

5-5. IRRIGATED REST AREAS

Many rest areas use irrigation as a means of enhancing the beauty of the area. As such, the anticipated demand for irrigation water must be added to the water demand calculated for the rest-area comfort facilities to obtain the total water demand for the rest area. In this manner a water supply system may be properly sized for the entire rest area, or the design engineer may wish to design two separate systems, one for irrigation and one for comfort facilities.

5-6. REST AREAS WITH TRAILER DUMPS

At many rest areas, trailer dump stations have been installed to accommodate the traveler with camping trailers equipped with toilets and holding tanks. Most trailer dump stations are also equipped with a water hose for flushing out the holding tanks after the contents have been emptied into the trailer dump. At present the trailer dump stations use one of three methods for disposing of trailer dump wastes: drainage into a holding tank that is later pumped out and disposed of at a municipal treatment plant; separate treatment system for the trailer dump station such as a septic tank leach field; or connection with the wastewater treatment facilities that service the comfort stations.

At rest areas where the trailer dump stations have been connected with the wastewater-treatment facilities the design engineer must take into account the added flow from the dump station and the high constituent concentration of the trailer dump waste. Because of the high strength of the trailer dump wastes, the design engineer may wish to design a holding tank for the dump wastes and have the wastes pumped throughout the day to the treatment plant, thus avoiding any possible shock loading to the plant.

5-7. GENERAL DESIGN PROCEDURE

The previous sections of this report have dealt with the problems existing at rest-area wastewater-treatment systems, the 1977 effluent requirements of PL 92-500, and methods for determining water usage, wastewater production, and wastewater-constituent concentrations. The following sections (Chapters 6-16) are design guidelines for wastewater-treatment processes at rest areas. The treatment processes discussed will, if correctly designed, constructed, and operated, produce an effluent that will meet or surpass the requirements of PL 92-500. A logic flow chart for the proposed general design procedures is shown in Figure 5-1. The first two pages of this figure may be used as a key to the individual modules contained in the flow chart itself.

A few notes on use of the logic flow chart will help the design engineer responsible for rest area design and/or construction. Coordination should be made and maintained throughout the design and construction of the rest area with other State agencies such as the State Environmental Agency, and the State Health Agency. Coordination should also be maintained with State, and/or Federal agencies that may have knowledge of the rest area location such as the Soil Conservation Service or the State Geological Service or the National Weather Service. It is also not necessary to produce a design for each and every treatment process given in Chapters 6-16. If some systems are not allowed by various state and/or local agencies then disregard those systems. In short, select the system you believe in your engineering judgement will work best at your particular rest area location.

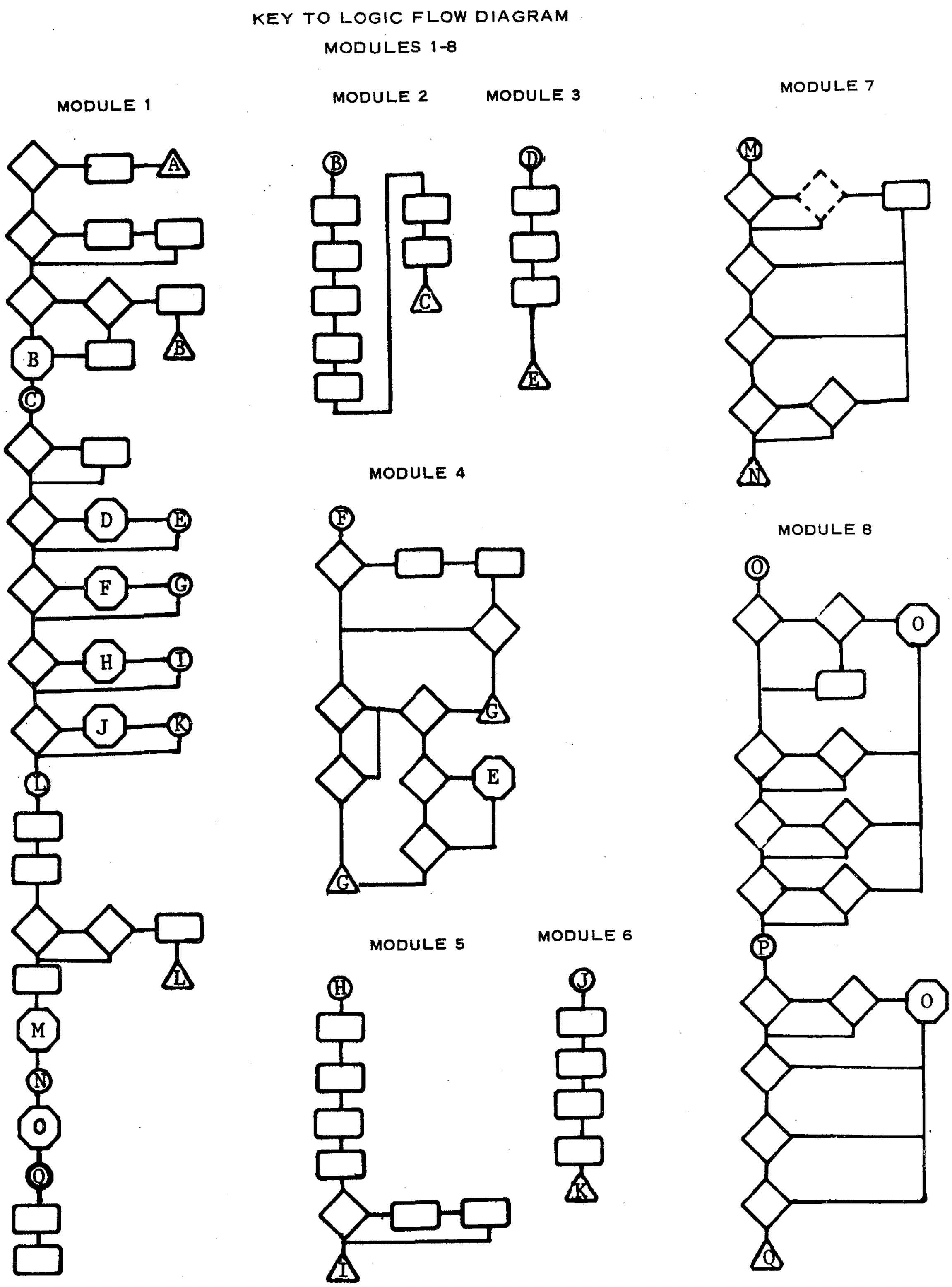


Figure 5-1. Logic flow diagram. (sheet 1 of 21)

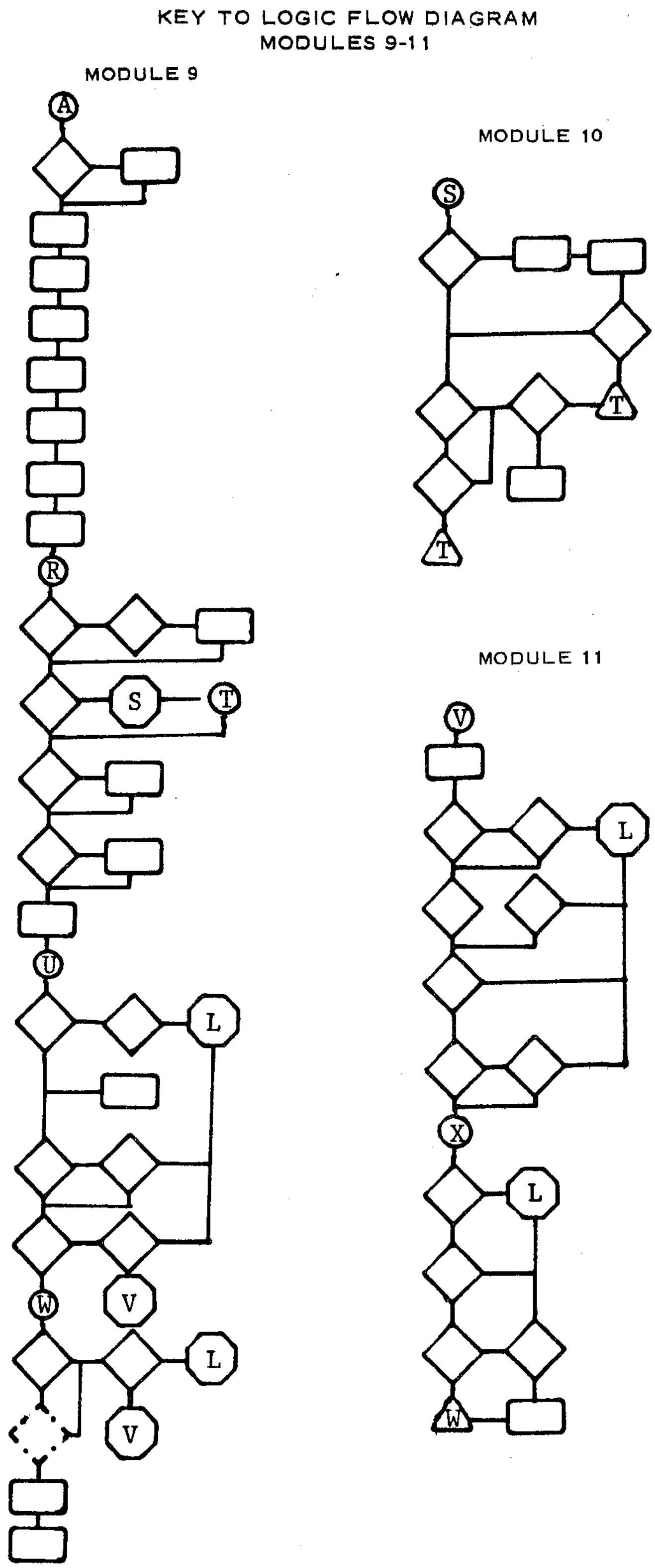
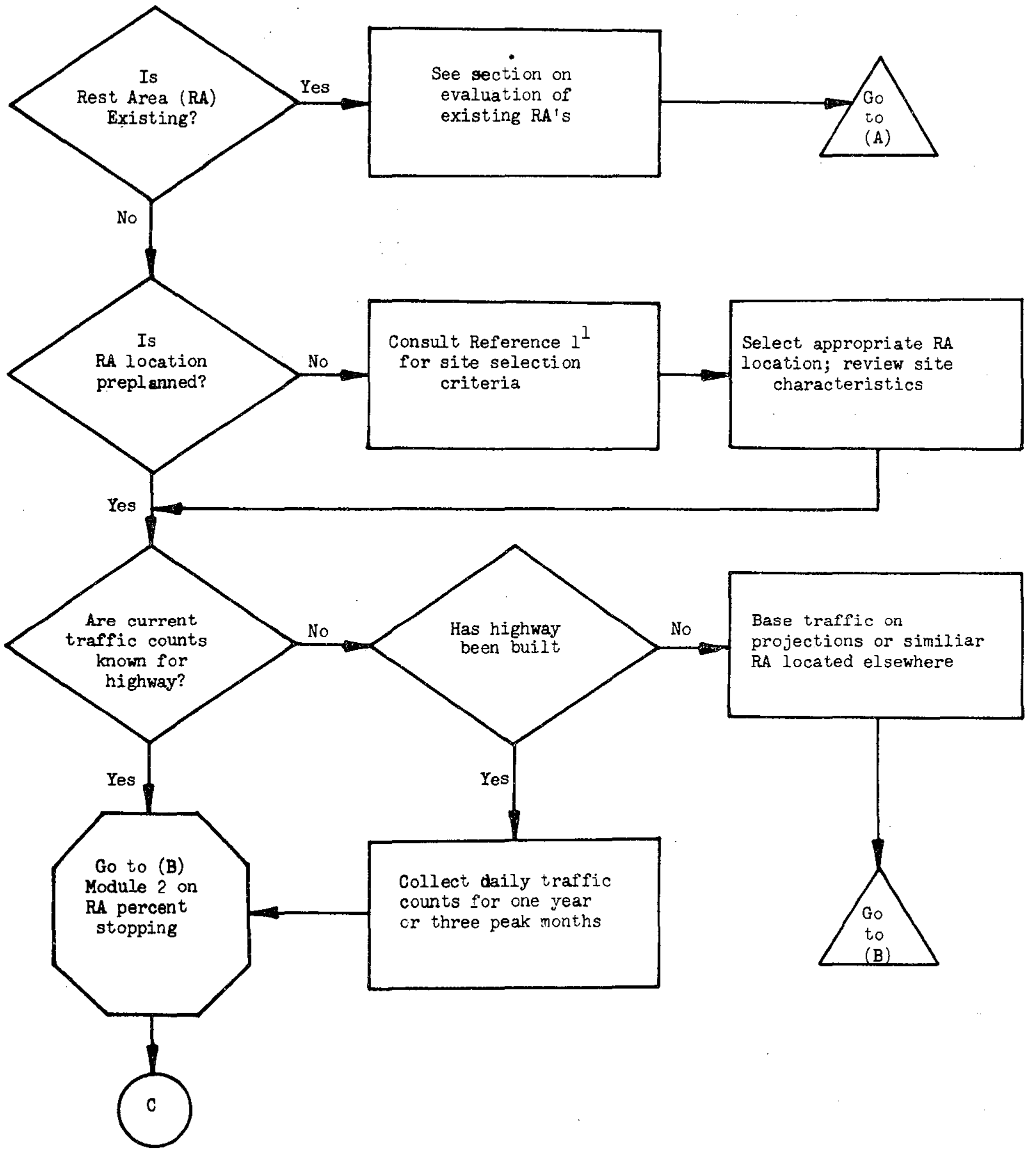


Figure 5-1 (Continued).

(sheet 2 of 21)

MODULE 1. REST AREA WASTEWATER TREATMENT SYSTEMS,
GENERAL DESIGN PROCEDURE

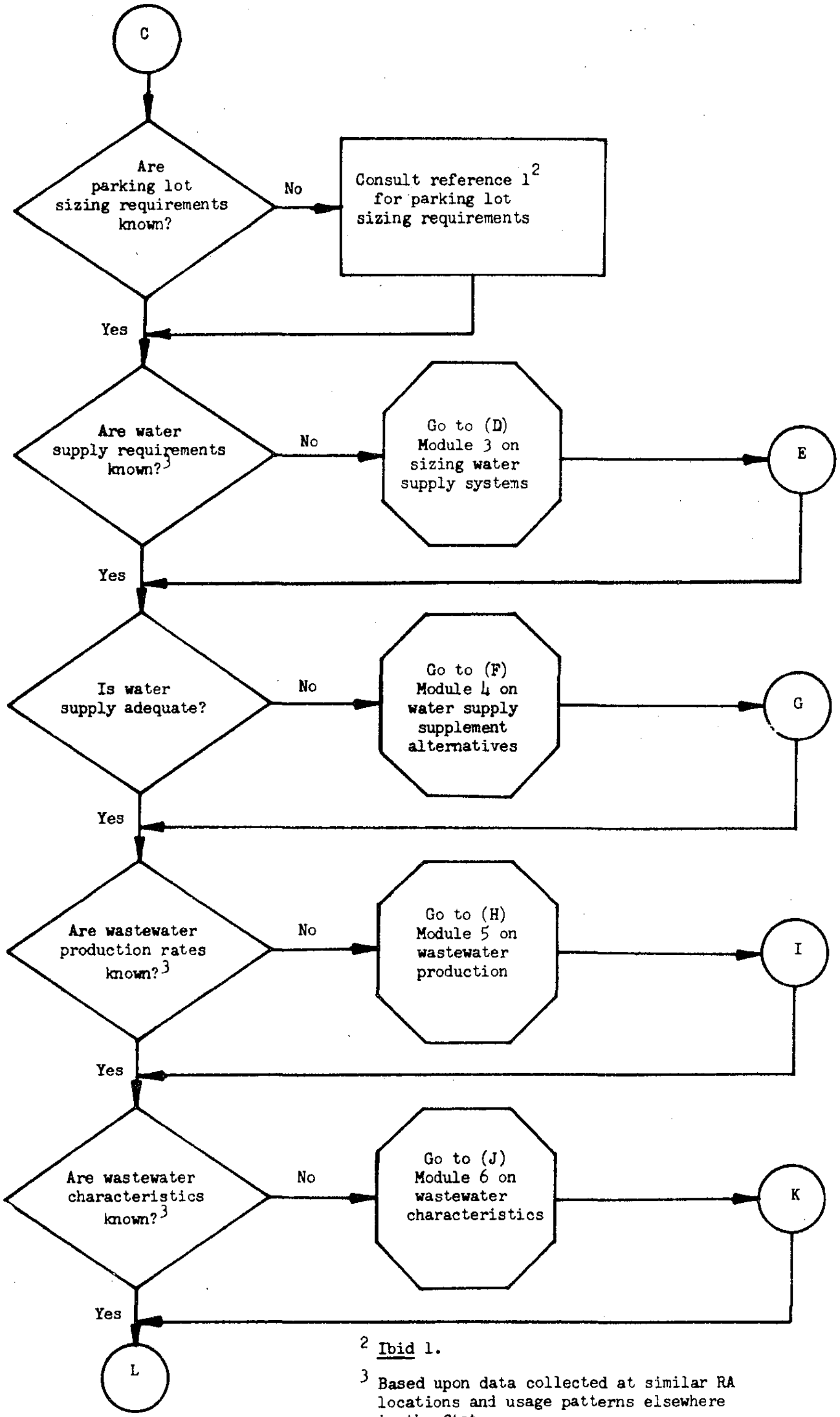


[1] "A Guide on Safety Rest Areas for the National System of Interstate and Defense Highways," published by American Association of State Highway Officials, 341 National Press Building, Washington, DC 20004.

Figure 5-1 (Continued).

(sheet 3 of 21)

MODULE 1 (CONTINUED). REST AREA WASTEWATER TREATMENT SYSTEMS, GENERAL DESIGN PROCEDURE



[2] Ibid 1.

[3] Based upon data collected at similar RA locations and usage patterns elsewhere in the State.

Figure 5-1 (Continued).

(sheet 4 of 21)

MODULE 1 (CONTINUED). REST AREA WASTEWATER TREATMENT SYSTEMS, GENERAL DESIGN PROCEDURE

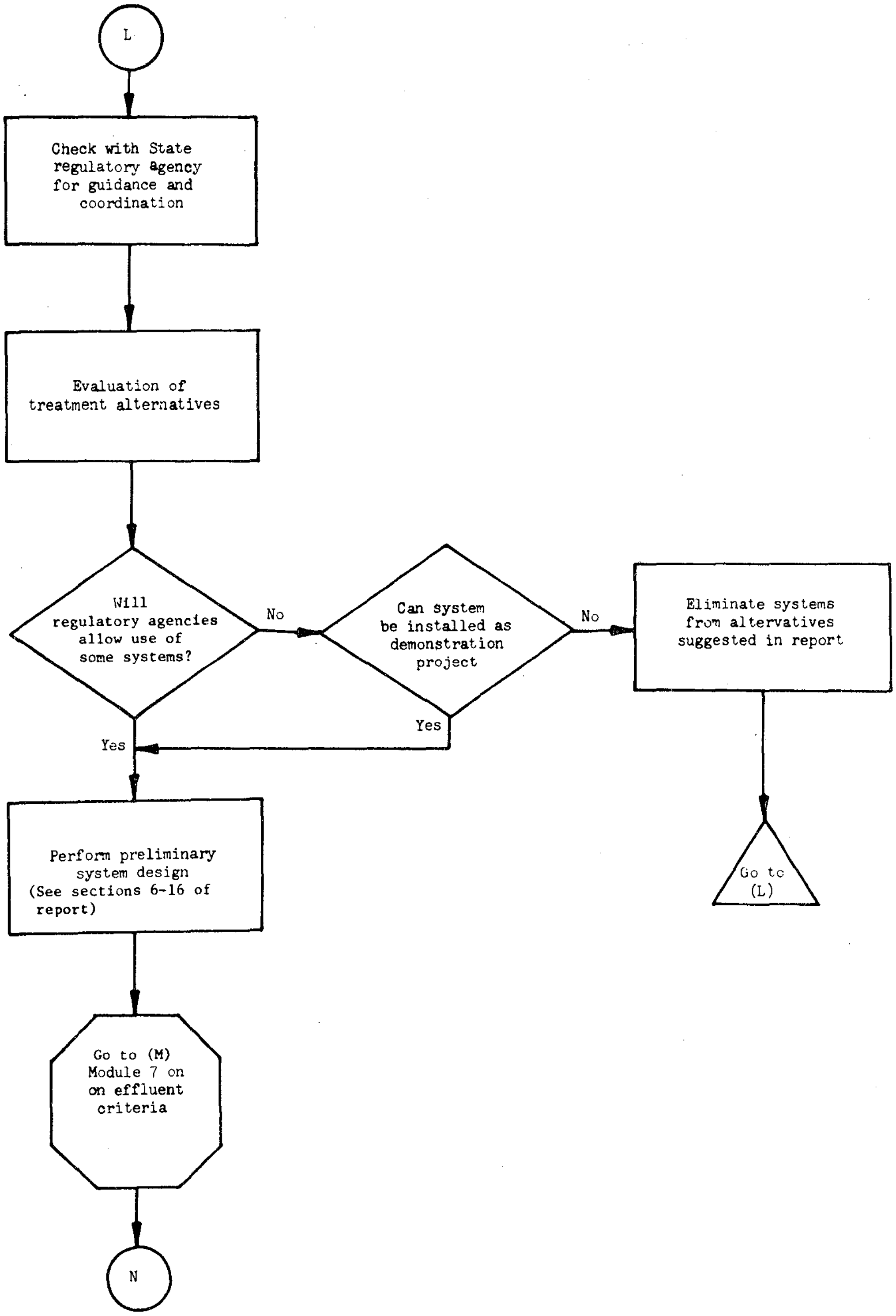


Figure 5-1 (Continued).

(sheet 5 of 21)

MODULE 1 (CONTINUED). REST AREA WASTEWATER TREATMENT SYSTEMS, GENERAL DESIGN PROCEDURE

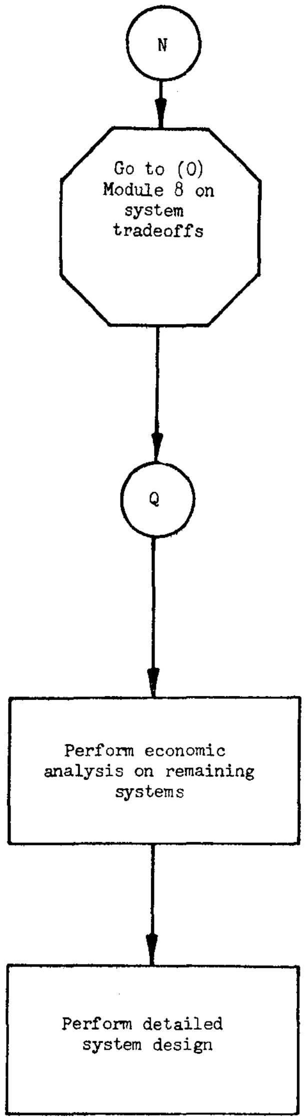
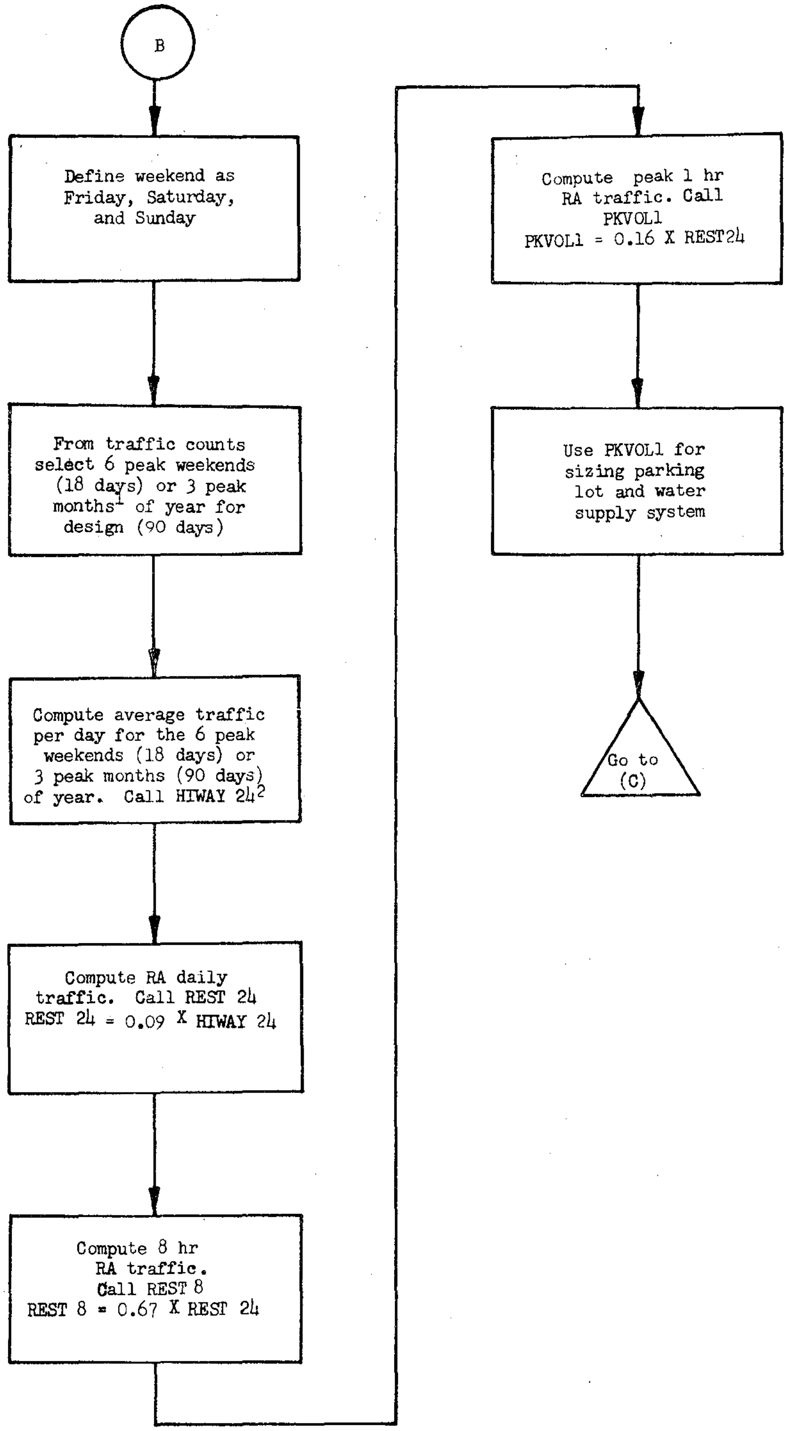


Figure 5-1 (Continued).

(sheet 6 of 21)

MODULE 2. REST AREA TRAFFIC



[1] Peak months or weekends do not have to be consecutive.

[2] If RA serves both directions of traffic use 1.0 X HIWAY 24; if RA serves one direction of traffic use 0.5 X HIWAY 24.

Figure 5-1 (Continued).

(sheet 7 of 21)

MODULE 3. WATER USAGE

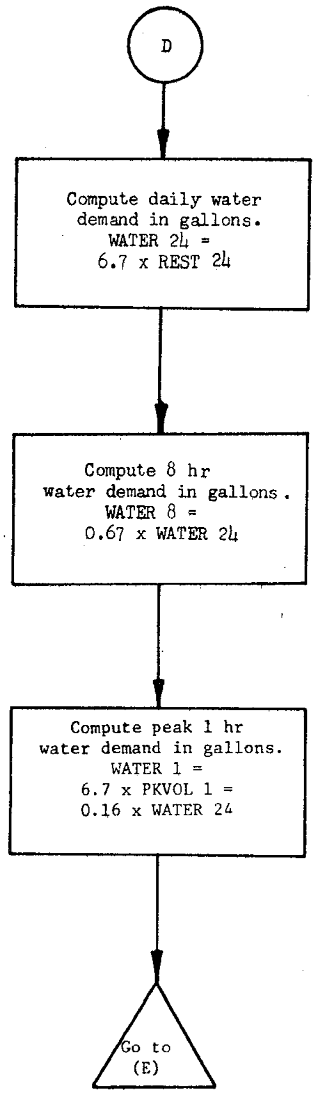


Figure 5-1 (Continued).

(sheet 8 of 21)

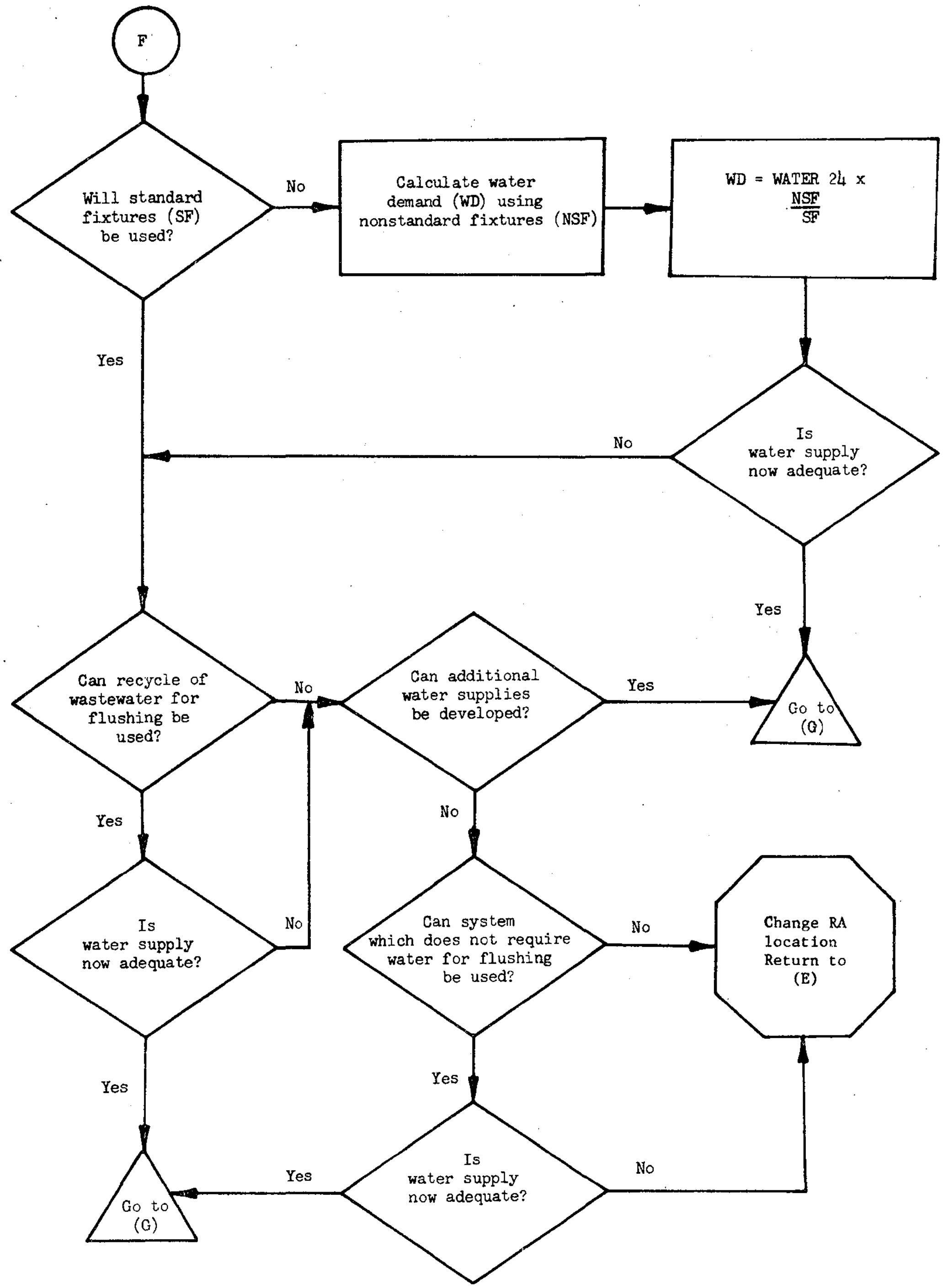


Figure 5-1 (Continued)

(sheet 9 of 21)

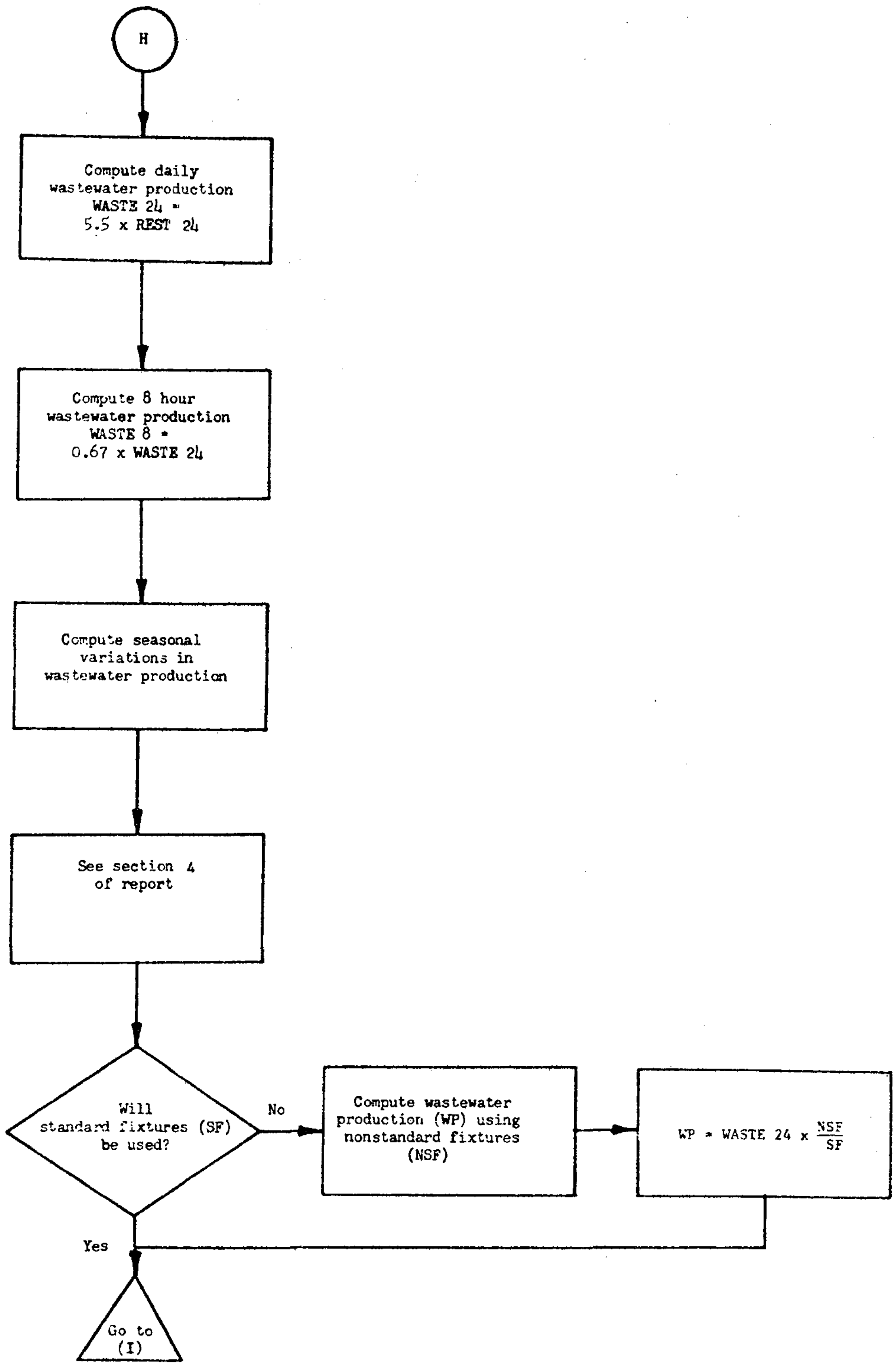
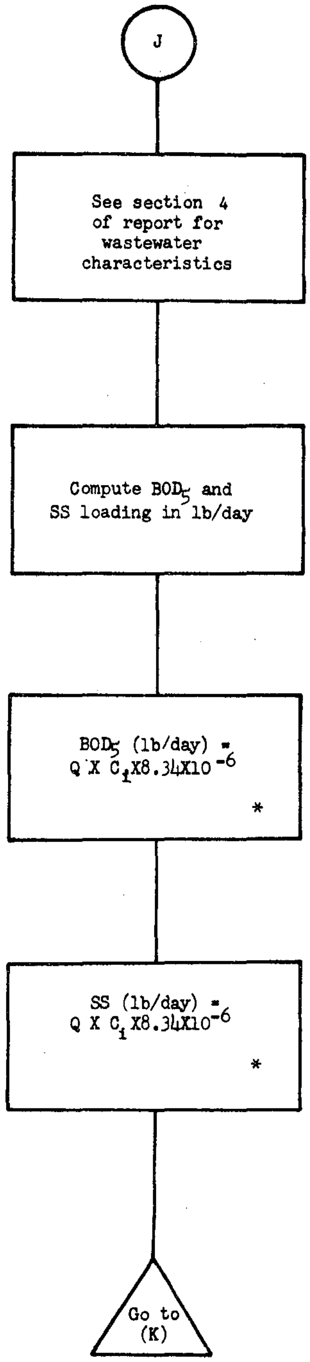


Figure 5-1 (Continued).

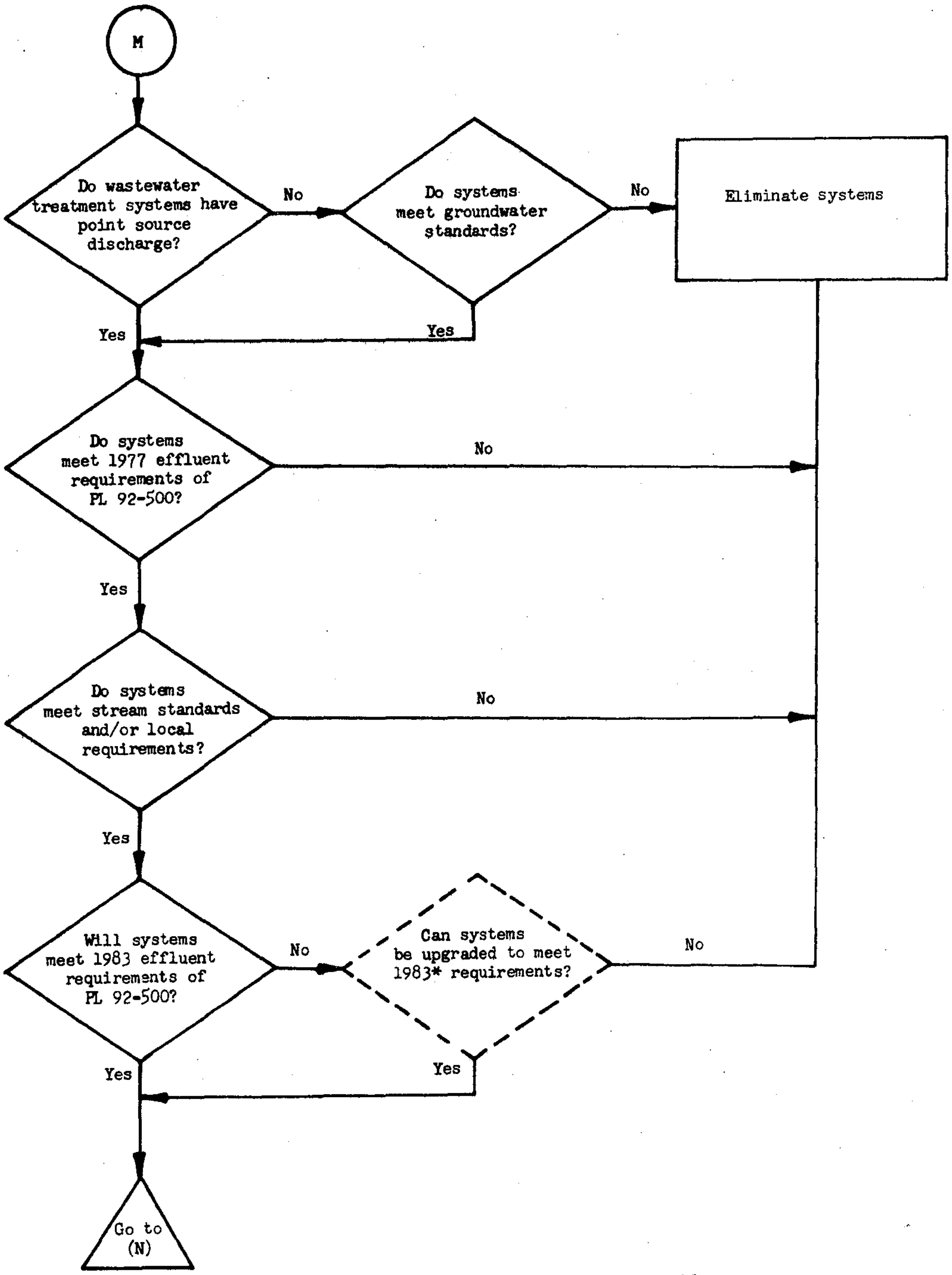
(sheet 10 of 21)



* Q = flow, gal/day.
C_i= constituent concentration, mg/l.

Figure 5-1 (Continued).

(sheet 11 of 21)



* 1983 requirements not yet available.

Figure 5-1 (Continued).

(sheet 12 of 21)

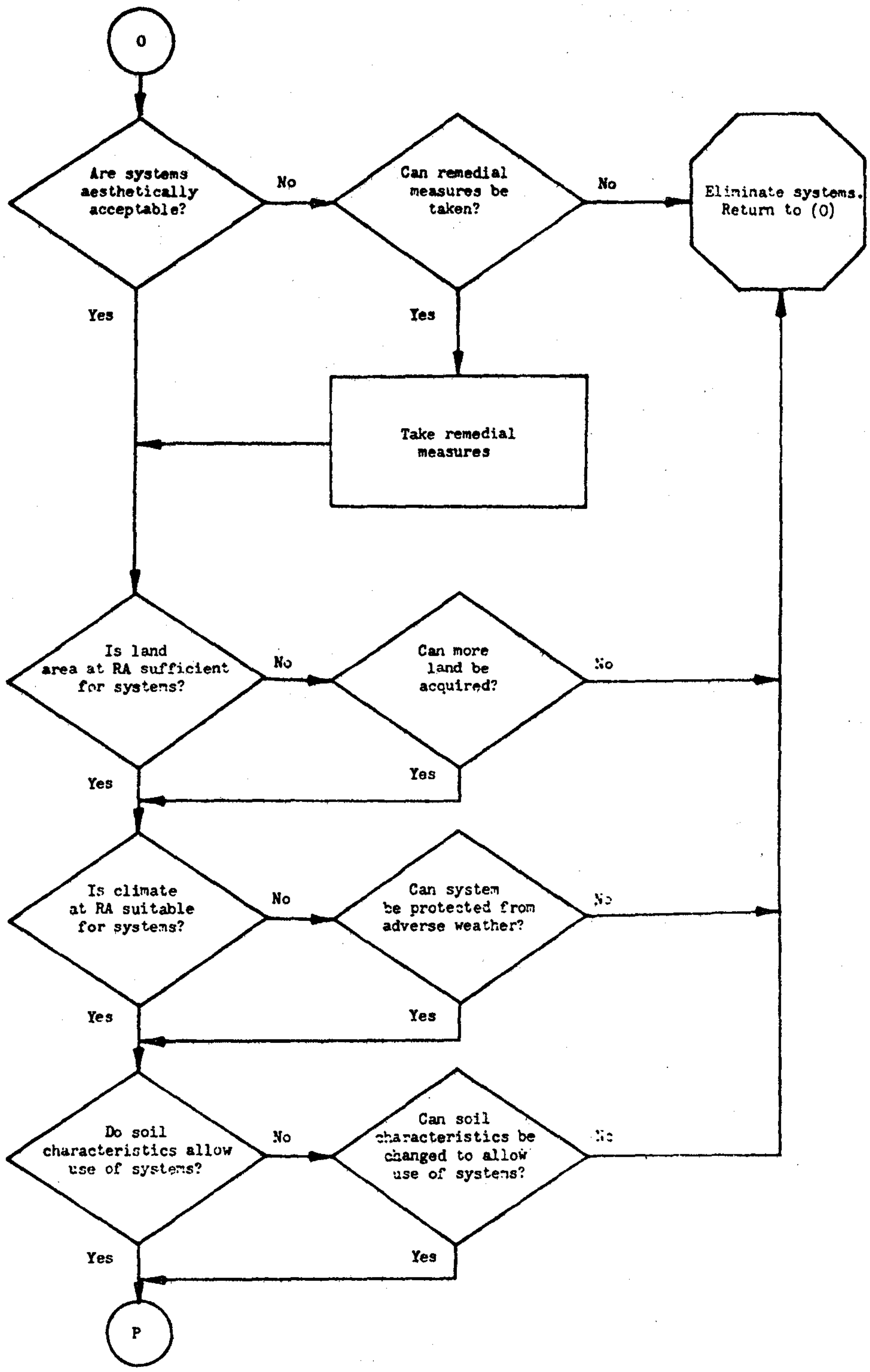


Figure 5-1 (Continued).

(sheet 13 of 21)

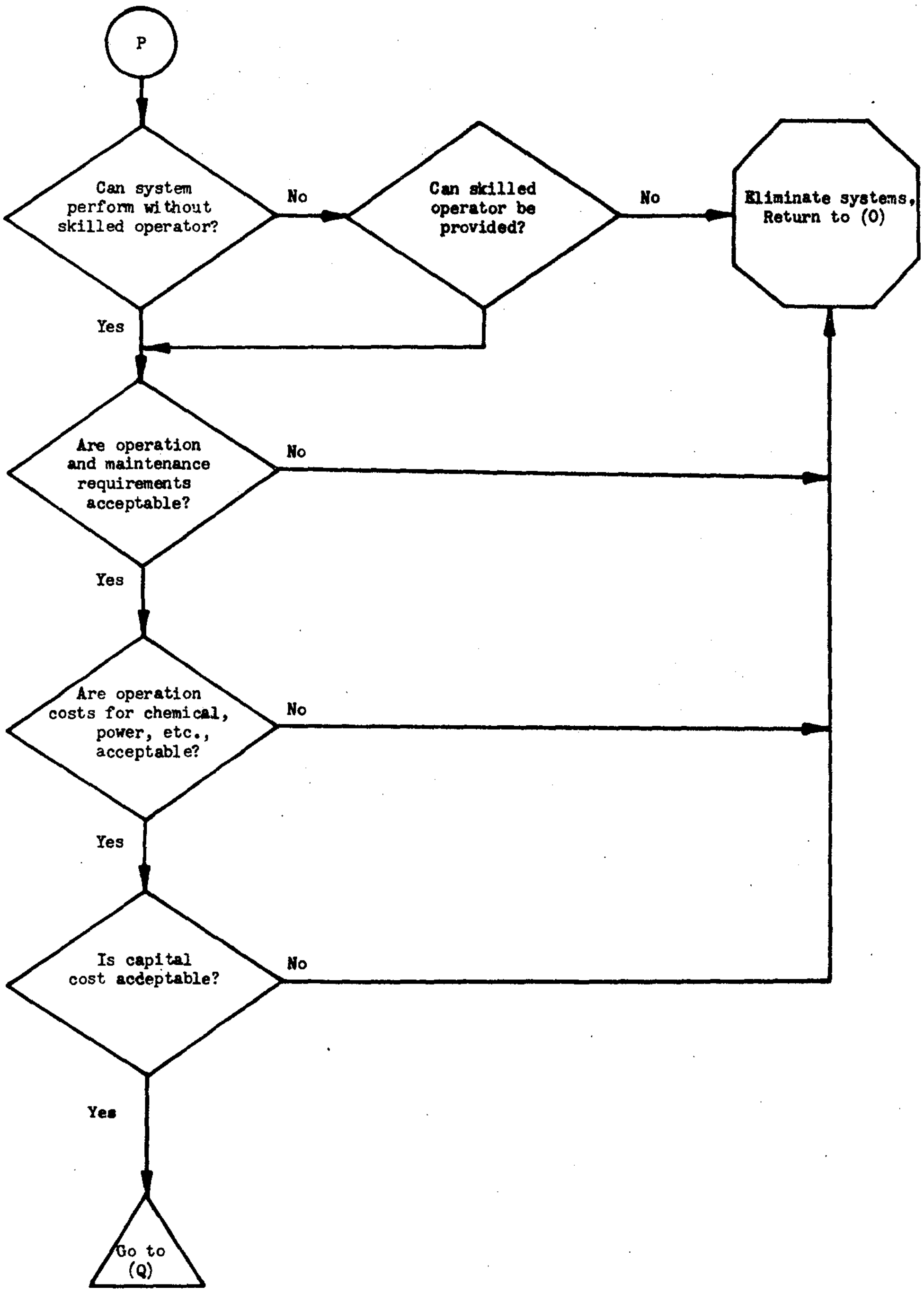
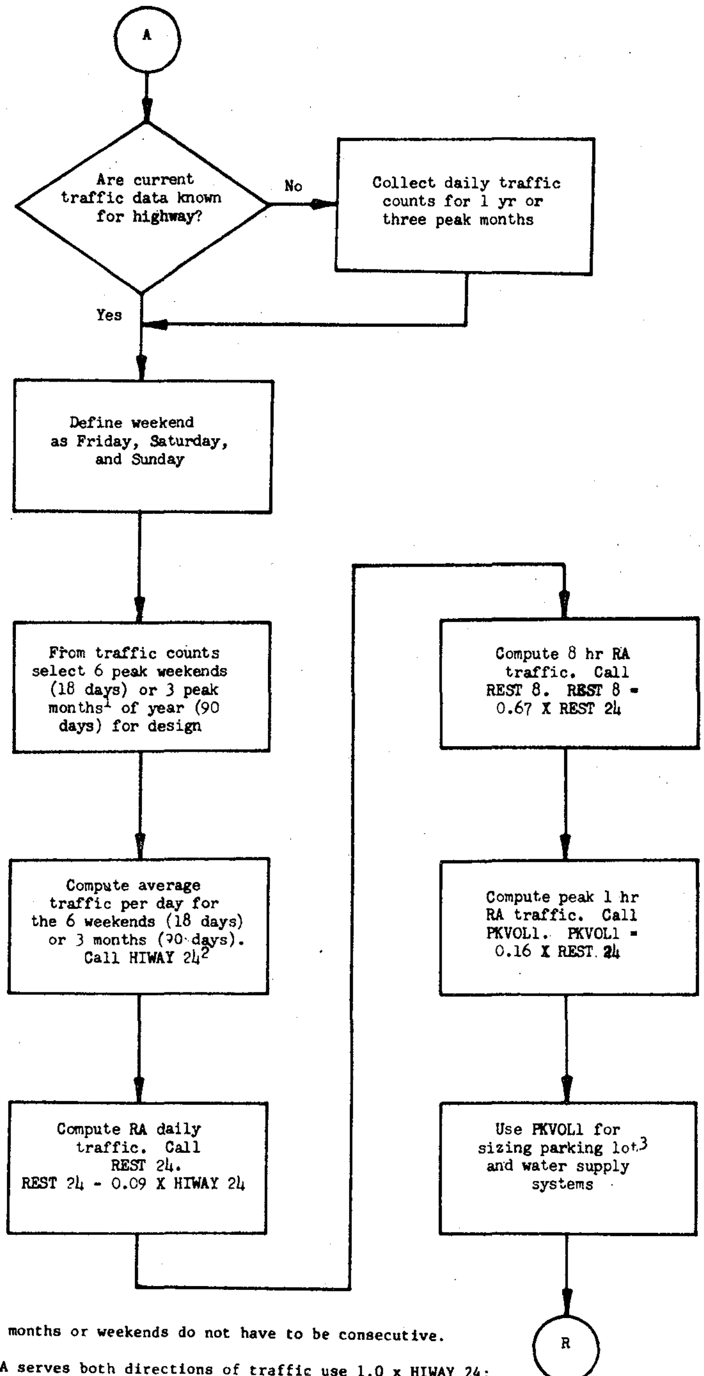


Figure 5-1 (Continued).

(sheet 14 of 21)

MODULE 9. EVALUATION OF EXISTING REST AREAS



[1]Peak months or weekends do not have to be consecutive.

[2]If RA serves both directions of traffic use 1.0 x HIWAY 24; if RA serves one direction of traffic use 0.5 x HIWAY 24.

[3]"A Guide on Safety Rest Areas for the National System of Interstate and Defense Highways," published by American Association of State Highway Officials, 341 National Press Building, Washington, DC 20004.

Figure 5-1 (Continued).

(sheet 15 of 21)

MODULE 9 (CONTINUED). EVALUATION OF EXISTING REST AREAS

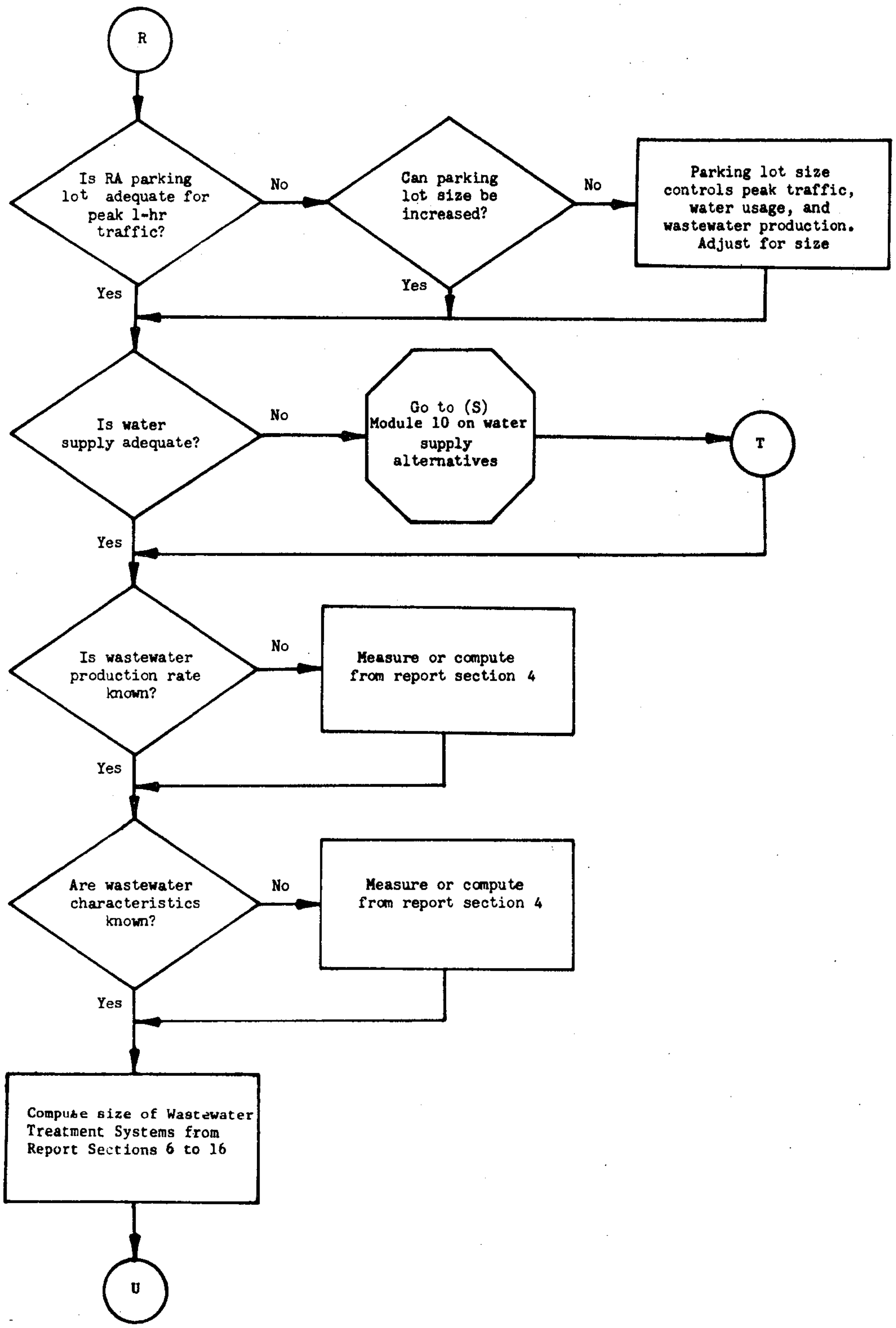


Figure 5-1 (Continued).

(sheet 16 of 21)

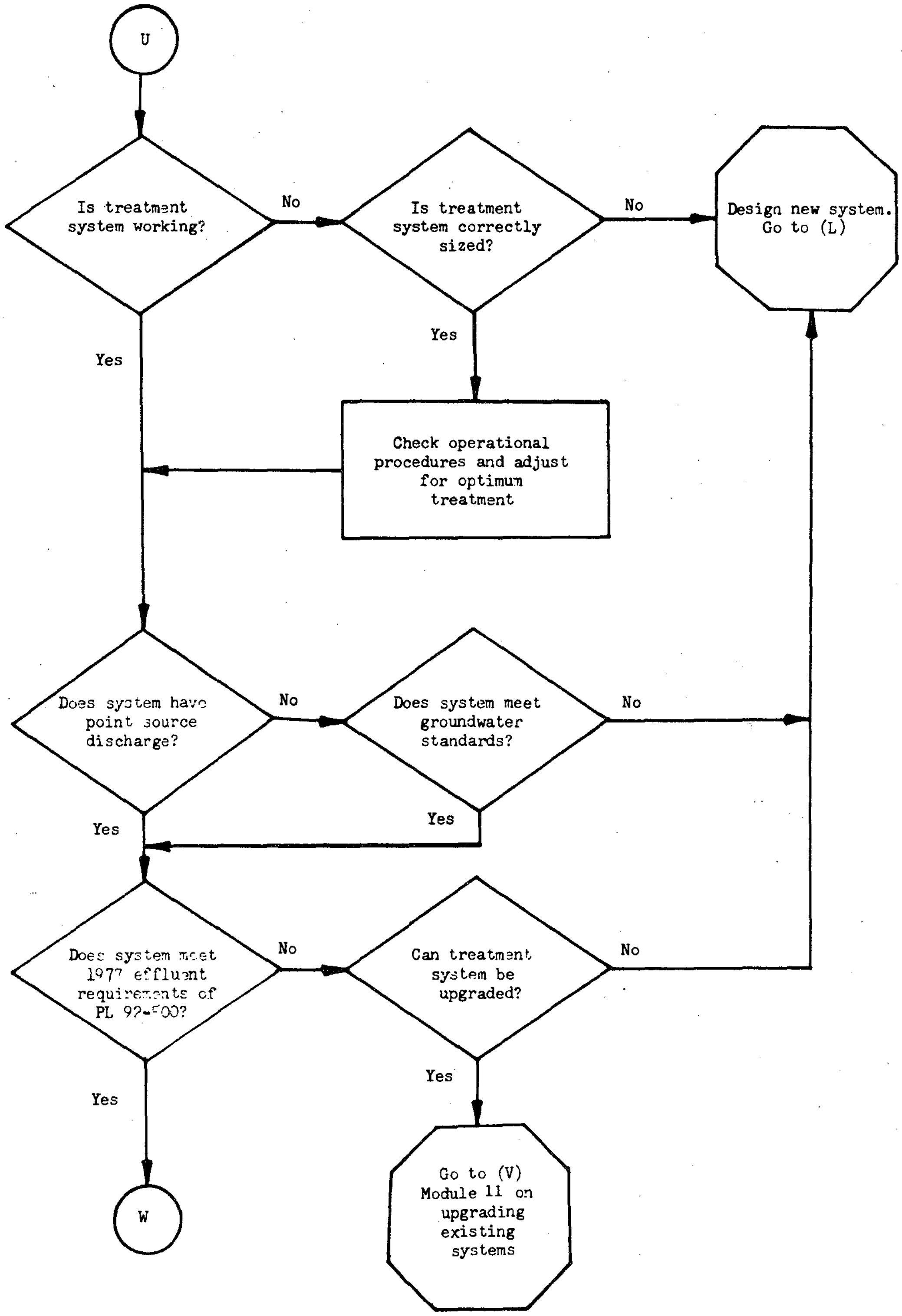
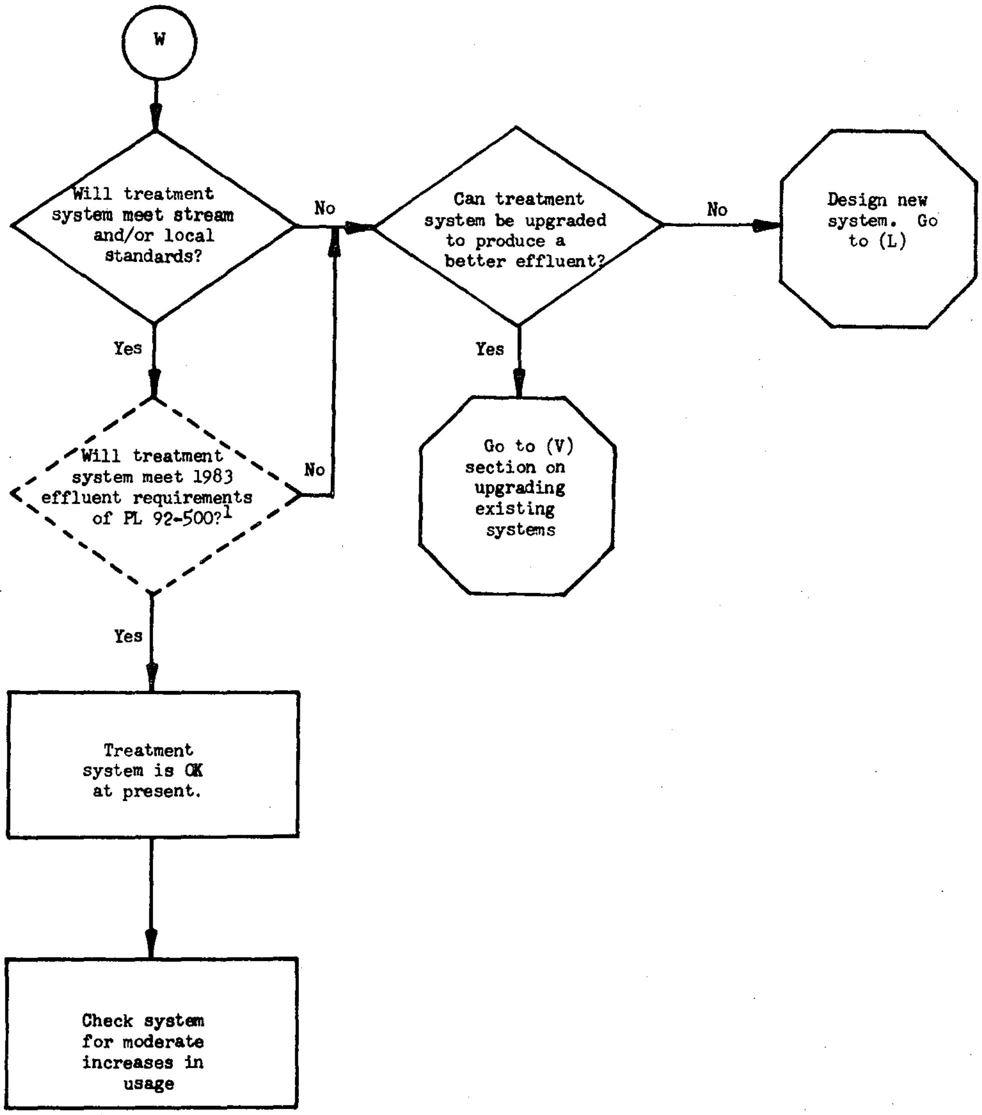


Figure 5-1 (Continued).

(sheet 17 of 21)

MODULE 9 (CONTINUED). EVALUATION OF EXISTING REST AREAS



[1] 1983 effluent criteria not yet defined.

Figure 5-1 (Continued).

(sheet 18 of 21)

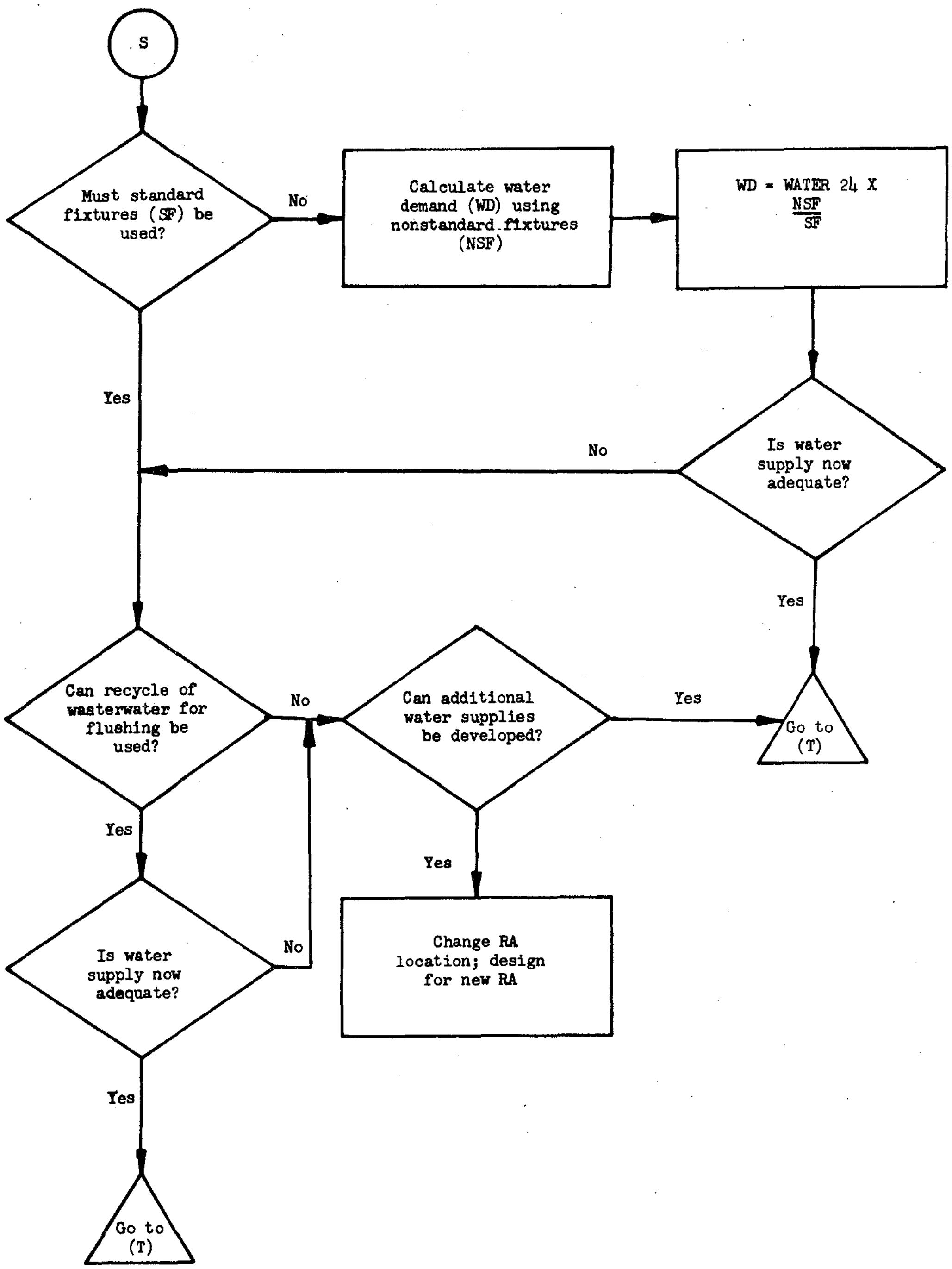


Figure 5-1 (Continued).

(sheet 19 of 21)

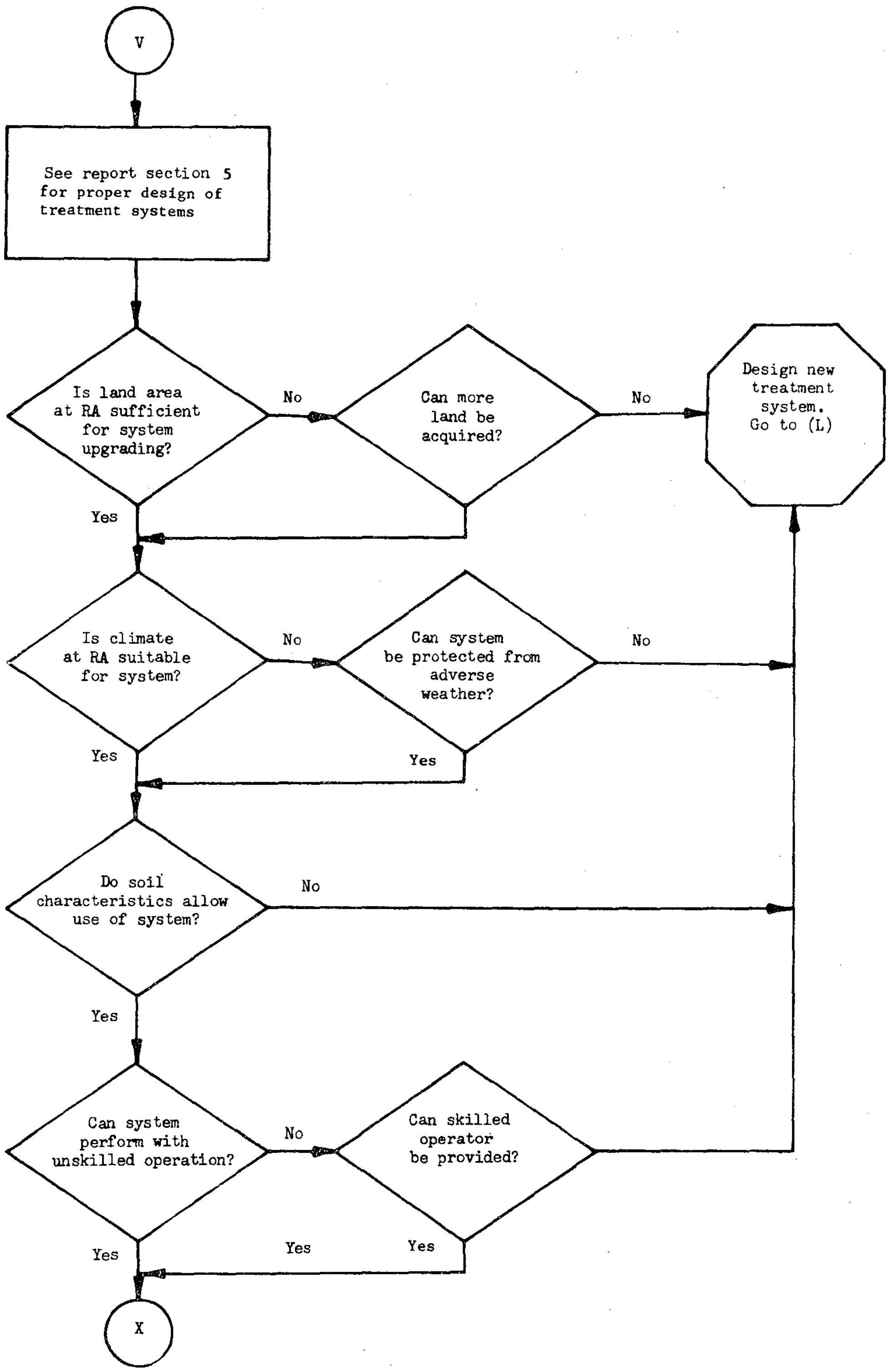


Figure 5-1 (Continued).

(sheet 20 of 21)

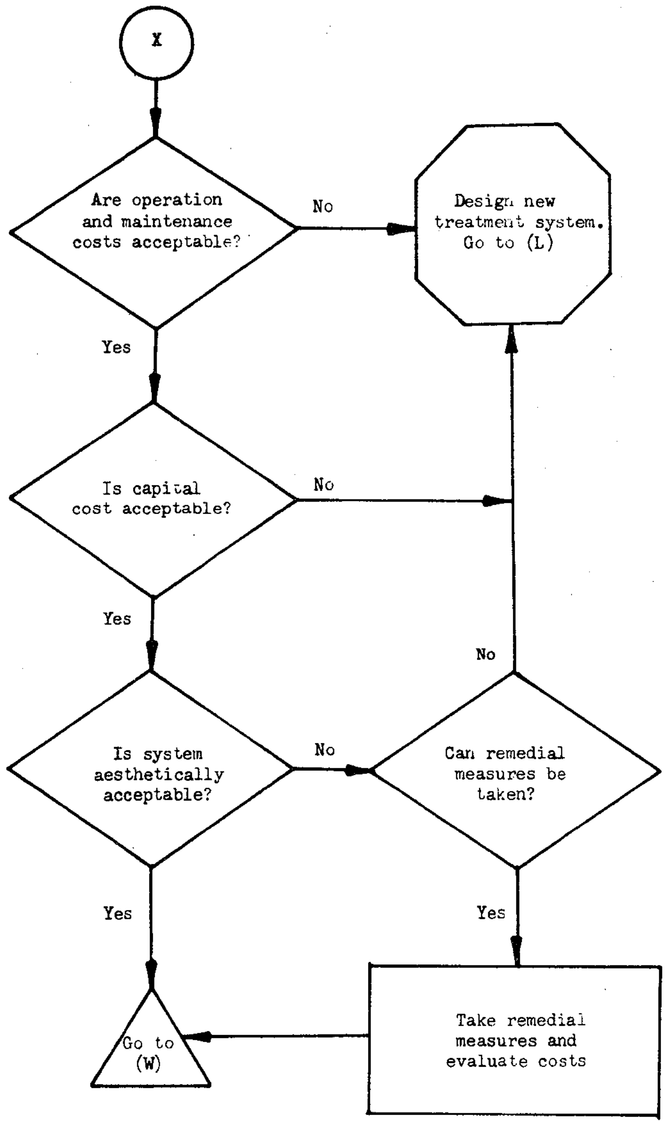


Figure 5-1 (Continued).

(sheet 21 of 21)

6. EQUALIZATION OF WASTEWATER FLOWS

6-1. PROCESS DESCRIPTION

Equalization is a unit operation designed to dampen peak loadings, thereby providing an evenly distributed loading on downstream wastewater-treatment processes. An equalization system may be designed to equalize fluctuating loadings caused by variations in wastewater flow or wastewater concentration. As has been stated previously, rest-area-generated wastewater generally exhibits significant variations in flow, but relatively small variations in strength. However, dumping of recreational vehicle wastewaters, where such facilities are provided at rest areas, would result in high concentration loadings of short duration to the treatment system. Discussion in this section will center on wastewater flow equalization, but the same principles may be applied toward equalizing concentration variations.

In addition to the flow fluctuations caused by changes in rest area usage, many small wastewater-treatment systems at rest areas experience shock hydraulic loadings as a result of pump station design. Raw wastewater must be pumped to the treatment plant where the elevation of the plant influent port is greater than the elevation of the sewer line from the rest-area facilities. This pumping is often provided by centrifugal sewage pumps. To prevent clogging of the pump due to raw wastewater solids, engineers specify that the pump be capable of passing a 2-1/2- to 3-in. sphere. The minimum size centrifugal pump that will meet this specification and operate efficiently is 50 to 100 gpm. The flow for a 10,000-gpd treatment plant averaged over a 24-hr day is only 7 gpm, or 21 gpm if all the flow is received in 8 hr. Wet wells are generally sized to allow the pump to run at least 2 min between high and low levels. Therefore, to pump 10,000 gal, a 100-gpm pump would pump 200 gal 50 times during the day. These surges may be of sufficient magnitude to affect the performance of the treatment systems. More uniform pumping rates may be achieved using equalization systems, using pneumatic or air-lift pumps, or providing flow splitter boxes to recycle a portion of the flow back to the wet well.

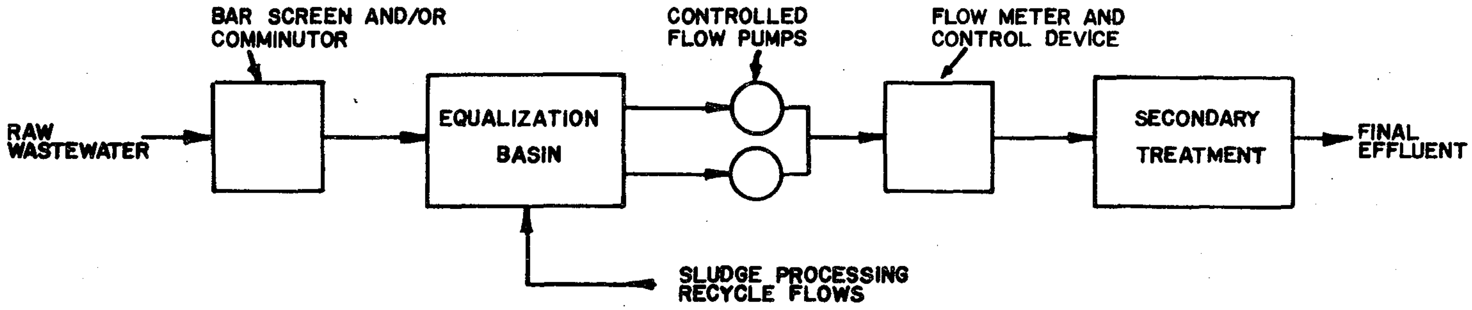
Flow equalization can benefit any wastewater-treatment system, particularly where wide fluctuations in flow are experienced. Most fundamental process design equations assume steady-state conditions, i.e., flows and concentrations that are constant with time. Obviously this rarely occurs in domestic wastewater-treatment systems; therefore, operational data are relied on for design guidance. Such operational data can indicate the degree of load fluctuation that the treatment system can withstand. Larger wastewater-treatment systems, such as those for municipalities, do not experience the degree of load fluctuation occurring at rest areas, and can usually accommodate the diurnal load variations without flow equalization. Flow equalization for rest area wastewater will provide a constant load so that the treatment system can be designed with more confidence using steady-state design equations.

Treatment systems that are most susceptible to severe load fluctuations include extended aeration, rotating biological discs, and physical-chemical systems. Primary problems for extended aeration plants include washout of solids in the final clarifier due to increased overflow rates, unsteady food-to-microorganism ratios in the aeration tank, and increased oxygen requirements for peak loading periods. Flow equalization at an extended aeration plant would allow design of the final clarifier and aeration equipment to be based on average flows, thereby reducing the size of these units. A higher quality effluent could be achieved because the microbial mass will function more efficiently in a stabilized environment, i.e., a constant F/M ratio. Rotating biodiscs do not perform as well under fluctuating flows because of decreased detention times in the reactor under peak loading conditions and loss of solids from the final clarifier as a result of hydraulic surges. Physical-chemical systems are more difficult to operate under fluctuating loading conditions because of the disruption in chemical feeding schedule, increased overflow rates in settling tanks during peak loadings, and channeling or short circuiting in filtration systems. Uniform flow to physical-chemical systems would reduce the magnitude of such problems.

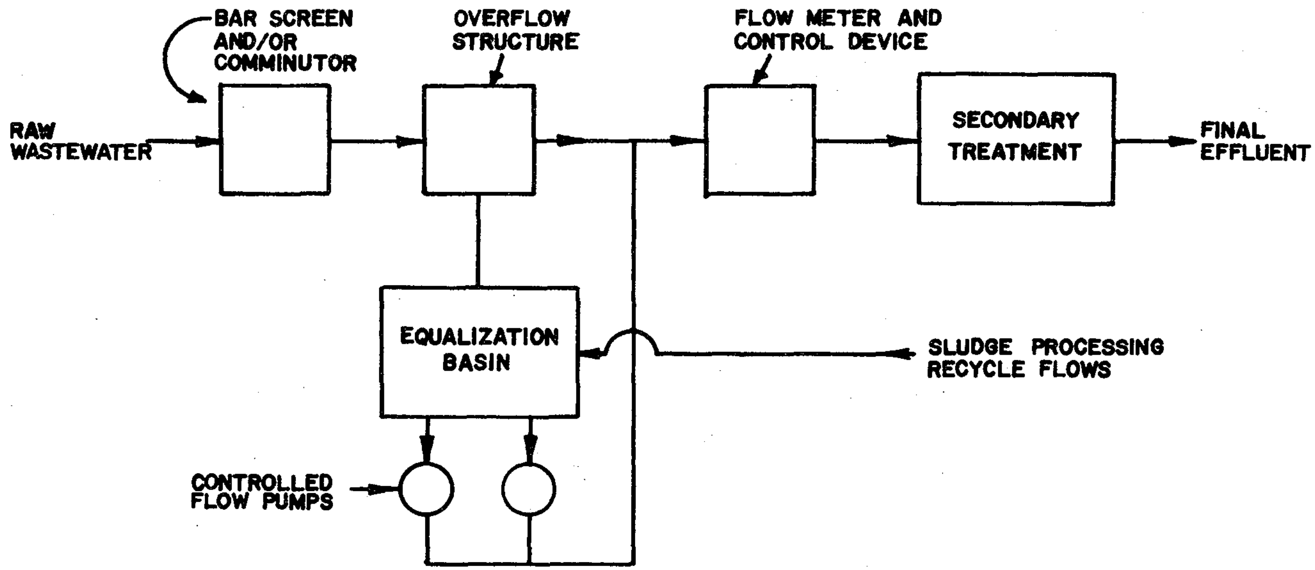
Equalization of flow is a technique that can be applied to upgrading existing plants, as well as to design of new plants. Metcalf and Eddy[1] in EPA's Technology Transfer publication "Flow Equalization" report several case histories of municipal wastewater-treatment systems that have been upgraded to an increased capacity or a higher quality effluent through employment of equalization systems. Gaines[2] reported operational difficulties and their solutions for a municipal flow equalization system. Operational data are not available for equalization of flow at rest areas, but, as in the case of municipal systems, equalization may be the most economical method of improving the performance of an existing system that presently will not comply with state and Federal regulations.

Two flow schemes, termed in-line and side-line, may be used to accomplish equalization. Flow diagrams for these two schemes are presented in Figure 6-1. An in-line equalization basin is connected directly to the influent line for the treatment system. All of the wastewater flow enters and exits the in-line basin before being subjected to secondary treatment. An important advantage of the in-line system is its ability to dampen fluctuations in waste strength as well as hydraulic fluctuations. In contrast, a side-line equalization basin receives influent wastewaters only when the flow rate exceeds the design average daily flow. The wastewater thus stored in the side-line basin is then pumped to the treatment system during periods when the influent flow is less than the design daily average. Since only a portion of the influent flow is stored, dampening of concentration variations is less with a side-line basin than with an in-line basin. One advantage of a side-line basin is the reduced pumping requirement since only a portion of the flow must be pumped.[1]

The method of determining the size of an equalization basin depends on the desired effect of the basin. Dampening of influent concentrations may be based on statistical evaluation of the variation in waste strength in order to estimate the desired level of confidence for an equalization basin.[3] The method that will be discussed here is a graphical technique using measured or estimated concentrations or flows.



a. Inline flow equalization.



b. Sideline flow equalization.

Figure 6-1. Flow equalization in secondary treatment plant.

The basic data needed for design are the variations in wastewater flow, preferably on an hourly basis, for the time period chosen for equalization. For municipal systems, equalization of the diurnal variation is usually sufficient. However, at rest areas equalization may be most beneficial over a weekly period. Wallace[3] reports that at least ten of the selected equalization periods (usually of one week duration each) should be evaluated in order to determine the most extreme conditions. If equalization is being provided for an existing plant, actual flow data should be collected. For new rest-area treatment systems where flow data are not available, highway traffic flow variations may be used in conjunction with estimates of rest-area usage and wastewater production for each vehicle. An equalized highway traffic flow could be determined and then converted to a required liquid volume of equalization. Once traffic or wastewater flow data have been collected, a hydrograph should be constructed to graphically illustrate the variations in flow over the given time period. An example of hourly traffic volumes for an Illinois rest area[4] is given in Figure 6-2.

Graphical determination of equalization volume is achieved by construction of a cumulative flow diagram, that is, a plot of cumulative flow or volume versus time. Cumulative flow is the area under the hydrograph curve for a given time interval and is therefore the volume for that interval of time. An example of a cumulative flow diagram using traffic flow as the equalized parameter is given in Figure 6-3 for the hydrograph shown in Figure 6-2. A simplified example is given in Figure 6-4.

The slope of a line drawn from the point of origin to the last point on the cumulative flow line (the dashed line in Figure 6-3) represents the average flow rate and is the equalized flow rate to the wastewater-treatment facility. In construction of a cumulative flow diagram the cycles of flow must be over a complete time period; i.e., each cycle is a full 24 hours. Next, two lines A and B are drawn parallel to the average flow line such that one is tangent to each extremity of the cumulative flow line. The vertical distance between lines A and B is the minimum equalization volume of traffic. This

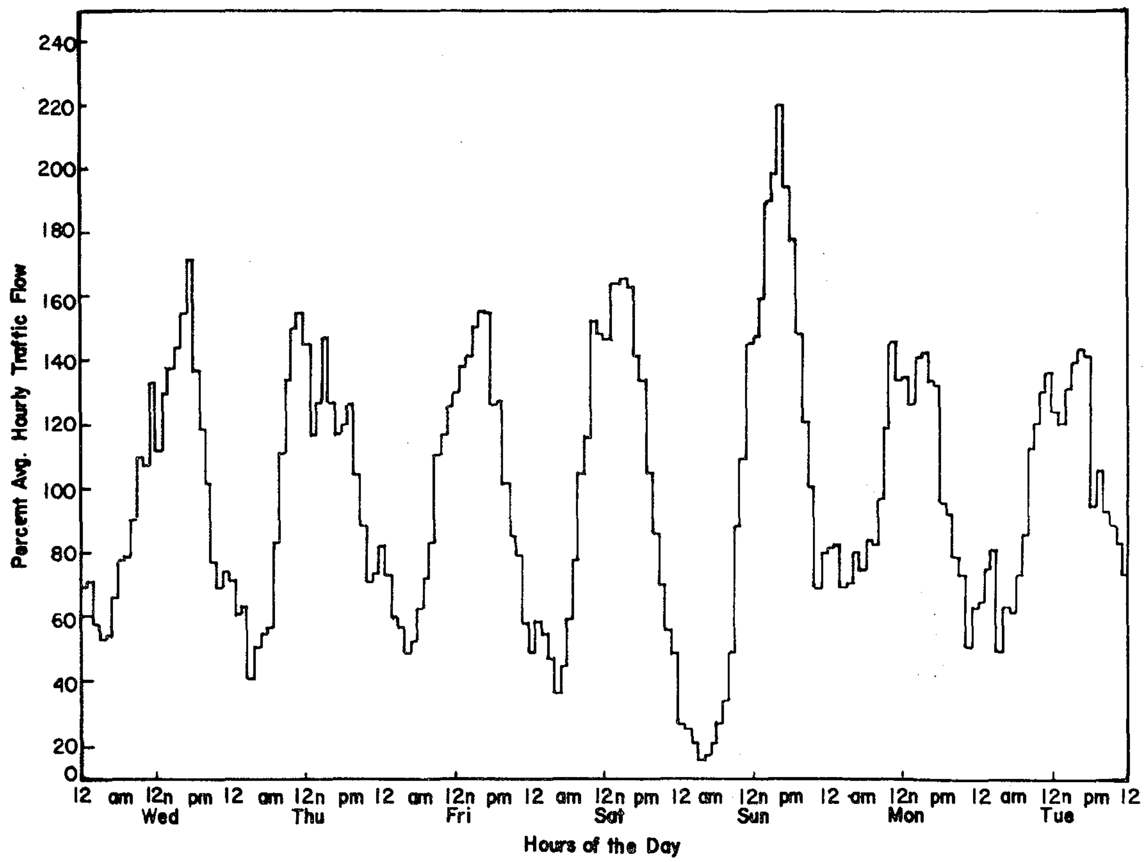


Figure 6-2. Example of hourly traffic volumes,
Illinois rest area.

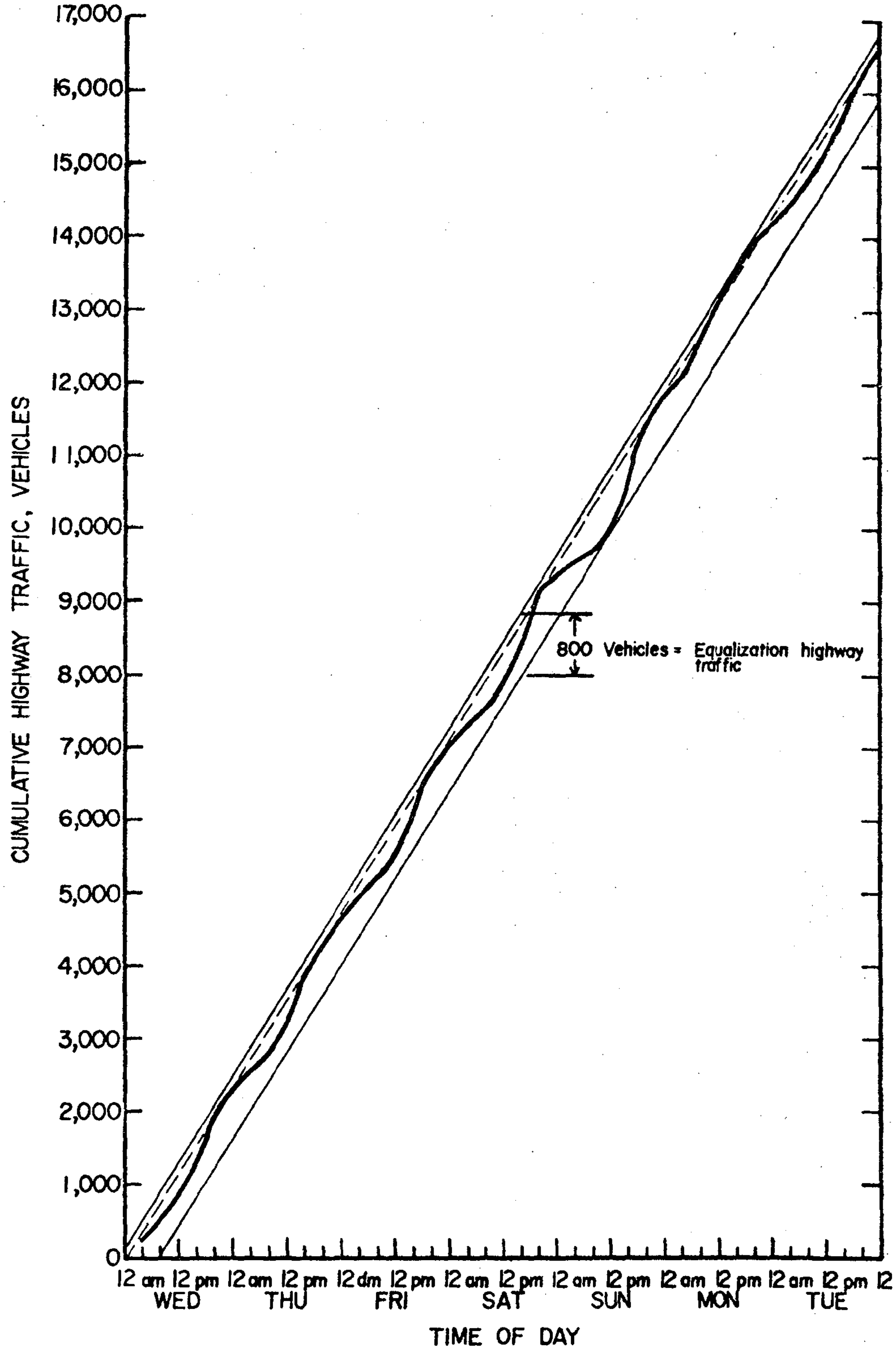


Figure 6-3. Cumulative flow diagram.

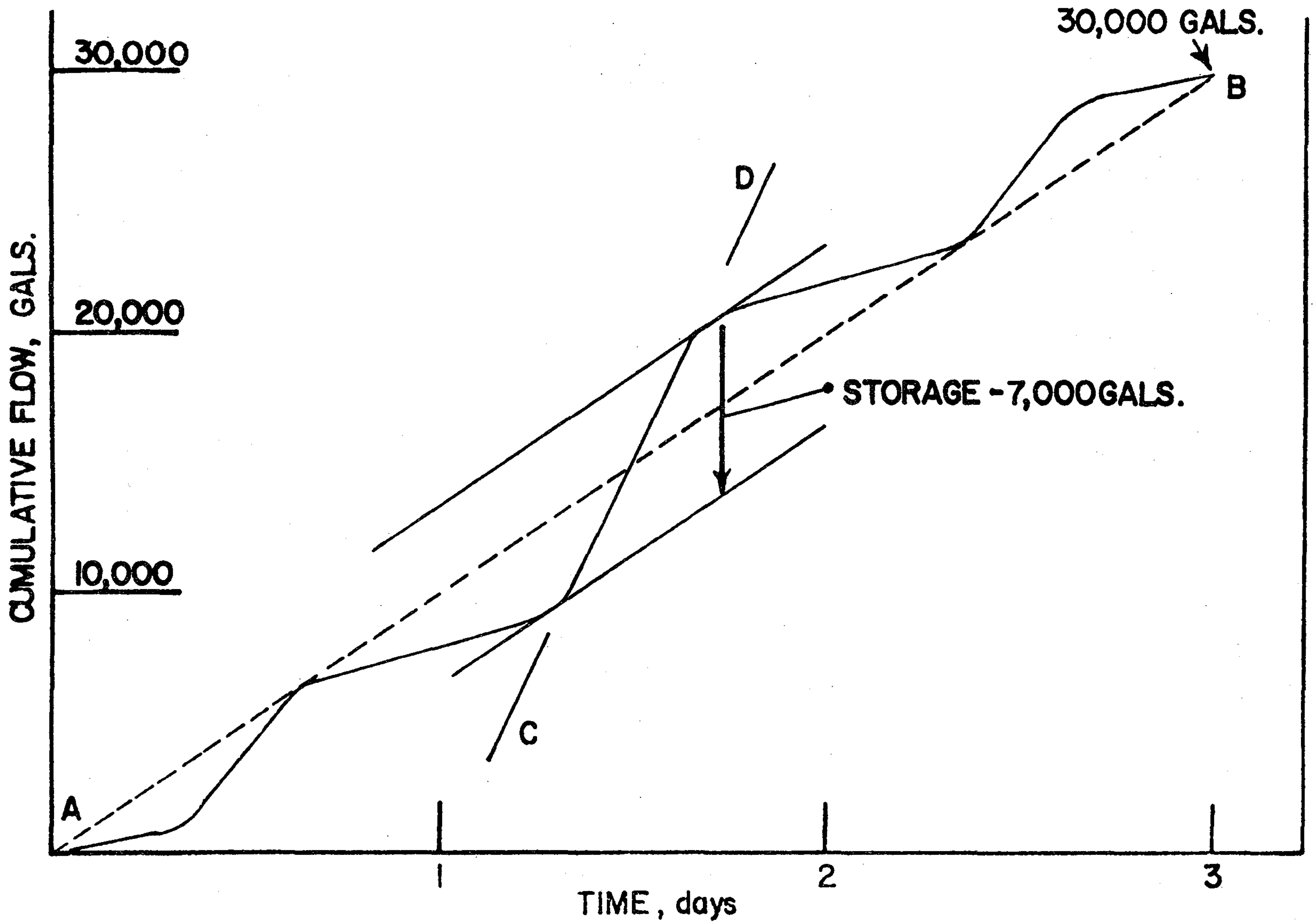


Figure 6-4. Simplified example of cumulative flow diagram.

equalization volume of traffic is then converted to flow by multiplying by 5.5 gal/veh.

The equalization volume calculated above is the minimum requirement for the time period evaluated. Design for future increases in average flow will require provision of additional capacity. Also, mixing and aeration equipment may require a minimum volume to be maintained in the tank. If future flows are projected to be considerably greater than present flows, the equalization basin should be staged or compartmented to allow for increased equalization capacity. For rest areas, provision for this flexibility should be considered on a seasonal basis as well.

Equalization tanks receive raw wastewater that has been subjected only to comminution or other preliminary treatment. The wastewater solids must be kept from settling in the tank since removal of settleable solids from the tank would increase the cost of equalization with little benefit to the downstream wastewater-treatment system. Additionally, detention of the wastewater in a quiescent tank may result in septic odor problems caused by anaerobic decomposition of the organics in the wastewater. Aeration and mixing may be provided by mechanical mixing, mechanical aeration, or diffused aeration. For smaller systems and for systems that already use diffused aeration for treatment processes, diffused aeration is the likely choice. If the equalization basin is large enough to warrant construction of an earthen basin, then floating surface aerators may be more advantageous. Air requirements to achieve complete mixing are normally in excess of that required for oxygen transfer. Metcalf and Eddy[5] report that 20 to 30 cfm/1000 cu ft of tank volume are required to ensure good mixing with a diffused aeration system. This requirement for mechanical aerators varies from 0.5 to 1.0 hp/1000 cu ft. Complete mixing may not be necessary; suspended solids will not settle out if 0.15 to 0.30 hp/1000 cu ft are maintained.[1] Variances in requirements are due to different aerator and basin designs and geometry of the basin. The oxygen transfer capacity of the aeration system should be checked against the oxygen uptake rate of the raw wastewater to insure that septic conditions will not develop. An uptake rate of 15 mg/ℓ/hr has been reported for domestic wastewater.[6] Aerobic conditions can be maintained by supplying air at a rate of 9.4 to 15 cfm/1000 cu ft of storage and Metcalf and Eddy suggest that the most economical design is to provide mixing equipment to hold solids in suspension and satisfy minimum oxygen requirements with a diffused aeration system.[1]

Perhaps the most difficult aspect of the design of flow equalization systems for the relatively small rest-area flows is the mechanism for metering wastewater flow into the plant at the average daily flow rate. Centrifugal sewage pumps capable of handling sewage solids will not operate efficiently at rates in the 10-gpm range typical of many rest-area flows. Allowing the pump to cycle on a timed schedule will provide flow during periods of little use, but the surges produced when the pumps begin operating may reduce to some extent the benefits of equalization. Pneumatic ejectors and air-lift pumps are often used in lieu of centrifugal pumps for smaller flows. Clogging is less of a problem with pneumatic ejectors than with air-lift pumps. In some cases, gravity discharge from the equalization tank to the treatment system may be possible, but an automatically controlled flow-regulating device would be required.[1] Instrumentation to regulate flow may be a significant cost to the system.

6-2. PERFORMANCE

A flow equalization basin will likely improve the performance of the wastewater-treatment facility by dampening peak loadings and reducing severe underloading conditions. Additionally, as much as 10 to 20 percent BOD reduction may occur in an in-line equalization system.[1] The tank will operate similarly to an aerated lagoon, except that the detention time and sludge age will be low and variable. The plant effluent discharged to the stream will be more uniform and will not degrade the stream's water quality as much as with an unequalized system. Equalization will also result in more uniform operation of treatment plant equipment, thereby increasing service life and reducing wear.

6-3. OPERATION

The equalization system will require some additional maintenance by the plant operator, but the reduced operational problems experienced by the rest of the plant should outweigh the time spent on the equalization system. Foaming and buildup of solids are the primary problems associated with the tank. Periodically, the tank must be drained to remove any buildup of grit or other sediment in the tank. Provision to use an alternate tank or bypass to the treatment plant will be necessary for cleaning the tank. Frequent inspection of any flow-regulating device and adjustments for increases or decreases in wastewater production are necessary.

6-4. DESIGN

1. Input required.

Variations in hourly flow or traffic flow for peak weekly period.

2. Design procedure.

a. Construct cumulative flow diagram as in Figure 6-3.

b. Determine equalization traffic volume as in Figure 6-3 and convert to wastewater volume.

$$\text{Wastewater volume} = \text{Equalization traffic} \times 0.09 \times 5.5 \text{ gal/veh}$$

where

0.09 = percent traffic stopping at rest area

5.5 = gallons of wastewater produced per vehicle stopping

c. Determine mixing requirements.

(1) Diffused aeration: 20-30 cfm/1000 cu ft.

(2) Mechanical mixing: 0.15-0.30 hp/1000 cu ft.

d. Determine oxygen transfer requirements for aerobic conditions. O_2 required = 9.4 to 15 cfm/1000 cu ft.

e. Select larger system calculated in c or d.

3. Example calculations.

a. From Figure 6-3 equalization volume = 800 vehicles.

b. Wastewater volume = 800 vehicles × 0.09 × 5.5 gal/veh
Wastewater volume = 396 gal.
For conservation let wastewater volume = 400 gal.

c. Determine mixing requirements.

(1) Wastewater volume = 400 gal = 53.5 cu ft.
Diffused aeration = 30 cfm/1000 cu ft.
Diffused aeration = 30 cfm × 53.5/1000.
Diffused aeration = 1.6 cfm.

d. Determine oxygen transfer requirements.

Oxygen requirement = 15 cfm/1000 cu ft.

Oxygen requirement = 15 cfm × $\frac{53.5}{1000}$.

Oxygen requirement = 0.8 cfm.

e. Select larger system.

Air requirement = 1.6 cfm

6-5. REFERENCES

1. Metcalf and Eddy, Inc., "Flow Equalization," May 1974, U. S. Environmental Protection Agency, Technology Transfer.

2. Gaines, F. R., "Equalizing Wastewater Flow," Water Pollution Control Federation Highlights, Vol 12, No. 10, Oct 1975.

3. Wallace, A. T., "Equalization," Process Design in Water Quality Engineering, New Concepts and Developments, E. L. Thackston and W. W. Eckenfelder, ed., Jenkins Publishing Co., New York, 1972.

4. Pfeffer, J. T., "Rest Area Wastewater Treatment and Disposal Survey of Existing Practices," Interim Report -- Phase I, Aug 1973, University of Illinois, Urbana, Ill.

5. Metcalf and Eddy, Inc., Wastewater Engineering, McGraw-Hill, Inc., New York, 1972.

6. Smith, J. M., Masse, A. M., and Feige, W. A., "Upgrading Existing Wastewater Treatment Plants," presented at Vanderbilt, Sep 1976, as reported in Metcalf and Eddy, Inc., "Flow Equalization," May 1974, U. S. Environmental Protection Agency, Technology Transfer.

7. SEPTIC TANK-ABSORPTION FIELDS

7-1. PROCESS DESCRIPTION

The primary purpose of the septic tank is to remove settleable and floatable materials. Wastewater enters directly into a septic tank where it is detained for a period of time, determined by the design of the tank size and actual wastewater flows. While in the septic tank, materials undergo anaerobic decomposition and their volume is reduced slightly. Other portions of the solids in the wastewater (particularly the toilet paper) rise to the surface and are retained in the septic tank in the form of a floating scum layer. The liquid fraction containing the unsettleable percentage of the suspended solids, bacteria, soluble organics, and nutrients (nitrogen and phosphorus) occupies the majority of the tank volume and is that portion of the wastewater that is to be discharged for further treatment. A well designed septic tank will provide an effluent relatively low in suspended solids, but high in organics, nutrients, and bacteria. A typical analysis of septic tank effluent is given in Table 7-1. <u>This effluent is not acceptable for direct discharge to surface waters.</u>

Table 7-1. Septic Tank Effluent Characteristics.[1]

Rest Area	pH	Kjeldahl Nitrogen as N	Nitrate as N	Suspended Solids mg/ℓ	COD mg/ℓ	BOD mg/ℓ	Settleable Solids mg/ℓ
1	8.70	237.2	0.25	49.0	217	103	0.1
2	8.70	272.8	0.45	66.2	233	135	0.1
3	7.78	221.1	0.0	58.4	221	133	0.1
4	7.50	163.6	0.1	78.8	264	160	0.4
Mean[a]	8.2	233.0	0.2	63.2	233	133	0.2

[a]One sample taken at each rest area.

The most commonly used method of disposing of the liquid fraction of the wastewaters from septic tanks is absorption by the soil in what has been called an "adsorption system" in this publication (Figure 7-1). Adsorption system is synonymous with "leach field" and refers to

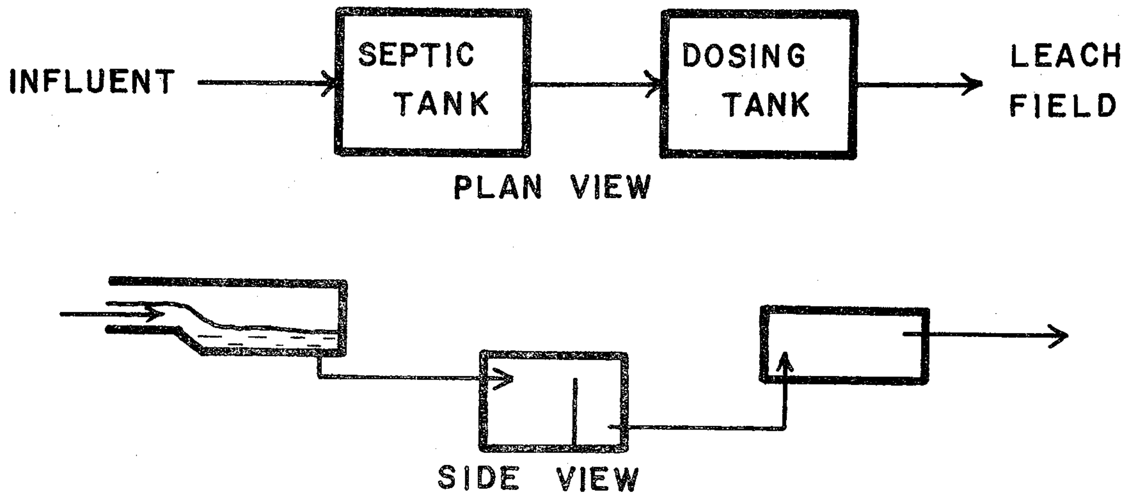


Figure 7-1. Septic tank leach field.

subsurface disposal of the wastewater and subsequent percolation into the groundwater with negligible loss attributable to evapotranspiration.

Little operational data are available on rest-area use of septic tanks. However, Sylvester and Seabloom[1] did monitor the influent and effluent of septic tanks showing the reduction of suspended solids, chemical oxygen demand, and biochemical oxygen demand (Table 7-2). From

Table 7-2. Septic Tank Reduction of Wastewater Parameters.[1]

Parameters	SS mg/ℓ	COD mg/ℓ	BOD_5 mg/ℓ
Septic Tank Influent Mean	165	405	165
Septic Tank Effluent Mean	63	233	133
Percent Reduction	62	43	20

this work they were able to recommend the use of septic tanks followed by leach fields in areas where the soil porosity is high and where the groundwater table is low. Data collected in the WES survey (Table 2-5) have shown the use of septic tanks followed by leach fields to be one of the most frequently used forms of sewage treatment employed at rest areas. Problems encountered at rest areas with septic tank leach fields are mainly due to improper sizing, construction, and operation of leach fields and inadequate soil investigations.

7-2. ABSORPTION SYSTEMS

Septic tank effluent may be disposed of in three types of soil absorption systems: narrow trench, seepage bed, or seepage pit. The narrow trench system (Figure 7-2) employs trenches 12 to 18 in. wide

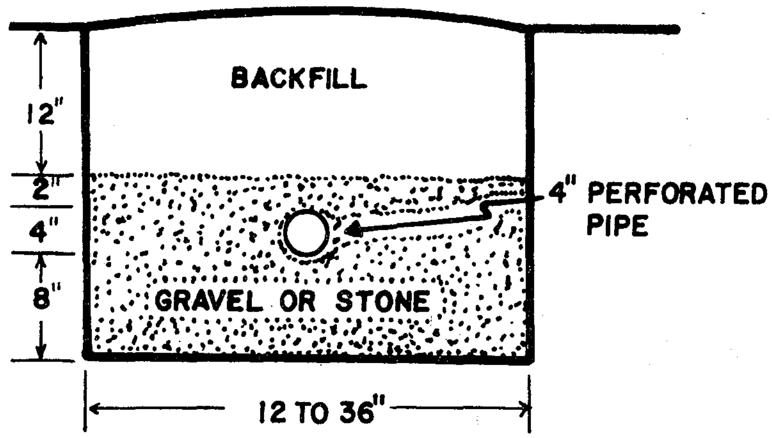


Figure 7-2. Narrow trench system.

with a 4-in. tile line (perforated or open jointed) resting on 8 in. of coarse material, gravel or stone, in the bottom of the trench. The depth of the trench determines the type of treatment provided. Shallow trenches 2 to 3 ft deep are preferable since this will allow aerobic biological activity for conversion of organic material to less harmful, stable materials. In deeper systems, a design required for cold regions where the frost line penetrates several feet below the surface, little opportunity for aerobic stabilization of organics is available. However, partial anaerobic treatment will occur which may lead to the percolation of insufficiently treated wastewater to the groundwater. Therefore, care must be taken to design a system that is below the frost line yet has its bottom at least 4 feet above the high water table. In this manner only sufficiently treated wastewater reaches the groundwater. The shallow, narrow trench system is recommended because it provides maximum infiltrative surface in the aerobic zone activity.

The seepage bed (Figure 7-3) or wide trench system employs trenches

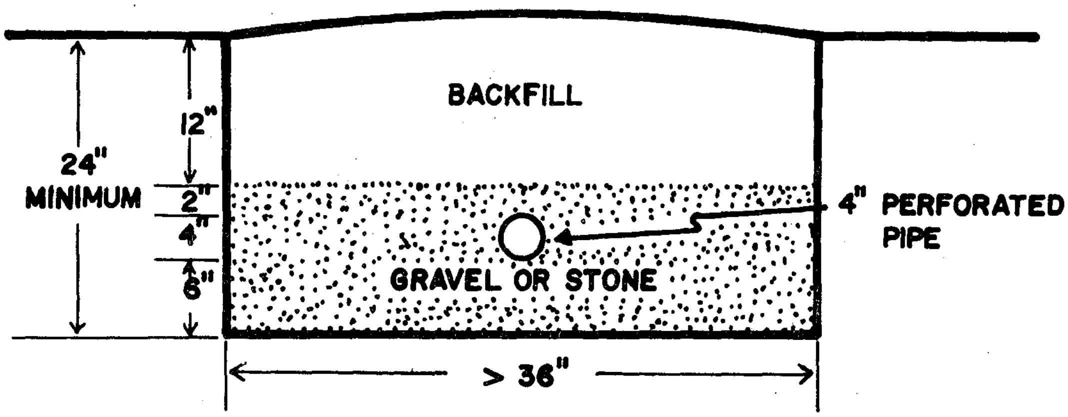


Figure 7-3. Seepage bed.

2 to 3 ft wide or wider. Little sidewall area is provided per volume of trench; therefore, the effective infiltrative surface is less than that for a narrow trench system of similar area. Construction may be simpler, but the degree of treatment and life of the field will be less than those for a narrow trench system.

The seepage pit system is a deep pit extending as far as 70 ft into the ground. Little aerobic treatment is possible and unstable material including organics, ammonium, and bacteria may be discharged to the groundwater. <u>The seepage pit is not recommended for rest areas.</u>

7-3. PERFORMANCE

Septic tank leach field systems are capable of effectively disposing of and treating rest-area-generated wastewater in any of the FHWA regions provided that they are properly designed and constructed. If the procedures outlined in the design section are followed, with particular emphasis placed on thorough soil investigations and proper and thorough inspections and supervision of construction and installation, then septic tank leach field systems can be used for treating rest-area-generated wastewater and will be in compliance with the requirements of PL 92-500.

Septic tanks are ideally suited for use at rest areas, particularly those that experience only limited use. Septic tanks function satisfactorily in cold as well as warm climates provided that the leach fields have been designed to operate below the frost line in cold climates. Excessive rainfall may cause failure of a leach field if the soil becomes saturated. However, if the leach field is designed with a properly graded surface most rainfall will run off and the leach field will continue to operate properly.

Even though large fluctuations in wastewater flow are experienced at rest areas, septic tank leach field systems can perform adequately when the septic tank has been properly sized to handle the expected peak flows. If the septic tank is undersized, solids may be washed out of the tank during periods of peak flow and may ultimately clog the leach field.

7-4. FLEXIBILITY

Septic tank leach field systems do not readily lend themselves to expansion. Because the system is constructed underground, expansion would require the excavation and replacement of the existing septic tank. It would also require the lengthening of existing leach field lines or the construction of additional leach fields. A method for adding flexibility to the septic tank leach field system is the installation of a splitter box on the effluent line from the septic tank. In this manner as the first leach field becomes hydraulically overloaded or clogged flow may be diverted to a second leach field.

7-5. RELIABILITY

Septic tank leach field systems can consistently and effectively treat the wastewater produced at a rest area. Leach fields should operate up to 20 years without clogging.

7-6. OPERATION

Failure of absorption systems is normally caused by clogging of the infiltrative surface. Sometimes high seasonal groundwater conditions result in pooling of septic wastewater in trenches and seepage to the surface. The volume of effluent that can be absorbed by the soil is related to available surface area, characteristics of the soil, and reactions between the constituents of the soil and the wastewater. Traditionally the ability of a soil to accept waste effluent is measured by the percolation test, which was first developed by Henry Ryon in 1926.[2] Simply described, this test measures the amount of clean water that can move through the soil in a given time. A description of the test as recommended by U. S. Public Health Service is given in Reference 3. While this test does define the percolative capacity of the soil, it does not define the infiltrative capacity of the soil. Infiltration as defined by McGauhey[4] is the rate at which liquid will pass through the soilwater interface. Percolative and infiltrative capacity are equal at the beginning of an operation, but chemical, biological, and physical phenomena change the characteristics of the first few inches of soil and may reduce the infiltrative capacity to the point of completely clogging the soil surface. Clogging results in unsaturated conditions in the soil below the surface thereby reducing the hydraulic conductivity of the soil to less than that for saturated conditions as measured by permeability tests. The most significant cause of soil clogging is anaerobic biological activity which produces slimes, ferrous sulfide, and polysaccharides that form an impermeable crust on the surface. These materials may be formed irrespective of the type of soil present when oxygen is continuously excluded from the soil. Therefore, the rate at which septic tank effluent can be applied to the soil cannot be directly computed from the percolation test. Henry Ryon performed his test on a number of sites in New York State; he evaluated existing soil absorption systems and related the success or failure of these systems to the results of the percolation tests. The results indicated that infiltrative rates were significantly less than the percolation rates. A curve was developed for Ryon's data relating loading rates that did not produce system failure to percolation test measurements. The results of Ryon's study have been used for design purposes even when conditions have been significantly different.

Septic tank leach field systems are perhaps the simplest wastewater treatment systems to operate. Once every six months (if adequately sized) the septic tank should be investigated to determine the thickness of the scum layer and the depth of the sludge layer. If the bottom of the scum layer or the top of sludge layer is approaching the bottom of the outlet structure (Figure 7-1) then the contents of the tank must be pumped out; this is normally accomplished by a local septic tank clean-out service.

Operation of alternating leach fields is accomplished by switching the flow from one field to another. This duty and the septic tank investigation can be performed by the person performing custodial service on the rest area. It is recommended that leach fields be switched every 7 days. This allows each field to dry out and become aerobic.

Among operational problems that may occur in a septic tank leach field system at a rest area is the clogging of the leach field. This will become evident by the surfacing of wastewater in the leach field or the backup of the toilets in the rest rooms. If such conditions occur they must be immediately reported to the State highway department and remedial action taken. A recently recommended method of unclogging leach fields has been the addition of an oxidant into the leach field. Because of its low cost the recommended oxidant has been hydrogen peroxide.[6]

Hydrogen peroxide treatment of leach fields may be performed in two manners; it may be added to a clogged field for remedial action or it may be added periodically as preventive action. When hydrogen peroxide is used on a clogged field all wastewater must be removed from the field. This is performed by excavating a pit near the field and pumping all water from the pit. After the leach field has been pumped dry hydrogen peroxide (which is obtained commercially at 50% solution) is diluted with water using one part hydrogen peroxide to every 20 to 40 parts water. This mixture is then added to the leach field just prior to the leach field lines. The solution of hydrogen peroxide and water is added continuously to the field until the field is no longer clogged and the water being added goes into the surrounding soil and does not pond.

As a preventive action hydrogen peroxide may be added to the leach field system each time the septic tank is pumped out. In this manner proper functioning of both the septic tank and the leach field may be assured.

7-7. PRELIMINARY DESIGN

The most widely used reference for the sizing of septic tanks and leach fields is the USPHS Manual of Septic-Tank Practice.[3] From this and other design manuals the correct size of a septic tank for a given flow can be determined. However, it should be noted that it is better to oversize a septic tank than to undersize one. Since the primary purpose of a septic tank is to remove settleable solids (with anaerobic digestion of the settled material), oversizing may increase retention times and thus promote settling. Undersizing, however, may allow for some short-circuiting or washout of solids during periods of high flow and allow suspended solids to enter the leach field (or sand filter) and thus clog the infiltrative surfaces. For larger leach fields, i.e., those with greater then 500 lineal ft of tile, the USPHS recommends that a dosing tank be used to optimize distribution of sewage throughout the leach field and to give the absorption lines time to dry out between loadings. If the leach field is so designed that the field lines cannot dry out between loadings (i.e. the leach field is continuously receiving wastewater), an alternate field should be provided whereby two siphons can alternately dose the two absorption fields. The rest period prolongs the life of the leach field by allowing the system to dry out and become aerobic, thus minimizing the clogging of the soil that occurs under anaerobic conditions. There is no set method for determining when a leach field has dried out. One manner of determining this would be to take moisture readings at various depths of the leach field during the period of nonuse. This may be achieved by taking a soil sample and noting the moisture content at various depths throughout the period of nonuse.

The primary parameter to be determined for an absorption system is the land area required to treat a given volume of wastewater. This area is directly related to the characteristics of the soil, the type of operation, configuration of the field, construction procedures, and type of effluent applied. Soil characteristics are site dependent and, therefore, must be quantified as to their absorption capacity.

Evaluation of the soil characteristics should begin with collection of available soil maps of the area in question, history of septic tank usage in the area or on comparable soils, and soil borings to fill data gaps. Soil maps may be obtained from the Soil Conservation Service (SCS). A history of septic tank failures in the area may be available from either the SCS or the local or state health environmental organization. A history of septic tank performance may also be obtained by interviewing local owners of septic tanks. A soil scientist or engineer familiar with soils and their ability to accept septic tank effluent should be consulted in order to qualitatively determine if evaluation of an absorption system may proceed. If this initial evaluation proves the feasibility of this process, then the absorptive capacity of the specific area available for the absorption system must be determined.

Although the limitations of the percolation test are recognized, it continues to be used by regulatory agencies in sizing absorption areas. Methods to replace the percolation test with other measurements, most notably the crust test for hydraulic conductivity (Bouma),[5] have been proposed. The crust test attempts to relate the conditions that will exist after septic tank effluent reacts with soil constituents to form a crust resulting in an unsaturated soil beneath the crust. The percolation test saturates the soil resulting in faster rates of percolation through the soil. The percolation test measures the rate of fall of a water column in the soil and is normally reported in minutes per inch. The USPHS requires the percolation rate to be faster than 60 min/in. for absorption trenches. If the rate is faster than 1 min/in. then retention in the soil may not be sufficient to remove bacteria and unstable organics thereby increasing the chances of groundwater pollution. <u>Note: Absorption systems should not be located where the seasonal high groundwater table or impervious stratum is within 4 ft of the trench bottom.</u> Guidance for conducting the percolation test is given in the Manual of Septic-Tank Practice.[3]

a. Septic tanks may be designed using the following procedure.

(1) Input required:

Q = average daily flow, gpd

(2) Design criteria:

(a) Divide septic tank into a minimum of two compartments with the first compartments being 50 to 67 percent of the total volume.

(b) The depth of liquid in the tank should be 30 to 60 in. with 20 percent of the total depth of the tank being left for freeboard at the top.

(c) The smallest plan dimension should be 4 ft.

(d) The inlet device should meet the following specifications:

1. Invert elevation should be at least 3 in. above liquid level.

2. A vented tee or baffle should be installed to divert incoming sewage downward. The baffle should extend at least 6 in. below the liquid level, but it must be 3 in. higher than the outlet baffle (see Figure 7-4).

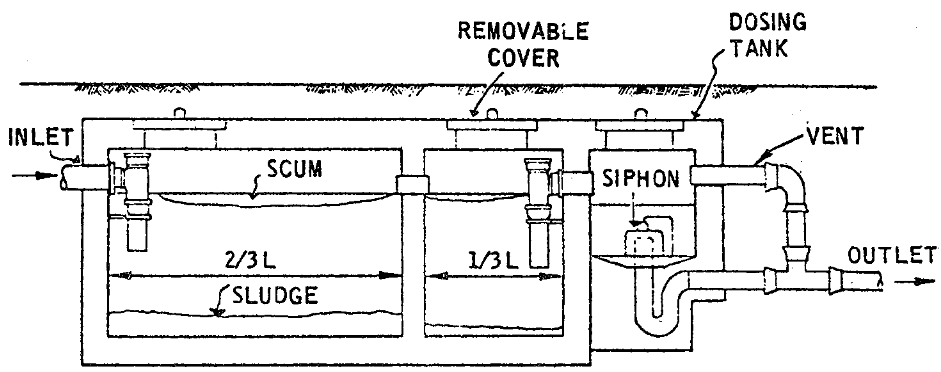


Figure 7-4. Septic tank with dosing tank.

(e) The outlet device should include a baffle that extends below the liquid to a depth equal to 40 percent of the liquid depth and above the liquid line to within 1 in. of the top of the tank.

(f) Access manholes or ports should be provided for each tank compartment, inlet, and outlet in order to check sludge and scum accumulation.

(3) Design procedure:

(a) Use 1.25 × average daily flow (Q) for design flow.

(b) Calculate volume (V) of septic tank, in gallons. Note: The smallest size tank for use at rest areas shall be 1500 gals.

1. If $Q \times 1.25 \leq 1500$ gpd, $V = 1500$ gallons

2. If $Q \times 1.25 \geq 1500$ gpd, $V = 1125 + 0.75\ (Q \times 1.25)$

b. <u>Absorption fields may be designed using the following Procedure.</u>

(1) Input required:

Q = average daily flow, gpd

t = percolation time, min/in.

(2) Design criteria:

(a) "Percolation" rate must be between 5 and 30 minutes per inch. (This converts to maximum and minimum infiltration rates of 2.2 to 0.9 gal/sq ft/day) (Figure 7-5).

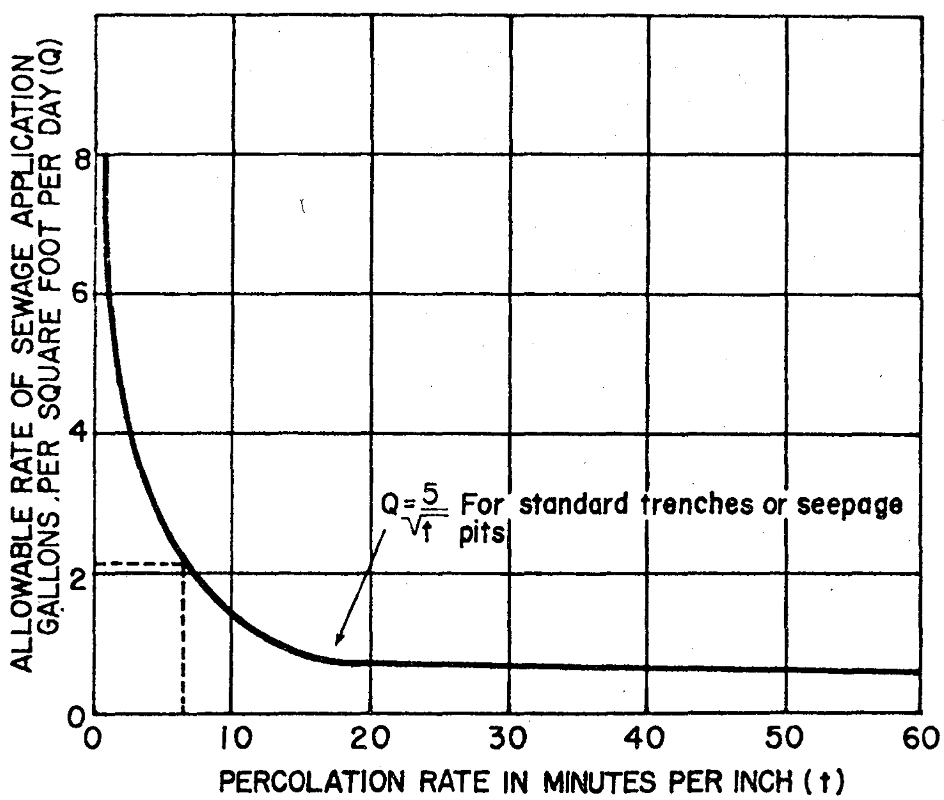


Figure 7-5. Determination of percolation rate.

(b) Seasonal high groundwater table must be greater than 4 ft below bottom of trench.

(c) Absorption field must be at least 100 ft from source of water supply and at least 50 ft from surface waters. Whenever possible adsorption fields should be located as distant as possible from water supplies. They must also be located at a lower elevation than the water supply. In this manner possible contamination of a water supply by wastewater from an adsorption field will be minimized.

(d) The drain tile field distribution lines should be laid level.

(e) The length of any individual line should not exceed 100 feet.

(3) Design procedures:

Application rates for soil absorption trenches have been determined from empirical data and described by the equation:

$$Q_a = \frac{5}{\sqrt{t}} \qquad (7\text{-}1)$$

where

Q_a = allowable rate of application of wastewater, gal per sq ft of trench sidewall area per day

t = percolation time, min/in.

McGauhey[4] stresses that the sidewall area is the most effective absorptive surface of the trench. Intermittent loading of the system, with sufficient intervals between loadings to allow drying, permits aerobic conditions to be established at the infiltrative surface on the sidewalls. The bottom area may remain inundated and be clogged by the polysaccharide and ferrous sulfide slime layers created by anaerobic conditions. Equation 7-1 includes a statistical allowance for the sidewall area of a 2-ft-wide trench. Application rates for deeper or wider trenches must be adjusted for the increase in infiltrative surface. Factors for this adjustment are given in the Manual of Septic-Tank Practice.[3] The maximum infiltration rate used should not exceed 2.2 gals/sq ft/day, and if the infiltration rate is less than 0.9 gal/sq ft/day, septic tank systems should not be used.

Only the sidewall area of the trench below the invert of the pipe should be used in determining the length of trench required. The sidewall depth below the pipe inlet should be between 6 in. and 4 ft. The infiltration rate used in design should be the minimum value of the soils encountered in this region.

The trench sidewall area should be designed for a peak factor of at least 2.

The design flow for sizing a septic tank absorption field system should be 1.25 times the average daily flow (Q, gpd). Then trench sidewall area A_t (in sq ft) can be determined as follows:

$$A_t = \frac{1.25Q}{Q_a} \times \text{peak factor} \qquad (7\text{-}2)$$

The length of tile drain required, L (in ft), is given by:

$$L = \frac{A_t}{2 \times \text{depth below pipe invert}} \qquad (7\text{-}4)$$

Laterals should not be greater than 100 ft in length, and for rest-area flows many laterals may be required. The spacing between laterals must be at least twice the depth of gravel in the trench in order to allow for percolation of effluent through the sidewalls. As a matter of practice due to construction considerations, 6-ft centers, as a minimum, are generally used. The total land surface area (A) of the field would then be given by:

$$A = L \times 6 \qquad (7\text{-}4)$$

If dosing siphons are to be used and two adsorption fields provided then the total land surface area will be doubled. When dosing siphons are used the following criteria should be met.

(a) A dosing siphon or pump should be provided to flood the drain tile field. The dosing rate should be at least twice the design flow rate. In the rest-area case, it is suggested that twice the peak hourly rate of flow be used. Alternating siphons should always be used when the length of trench required exceeds 500 ft. Alternating siphons should discharge into two separate and identically sized fields.

(b) The effective volume of the dosing tank should equal 60 to 75 percent of the interior volume of the distributor pipes to be dosed at any one time.

Absorption trenches usually consist of 4-in.-diameter pipe in 2- to 3-ft lengths for vitrified clay sewer pipe, or longer sections of perforated nonmetallic pipe. The trench bottom and tile distribution lines should be level in order to provide uniform distribution over the field. Trenches would normally be laid parallel to topographic conditions in order to have a uniform trench depth. However, if the terrain slopes significantly, successive parallel trenches at a higher elevation would be progressively deeper in order to maintain equal elevations for the field lines. At least 12 in. of coarse aggregate or gravel should be in the bottom of the trench with 6 in. below the bottom of the pipe and 2 in. above the top of the pipe. A 12-in. layer of earth should cover the layer of gravel. The trench filter material should contain enough void space to store one day's design flow. The percent of voids of the filter material must be determined. The volume of filter material required can be determined by the equation:

$$\text{Volume of filter material} = \frac{\text{Design daily flow}}{\text{Percent of voids}}$$

The trench width may be determined

$$\text{Trench width} = \frac{\text{Volume of filter material}}{\text{trench length} \times \text{trench depth}}$$

The trench width should never be less than 16 inches.

When digging the trenches, care must be taken to avoid smearing the sides and packing loose material in the trench bottom. This may require scarifying the sides after digging and carefully removing the loose material. When the pipe has been laid and fill is added, the trench should not be packed with machinery.

(4) Output obtained:

V = volume of septic tank, gal

Q_a = allowable rate of application of wastewater, gal per sq ft of trench sidewall area per day

A_t = area of trench bottom, sq ft

L = length of tile drain, ft

A = total area of leach field, sq ft

(5) Example calculations (Figure 7-5):

$Q = 6000$ gpd

(a) $V = 1125 + 0.75\ (1.25 \times Q)$

$V = 1125 + 0.75\ (1.25 \times 6000)$

$V = 6750$ gal

(b) $Q_a = 5/\sqrt{t}$; t has been determined to be 5 min/in.

$Q_a = 5/\sqrt{5}$

$Q_a = 2.2$ gal per sq ft per day

(c) $A_t = \frac{1.25\ Q \times \text{peak factor}}{Q_a}$; peak factor = 2

$$A_t = \frac{1.25(6000) \times 2}{2.2}$$

$A_t = 6818$ gals

(d) Assume depth below pipe invert = 2 ft

$L = A_t/2 \times$ depth below pipe invert

$L = 6818/2 \times 2$

$L = 1705$ ft

Use two fields and a dosing tank.

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8. LAGOONS

8-1. INTRODUCTION

Lagoons have been employed as either a secondary or teritary treatment method at rest areas in 11 of the 21 states visited by the WES survey team. The different types of ponds utilized were aerobic, facultative, and totally evaporative. The evaporative type was used only in the west and midwest regions where evaporation exceeds rainfall. In many states the use of lagoons is no longer approved by the pollution control regulatory agencies as an effective means of treating wastewater. However, because there are many rest areas still employing lagoons as a means of wastewater treatment, and because it is possible to upgrade these ponds to meet the 1977 requirements of PL 92-500 and state water quality criteria, this section will describe the design criteria normally employed in designing ponds of three types--facultative, facultative-aerated, and totally evaporative. Various techniques available for the upgrading of lagoon effluent are described in Sections 13 and 14. <u>It should be pointed out that the degree of treatment achieved in most facultative lagoons will not produce an effluent that is capable of meeting the requirements of PL 92-500.</u> Therefore, those techniques described in Sections 13 and 14 should be incorporated with the lagoon to comply with the 1977 requirements of PL 92-500. It must also be pointed out that the use of lagoons at rest areas may require large land areas, depending on flow rate and local climatic conditions. This may preclude the use of lagoons at a particular rest area.

8-2. PROCESS DESCRIPTION

<u>8-2-1. Facultative lagoons.</u> Facultative lagoons are man-made earthen basins filled with wastewater where bacteria metabolize the organic matter (BOD) present for energy- and growth-producing carbon dioxide and water (Figure 8-1). Facultative lagoons are characterized by three layers of biological activity: a top aerobic layer, a bottom anaerobic layer, and an intermediate layer that fluctuates between aerobic and anerobic conditions and is populated by facultative bacteria.

As the wastewater enters the lagoon a portion of the solids settle to the bottom and undergo anaerobic decomposition where bacteria present use the organic matter (BOD) in a two-stage process to first produce organic acids and finally to produce methane (CH_4), ammonia (NH_4), carbon dioxide (CO_2), and hydrogen (H_2). The remainder of the organic matter in the wastewater is used by the aerobic bacteria for energy and growth with the final production of carbon dioxide and water. In the intermediate layer, facultative bacteria use the organic matter present

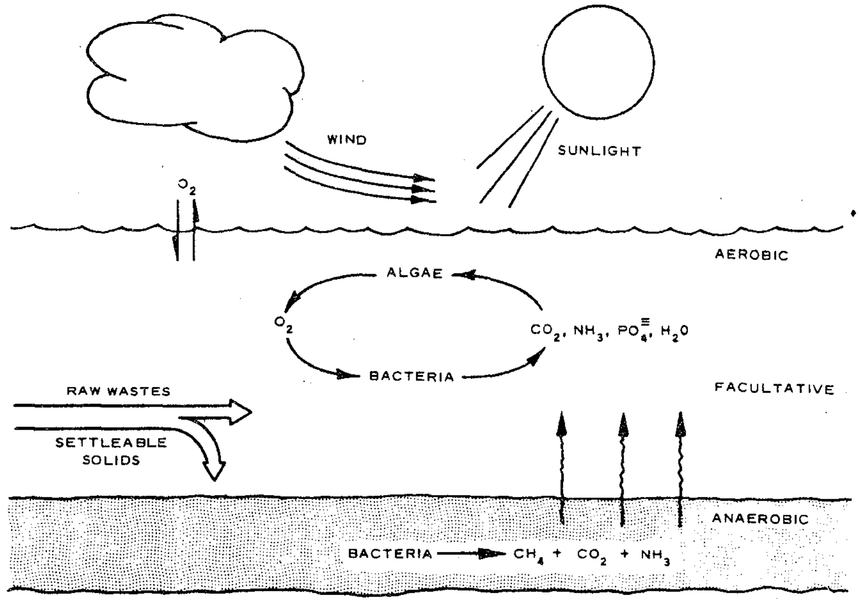

Figure 8-1. Schematic diagram of oxidation pond symbiosis between bacteria and algae.

for energy- and growth-producing carbon dioxide and water. The carbon dioxide produced by the bacteria is used by algae in the presence of sunlight and inorganic minerals present in the wastewater to produce protoplasm and oxygen. The oxygen produced by the algae, along with the oxygen present from surface aeration, is used by the aerobic and facultative bacteria to complete the cycle.

In this manner organic matter (BOD) entering a facultative lagoon is reduced for energy. However, the resulting algae may constitute a BOD as great as the untreated wastewater and for this reason must be removed from the effluent before discharge can be allowed.

8-2-2. Facultative-aerated lagoons. Facultative-aerated lagoons also consist of a man-made earthen basin containing a bacterial mass capable of metabolizing the organic matter present in the incoming wastewater. However, in facultative-aerated lagoons, mechanical aeration (either surface or diffused) is employed to supply additional oxygen to the bacteria. By supplying additional oxygen to the bacteria and mixing the lagoon contents, two functions are served. First, more of the bacteria come into contact with the wastewater in a shorter period of time than in a facultative lagoon and second, the availability of oxygen to the aerobic bacteria is increased. The bottom of the lagoon contains an anaerobic layer of bacteria.

8-2-3. Totally evaporative lagoons. Totally evaporative lagoons operate in much the same manner as facultative lagoons. However, in totally evaporative lagoons there is no effluent discharge; the only mechanism for reducing wastewater volume is evaporation. Thus, totally evaporative lagoons are constructed only where total yearly evaporation exceeds total yearly rainfall.

8-3. PERFORMANCE

8-3-1. Facultative lagoons. Facultative lagoons use bacteria for reduction of organic matter (BOD) present in the wastewater. The ability of the bacteria to use organic matter is a function of temperature, with a decreased metabolic rate experienced with decreasing temperatures. Thus, a reduced treatment efficiency is experienced in northern climates, particularly during the winter when ice may cover the lagoon. Because of decreased biological activity during periods of low temperature most facultative lagoons are designed with long detention times.

With the long detention times common to facultative lagoons, they have the ability to withstand the fluctuating wastewater flows experienced at rest areas. Also, because of their long detention times they are able to withstand a fluctuating organic loading and still produce a consistent degree of treatment. However, it should be pointed out that the degree of treatment achieved in most facultative lagoons will not produce an effluent that is capable of meeting the requirements of PL 92-500.

8-3-2. Facultative-aerated lagoons. Facultative-aerated lagoons are also able to withstand widely fluctuating organic and hydraulic loadings. Facultative-aerated lagoons are dependent on biological activity for reducing the amount of organic matter present in the wastewater. Therefore, the treatment efficiency is dependent on temperature.

The dependence of biological activity and thus treatment efficiency on temperature can best be shown from the following formulations.

For design of a single-cell facultative-aerated lagoon:

$$\text{BOD removal efficiency} = \frac{S_e \times 100\%}{S_o} = \frac{1}{1 + Kt} \times f \qquad (8\text{-}1)$$

where

S_e = effluent soluble BOD (not including the BOD in the algae), mg/ℓ

S_o = influent BOD, mg/ℓ

K = reaction rate constant, day^{-1}

t = detention time, days

f = seasonal correction factor, 1.2 for summer, 1.05 for winter

and the reaction rate constant K is expressed as

$$K = K_{20}(\Theta^{T-20}) \text{ (Figures 8-2 through 8-7)} \qquad (8\text{-}2)$$

where

K_{20} = reaction rate at 20°C (from laboratory tests) (usually 0.75)

Θ = temperature coefficient (from laboratory tests) 1.02 to 1.10 (usually 1.075)

T = design temperature, °C, average for winter or for summer

It should be pointed out that facultative lagoons and facultative-aerated lagoons have been used in northern climates to treat wastewater. However, detention time, and thus lagoon size, must be increased to compensate for periods of lowered biological activity.

8-3-3. Totally evaporative lagoons. Totally evaporative lagoons, because they have no discharges, are not as dependent upon temperature They are, however, dependent upon the amount of precipitation that occurs in relation to the amount of evaporation that is taking place. Because of this, totally evaporative lagoons can only be installed and

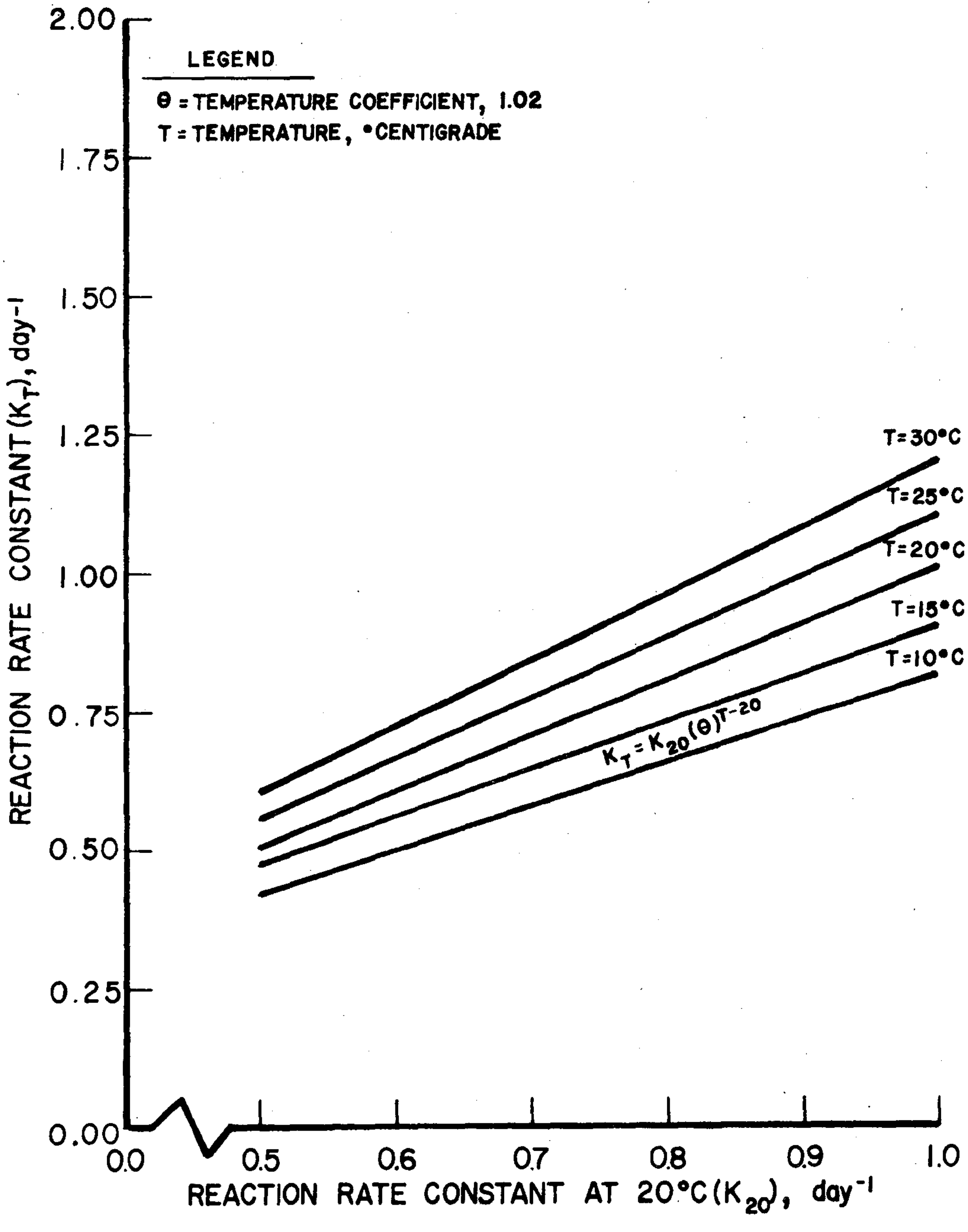

Figure 8-2. Reaction rate constant, $\Theta = 1.02$.

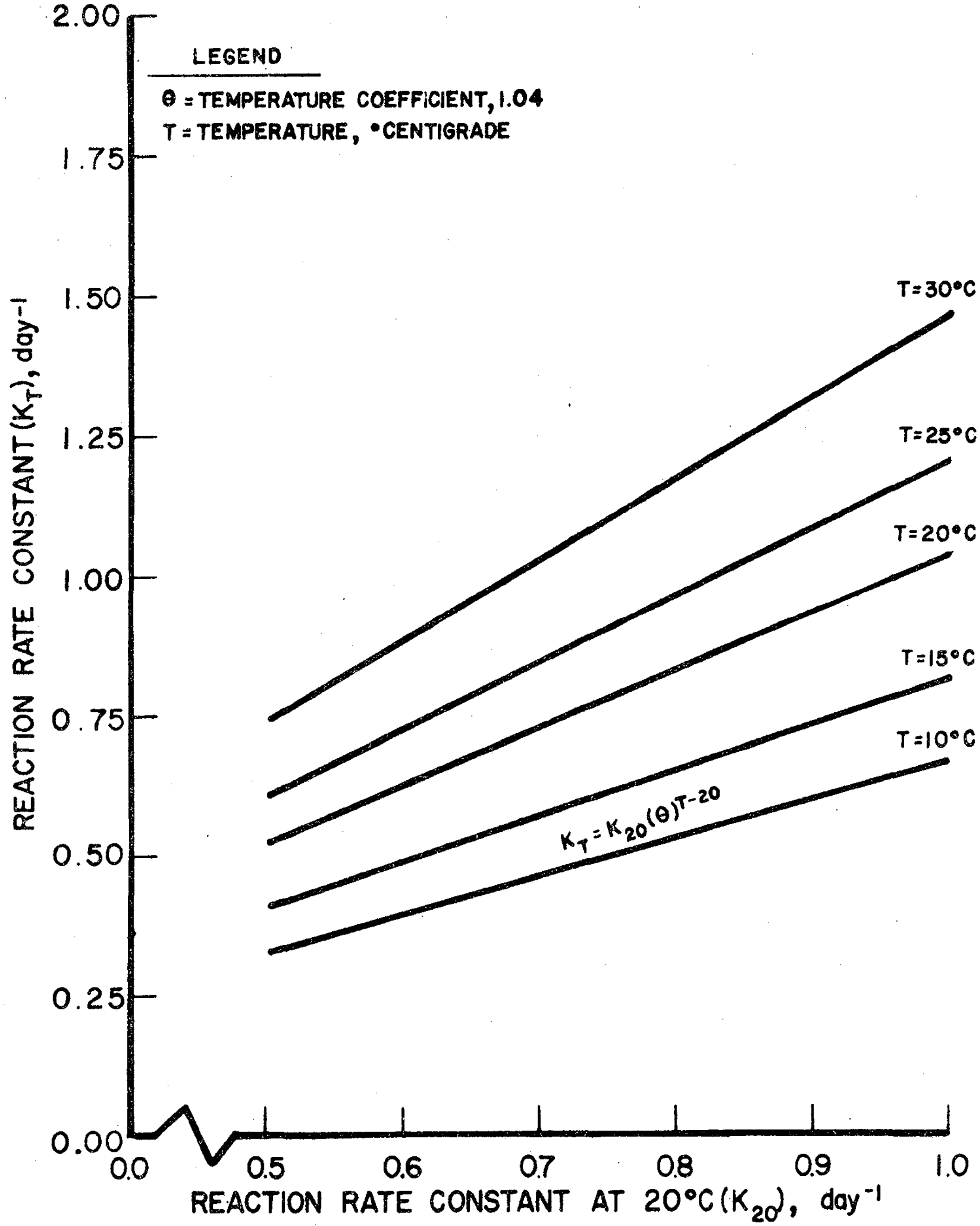

Figure 8-3. Reaction rate constant, Θ = 1.04.

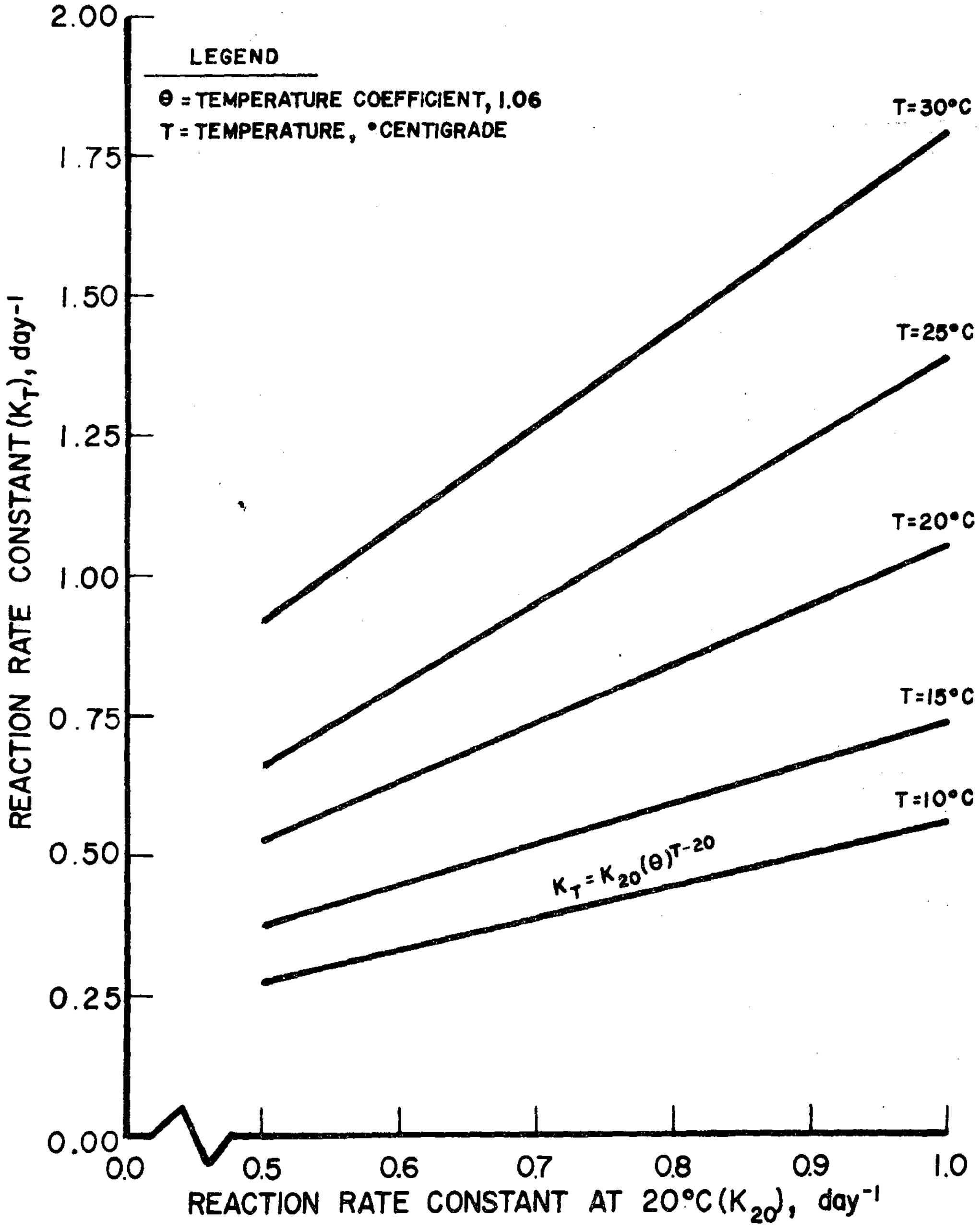

Figure 8-4. Reaction rate constant, Θ = 1.06.

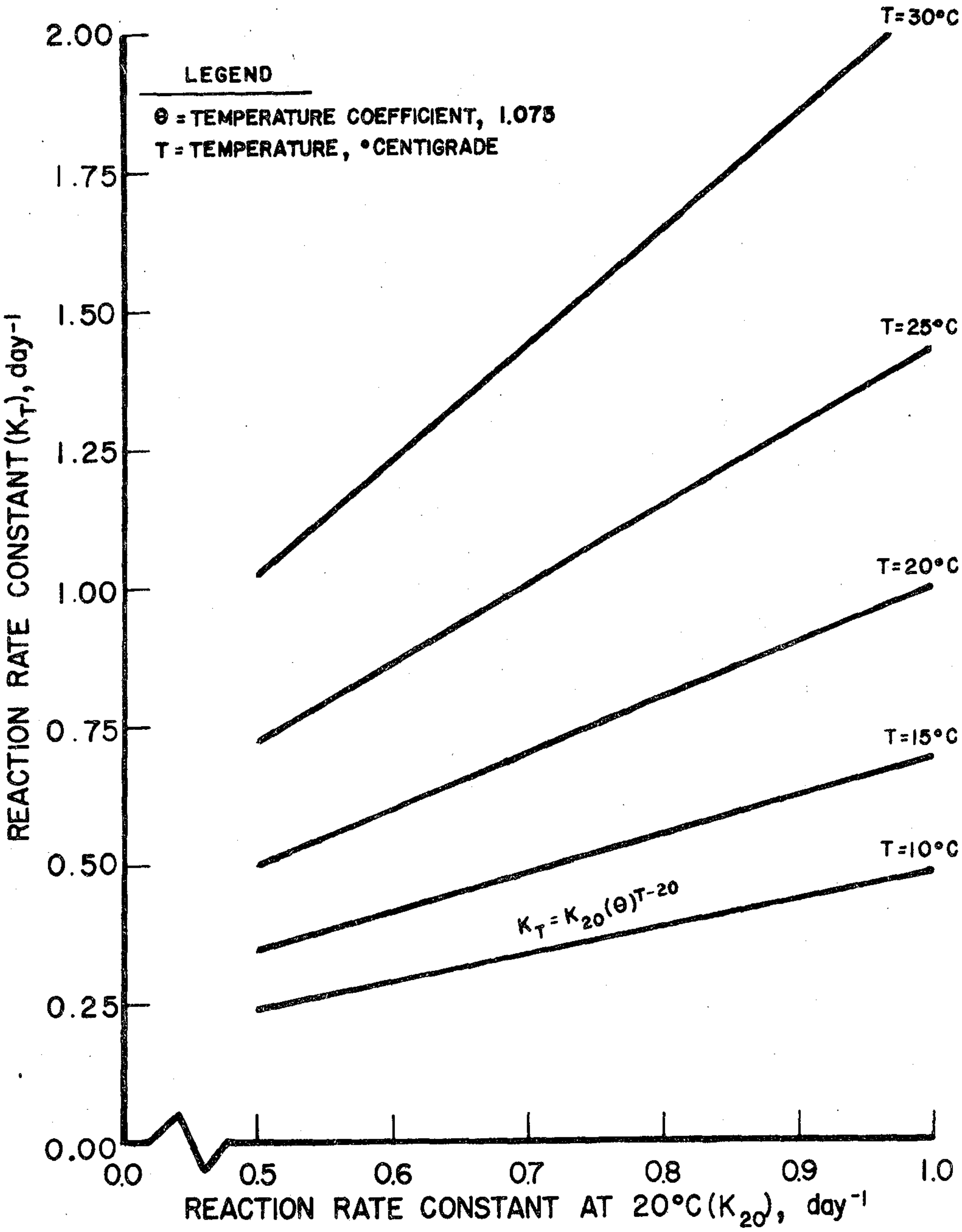

Figure 8-5. Reaction rate constant, Θ = 1.075.

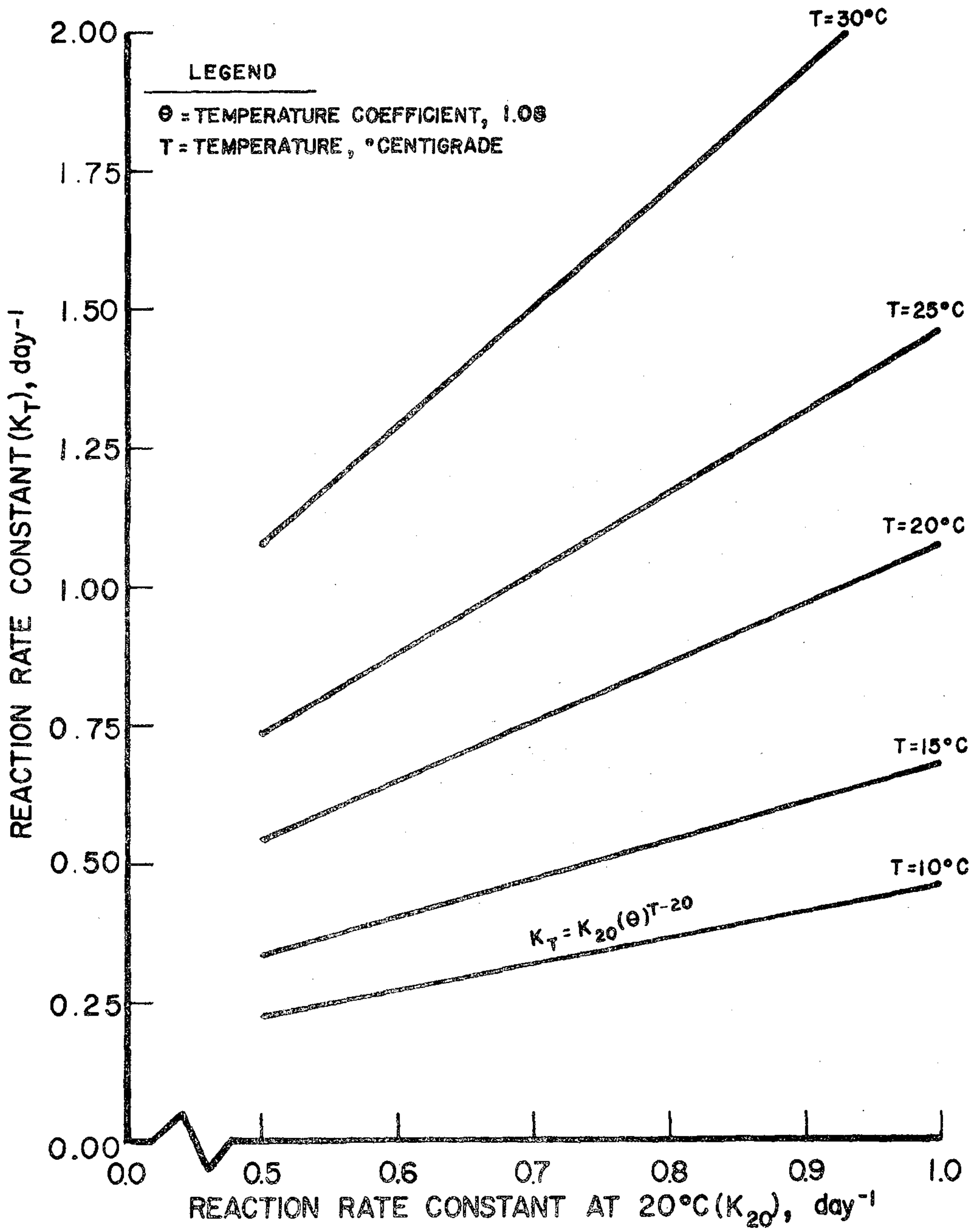

Figure 8-6. Reaction rate constant, Θ = 1.08.

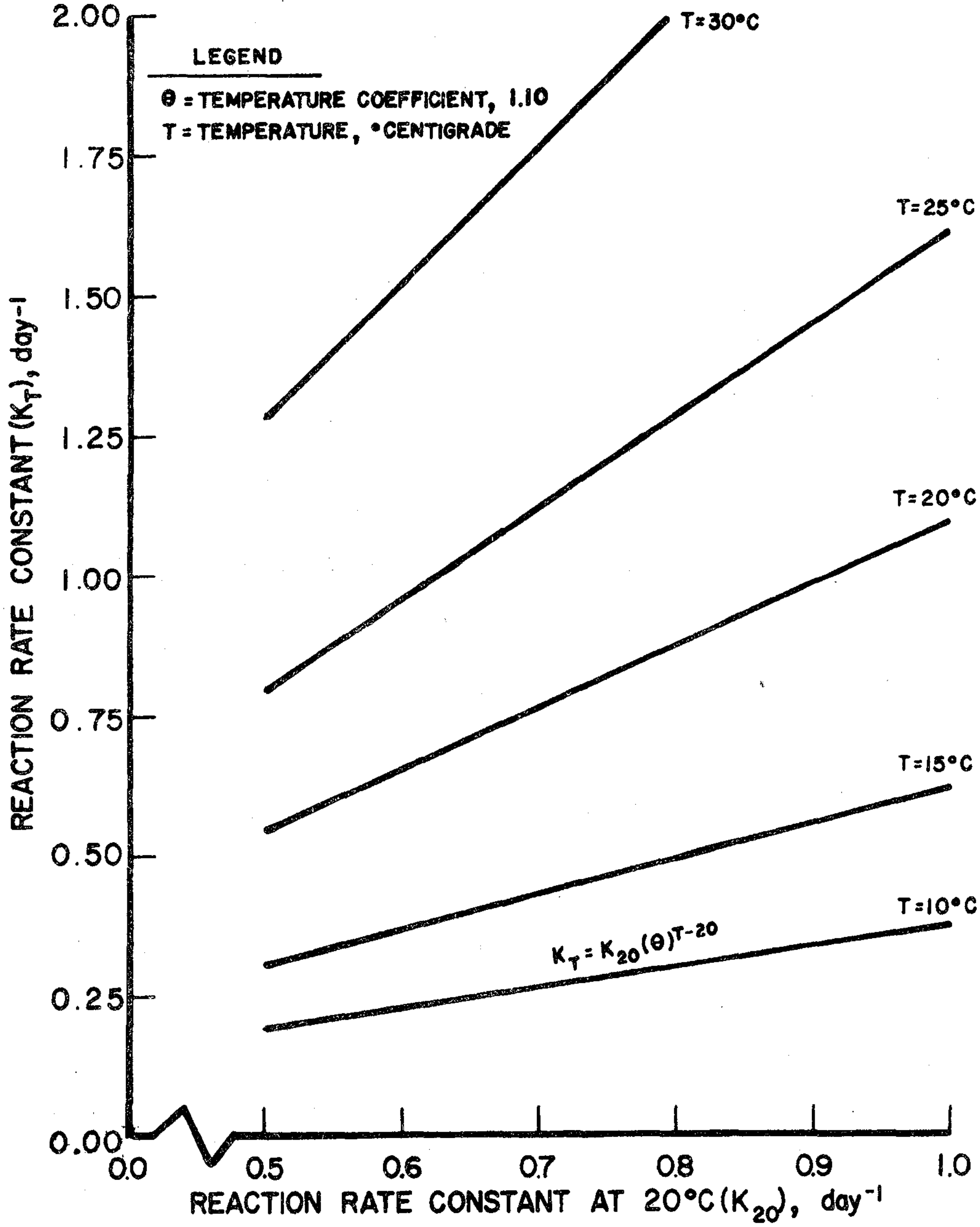

Figure 8-7. Reaction rate constant, Θ = 1.10.

operated in regions where total evaporation exceeds total precipitation. Possible infiltration to groundwater and subsequent contamination of the water supply aquifer must be prevented by making the lagoon bottom impermeable.

8-4. FLEXIBILITY

8-4-1. Faculative lagoons. Facultative lagoons are commonly constructed with one, two, three, or more cells arranged to operate in series or in parallel. As such, additional lagoons may be readily added in parallel (if there is sufficient land available) if the lagoon system presently in use is being hydraulically or organically overloaded. This overloading may take place when the original lagoon system has been underdesigned or when the rest area receives an increased usage due to completion of the highway system, removal of other nearby rest areas from service, etc. Facultative lagoons and facultative-aerated lagoons not only may be expanded, but because they have a point-source discharge they readily lend themselves to the inclusion of additional treatment processes such as intermittent sand filters, seepage beds, and chemical coagulation/sedimentation which is required if they are to meet the 1977 requirements of PL 92-500.

8-4-2. Facultative-aerated lagoons. Facultative-aerated lagoons may also be arranged in series and as such will allow for the addition of more lagoons to the system. However, since facultative-aerated lagoons may be followed by some process designed to separate the solids from the supernatant, additional lagoons may have to be located adjacent to the existing system and additional piping installed. Because facultative-aerated lagoons produce a point-source discharge, additional treatment processes may be added to upgrade the treatment system.

8-4-3. Totally evaporative lagoons. Totally evaporative lagoons may be expanded by the simple inclusion of additional lagoons. In this manner if a rest-area totally evaporative lagoon is receiving a wastewater flow in excess of its holding capacity, additional lagoons will provide increased holding capacity and increased surface area from which evaporation may take place.

It should be noted that the expansion and/or upgrading of all three lagoon systems is dependent upon the acquisition of additional land. Therefore, when a rest area is being designed additional land will have to be purchased for the future expansion of the lagoon system. If additional land is not available then the lagoon system will have to be replaced by a different type of treatment system that requires less land.

8-5. RELIABILITY

8-5-1. Facultative lagoons. Because of their simplicity, facultative lagoons are one of the most reliable treatment systems available for use at rest areas. With their long (usually 20 days or more) detention times they are able to withstand large fluctuations in hydraulic and organic loadings. However, it should be pointed out that the degree of treatment achieved in most facultative lagoons will not produce an effluent that is capable of meeting the requirements of PL 92-500.

8-5-2. Facultative-aerated lagoons. Facultative-aerated lagoons, because they rely on mechanical equipment to provide a supplementary oxygen supply, may be subject to periods of interruption due to mechanical failure. Facultative-aerated lagoons are able to withstand the large fluctuations in organic and hydraulic loadings that are experienced at rest areas.

8-5-3. Totally evaporative lagoons. Totally evaporative lagoons, because they have no discharge, provide a consistent and reliable form of disposal of rest-area-generated wastewater. If there is dike failure or if the lagoon has been improperly sized, totally evaporative lagoons will not give reliable treatment. Leakage may occur if the lagoon bottom has not been properly sealed.

8-6. OPERATION AND MAINTENANCE

Rest-area-generated wastewater either flows by gravity or is pumped into the lagoon, remains for a specified period of time, and is discharged. Operational procedures entail mowing the grass on the lagoon dikes, maintenance of at least a 3-ft liquid depth in the lagoons to prevent emergent plant growth, and breakup of any large floating algae mats that may form. Additionally, management of the discharge to periods of low algae production may be neccessitated.

8-6-1. Facultative lagoons. Because of the simplicity of operation of facultative lagoons it is generally required that the rest-area wastewater-treatment plant operators hold only the lowest class operator's license for any particular state. Also, because the operation of facultative lagoons requires a relatively short period of time each week (a few hours at most) the wastewater-treatment plant operator will be available to perform normal custodial duties at the rest area. Conversely, the fact that facultative lagoons produce an effluent that will not meet the requirements of PL 92-500 means additional treatment will be necessary. This treatment may require more skilled treatment plant operators.

8-6-2. Facultative-aerated lagoons. Facultative-aerated lagoons are normally constructed with one of two lagoons operated in series or parallel and may be followed by a clarifier for removal of solids. Operational requirements include control of emergent plant growth, dispersion of any floating algae mats that may form, maintenance of a sufficient oxygen supply by adjustment of the aeration equipment, mowing of lagoon dikes, inspection of the clarifier for proper settling of sludge, possible adjustment of the effluent weir of the clarifier, and possible collection and analysis of effluent samples.

Because mechanical equipment is used to provide additional oxygen to the bacteria, the degree of skill required of the treatment plant operator is more than that for the operator of facultative lagoons. The operator may have to hold a higher class operator's license, and he must be able to adjust the mechanical equipment to insure proper operating conditions. The operator must also be able to determine by visual assessment if the plant is operating correctly. He should check to see if the final effluent is clear or if solids are being carried over the effluent weir. He must also be able to tell by smell and sight if there is a sufficient air supply to the lagoon to maintain aerobic conditions. He may also take dissolved oxygen (DO) measurements in the lagoon. Time required for operation of a facultative-aerated lagoon system at a rest area will require about 2 hr per workday and may require longer if it is found that the system is not operating properly.

8-6-3. Totally evaporative lagoons. Totally evaporative lagoons may be operated singularly or in series and/or parallel. Operation consists of breaking up any algae mats that may form and mowing of the lagoon dikes. The degree of skill required of the treatment plant operator corresponds to the lowest class license available in that particular state. Operation time will be 2 hr or less per workweek.

In all three lagoon systems the treatment plant operator must also conduct weekly dike inspections to guard against seepage through the dikes and erosion of the inside slope of the dikes. If dike seepage or erosion is noted the operator should immediately contact the State highway department to take remedial action. The operator must also, on a periodic basis, inspect the influent and effluent control structures for clogging and rusting. Any clogging noted should immediately be rectified and any rusting reported to the State highway department.

8-7. PRELIMINARY DESIGN

8-7-1. Facultative lagoons. Facultative lagoons may be designed using the following procedure.

a. Input required:

Q = average daily flow, gpd

C_i = average BOD_5, mg/ℓ

Flow and BOD_5 can be approximated through the procedure outlined in Section 4 of this report.

b. Design criteria.

Surface loading rate, lb BOD_5/acre/day (15 to 50)

Depth, ft (3 to 6)

c. Design procedure:

(1) Compute BOD_5, lb/day, present in the waste.

$$BOD_5 = Q \times C_i \times 8.34 \times 10^{-6} \qquad (8\text{-}3)$$

where 8.34×10^{-6} is a conversion factor.

(2) Select a design loading, lb BOD_5/acre/day.

(3) Compute design surface area (SA), acres.

$$SA = \frac{\text{lb } BOD_5\text{/day in waste}}{\text{lb } BOD_5\text{/acre/day loading}} \qquad (8\text{-}4)$$

For the purpose of conservative design, surface area is that area of the lagoon at maximum depth. In this manner the lagoon is easily designed using design surface area and maximum operating depth.

(4) Compute the volume, selecting a depth of 3 to 6 ft (Figure 8-8). A minimum depth of 3 ft above the

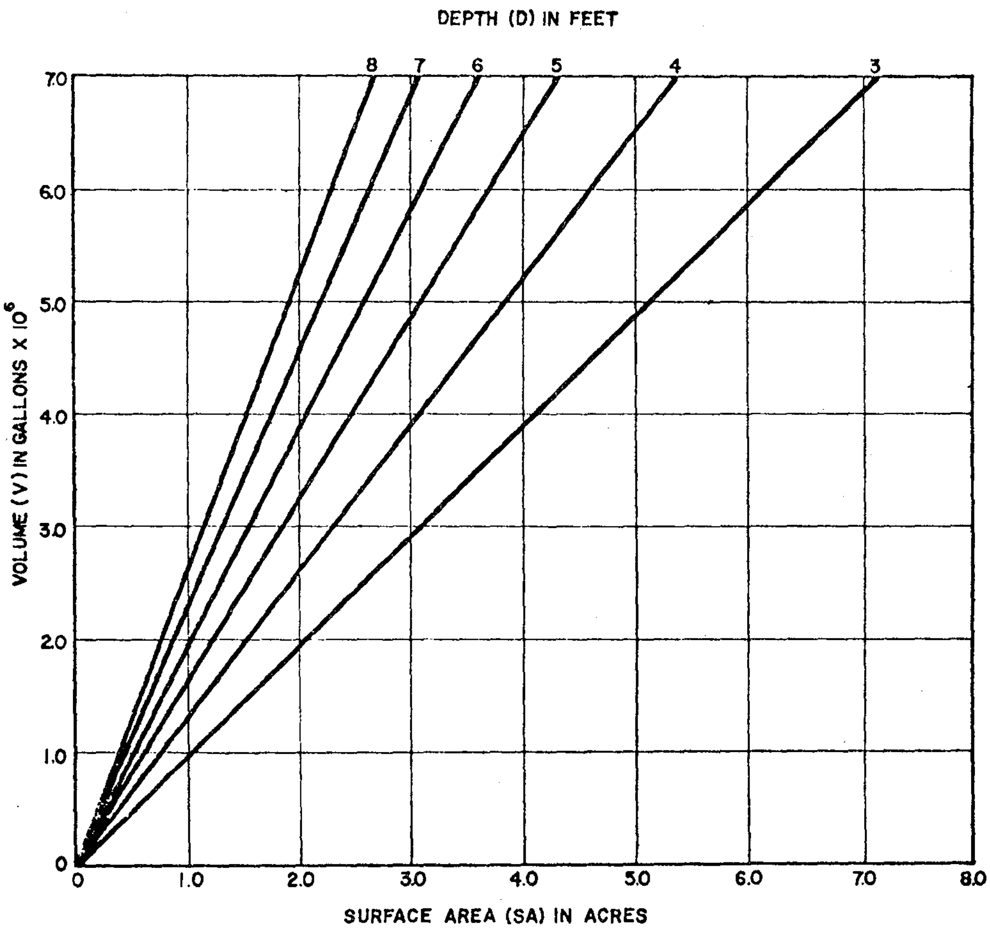

Figure 8-8. Determination of volume of facultative lagoon.

settled sludge is necessary to assure that there will be no emergent plant growth or odor problems.

$$V = SA \times D \times 325{,}830 \qquad (8\text{-}5)$$

where

V = volume, gal

SA = surface area, acres

D = depth, ft

(325,830 converts acre-feet to gallons)

(5) Compute detention time.

$$t = \frac{V}{Q} \qquad (8\text{-}6)$$

where

t = detention time, days

Note: Some states may have a required detention time, particularly in northern latitudes where winter ice cover inhibits microbial action. In this case, use the specified detention time and recompute the lagoon volume by increasing lagoon depth and/or surface area.

d. Output obtained:

SA = surface area, acres

V = volume, gal

t = detention time, days

e. Example calculations:

Q = 6000 gpd

C_i = 165 mg/ℓ

Loading = 35 lb/BOD_5/acre/day

D = 6 ft

(1) BOD_5 in waste = $6000 \times 165 \times 8.34 \times 10^{-6}$

BOD_5 in waste = 8.26 lb/day

(2) SA = 8.26/35 = 0.236 acre

(3) $V = 0.236 \times 6 \times 325{,}830$

V = 461,375 gal

(4) t = 461,375/6,000

t = 77 days

8-7-2. Facultative-aerated lagoons. Facultative-aerated lagoons for use at rest areas may be designed using the following procedure.

a. Input required:

Q = average daily flow, gpd

S_o = influent BOD_5, mg/ℓ

T = temperature, °C (winter and summer)

D = depth of lagoon

b. Design criteria:

K = reaction rate constant, day^{-1} (0.5 to 1.0, design avg 0.75)

θ = temperature correction coefficient (1.075)

a = fraction of BOD removed for energy (0.9 to 1.4, design avg 1.15)

MLVSS = mixed liquor volatile suspended solids, mg/ℓ (50 to 150)

BOD feedback from bottom sediment (summer = 20 percent; winter = 5 percent)

c. Design procedure:

(1) Check nutrient requirements

BOD:nitrogen:phosphorus = 100:5:1 minimum. This check will determine if there is sufficient nitrogen and phosphorus in the wastewater to promote microbial growth. If the wastewater is deficient in nitrogen or phophorus then the facultative-aerated lagoon will not operate at design removal for BOD. To protect against this, nitrogen and/or phosphorus must be added if they are not available in sufficient quantities in the wastewater.

(2) Compute the reaction rate constant (K) for mean winter temperature conditions.

$$K_T = K_{20}\theta^{T-20} \qquad \text{(8-2 bis)}$$

(3) Compute the detention time under mean winter temperature conditions (f = 1.05).

$$\frac{S_e}{S_o} = \frac{1}{1 + Kt}(1.05) \qquad \text{(8-1 bis)}$$

where

S_e = effluent soluble BOD_5, mg/ℓ (assumed by designer)

(4) Compute the detention time under summer conditions (f = 1.2).

$$\frac{S_e}{S_o} = \frac{1}{1 + Kt}(1.2) \qquad \text{(8-1 bis)}$$

(5) Select the larger detention time.

(6) Compute the volume of the lagoon.

$$V = Qt \qquad \text{(8-7)}$$

(7) Determine the oxygen requirements using desired degree of treatment, 85 percent removal of BOD.

$$S_r = 0.85 \times S_o \qquad \text{(8-8)}$$

$$O_2 = 1.4S_rQ(8.34)(1.2)(10^{-6}) \qquad \text{(8-9)}$$

where

S_r = BOD removal, mg/ℓ

O_2 = oxygen required, lb/day

(8) Calculate BOD in effluent.

$$BOD_{eff} = S_e + \left[0.3(VSS)_{eff}\right] \qquad \text{(8-10)}$$

where

BOD_{eff} = BOD in effluent, mg/ℓ

VSS_{eff} = effluent volatile suspended solids, mg/ℓ

d. Output obtained:

t = detention time, days

v = volume, gal

O_2 = oxygen required, lb/day

BOD_{eff} = BOD in effluent, mg/ℓ

e. Example calculations:

Given:

$Q = 6000$ gpd

$S_o = 165$ mg/ℓ

$N = 20$ mg/ℓ

$P = 15$ mg/ℓ

$T = 10°C$ (winter), $20°C$ (summer)

Assume:

$K_{20} = 0.75$

$S_e = 20$ mg/ℓ

85 percent BOD removal

VSS = 15 mg/ℓ

(1) Check BOD:N:P:
165:20:15
100:12:9

(2) Assume $K_{20} = 0.75$.

$$K_{10} = K_{20}\theta^{10-20}$$

$$K_{10} = 0.75(1.075)^{-10}$$

$$K_{10} = 0.364$$

(3) Assume $S_e = 20$ mg/ℓ.

For winter:

$$\frac{S_e}{S_o} = \frac{1}{1 + Kt}\ (1.05)$$

$$\frac{20}{165} = \frac{1}{1 + (0.364)t}\ (1.05)$$

$t = 21.1$ days

(4) For summer:

$$\frac{S_e}{S_o} = \frac{1}{1 + Kt}\ (1.2)$$

$$\frac{20}{165} = \frac{1}{1 + (0.75)t}\ (1.2)$$

$t = 11.9$ days

(5) Select $t = 21.1$ days for design detention time.

(6) $V = Qt$

$V = 6{,}000\ (21.1)$

$V = 126{,}600$ gal

(7) Assume 85 percent BOD removal.

$S_r = 0.85 \times S_o$

$S_r = 140.25$

$O_2 = 1.4 S_r Q(8.34)(1.2)(10^{-6})$

Assume $a' = 1.4$

$O_2 = 1.4(140.25)(6000)(8.34)(1.2)(10^{-6})$

$O_2 = 11.79$ lb O_2/day

(8) Assume VSS = 15 mg/ℓ.

$BOD_{eff} = S_e + \left[0.3(VSS)_{eff}\right]$

$BOD_{eff} = 20 + [0.3(15)]$

$BOD_{eff} = 24.5$ mg/ℓ

8-7-3. Totally evaporative lagoons. Totally evaporative lagoons may be designed using the following procedure.

a. Input required:

Q = average daily flow, gpd

b. Design criteria:

E = total yearly evaporation, in.

P = total yearly precipitation, in.

D = operating depth, ft (3 to 8)

c. Design procedure:

(1) Compute total annual excess (ΔV) of evaporation over precipitation (see Figure 8-9).

$$\Delta V = E - P$$

(2) Convert excess (ΔV) to gal/acre/yr (G) (Figure 8-9).

$$G = \Delta V \times \frac{1\ \text{ft}}{12\ \text{in.}} \times \frac{43{,}560\ \text{sq ft}}{\text{acre}} \times \frac{7.48\ \text{gal}}{\text{cu ft}}$$

$$G = \Delta V \times 27{,}152.4$$

(3) Compute total yearly wastewater production (Y) in gal/yr, or obtain directly from yearly rest area traffic.

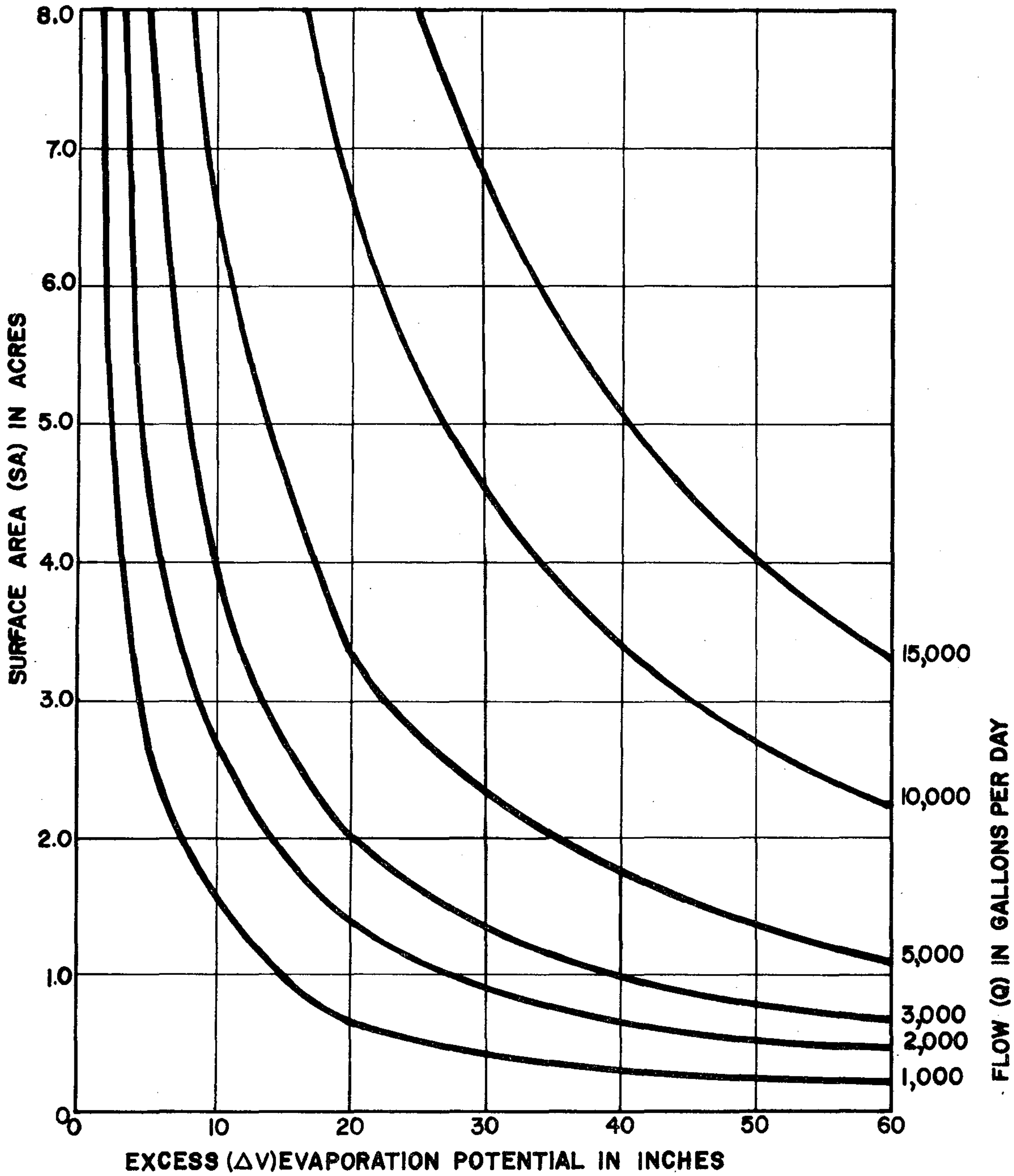

Figure 8-9. Conversion of excess evaporation potential to flow, gpd.

$$Y = Q \times 365 \frac{\text{days}}{\text{year}}$$

(4) Compute SA, acres, of the flat area of the lagoon.

$$SA = \frac{Y}{G}$$

(5) Select operating depth, ft (D).

(6) Compute volume (V), gal.

$$V = SA \times D \times \frac{43{,}560 \text{ sq ft}}{\text{acre}} \times \frac{7.48 \text{ gal}}{\text{cu ft}}$$

d. Output obtained:

SA = surface area, acres

V = volume, gal

e. Example calculations:

Q = 6000 gpd

E = 98 in./yr

P = 50 in./yr

(1) Compute excess (ΔV).

$\Delta V = E - P$

$\Delta V = 98 - 50$

$\Delta V = 48$ in.

(2) Convert ΔV to gal/acre/yr (G).

$G = \Delta V \times \frac{1}{12} \times 43{,}560 \times 7.48$

G = 1,303,315 gal/acre/yr

(3) Compute Y.

$Y = Q \times 365$

$Y = 6000 \times 365$

Y = 2,190,000 gal/yr

(4) Compute SA of the flat area or the lagoon.

SA = Y/G

SA = 2,190,000/1,303,315

SA = 1.68 acres

(5) Select D of 5 ft.

(6) Compute volume V.

$V = SA \times D \times 43{,}560 \times 7.48$

V = 2,737,000 gal

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9. EXTENDED AERATION PACKAGE PLANTS

9-1. PROCESS DESCRIPTION

Extended aeration is one of several modifications of the activated sludge process for treatment of wastewater. In an activated sludge process, raw wastewater enters an aeration tank and is mixed with a heterogeneous culture of aerobic microorganisms in the presence of oxygen. Aeration holds the microorganisms in suspension and provides the oxygen required by the microorganisms for their metabolic processes which remove contaminants (organics) from the wastewater. The contents of the aeration tank, termed mixed liquor, flow into a settling tank, or Clarifier, for removal of suspended solids (biological floc) prior to discharge. Most of the settled sludge is returned to the aeration tank in order to maintain a suitable concentration of active microbial mass in the mixed liquid, but a portion of the sludge is wasted from the system.

Activated sludge systems are designed primarily to remove biochemical oxygen demanding (BOD) organic matter. Organic carbon and other nutrients are used by the microorganisms, primarily bacteria, for energy and for synthesis of new cells. The energy reaction may be represented by the following expression:

$$\text{Food} + O_2 \xrightarrow{\text{microorganisms}} CO_2 + H_2O + \text{energy} \qquad (9\text{-}1)$$

Similarly, the synthesis reaction is:

$$\text{Food} + O_2 + NH_3 \xrightarrow{\text{microorganisms}} CO_2 + H_2O + \text{new cells} \qquad (9\text{-}2)$$

The organic fraction of the microorganism cellular mass, which is 90 percent of the total, may be represented by the formula $C_5H_7NO_2$. The remaining inorganic fraction is primarily P_2O_5. Therefore, it is evident that the synthesis reaction requires sufficient oxygen, carbon, nitrogen, and phosphorus for building celluar mass.

The growth of microorganisms is directly related to the amount of food available to the cell. Three phases are defined for bacterial growth: logrithemetric growth, declining growth, and endogenous growth. Figure 9-1 shows the phases of growth and the relationship between mass of microorganisms and food available. In the log growth phase there is usually always an excess of food for the microorganisms, and the growth rate is at a maximum, usually limited only by the ability of the microorganisms to process the food present. As the bacteria grow and consume the available food, their rate of growth decreases and this is termed declining growth. When the food concentration reaches a level insufficient to sustain growth, the walls of some cells deteriorate, and their protoplasm is released to solution to be used for food by the remaining cells. This process is called endogenous respiration or auto-oxidation and may be represented by the simplified reaction:

$$C_5H_7NO_2 + 5O_2 \rightarrow NH_3 + 5O_2 + 2H_2O + \text{energy} \qquad (9\text{-}3)$$

It is at the point where endogenous respiration is taking place that the activated sludge process is called extended aeration.

In addition to removal of organic carbon, another reaction that occurs to some extent in activated sludge processes is nitrification of the ammonia nitrogen from urine in the wastewater. For conservative design, it is assumed that all of the organic nitrogen is hydrolyzed to the ammonia form in the aeration tank. The sum of the organic nitrogen and ammonia nitrogen present is measured as total Kjeldahl nitrogen (TKN). Ammonium ion (NH_4^+) is oxidized to nitrites by <u>Nitrosomonas</u> bacteria as follows:

$$NH_4^+ + \frac{3}{2}O_2 \xrightarrow{\text{Nitrosomonas}} NO_2^- + H_2O \qquad (9\text{-}4)$$

The nitrite (NO_2^-) is further oxidized to nitrate (NO_3^-) by <u>Nitrobacter</u> bacteria to complete the nitrification process.

$$NO_2^- + \frac{1}{2}O_2 \xrightarrow{\text{Nitrobacter}} NO_3^- \qquad (9\text{-}5)$$

<u>Nitrosomonas</u> and <u>Nitrobacter</u> differ from the heterotrophic

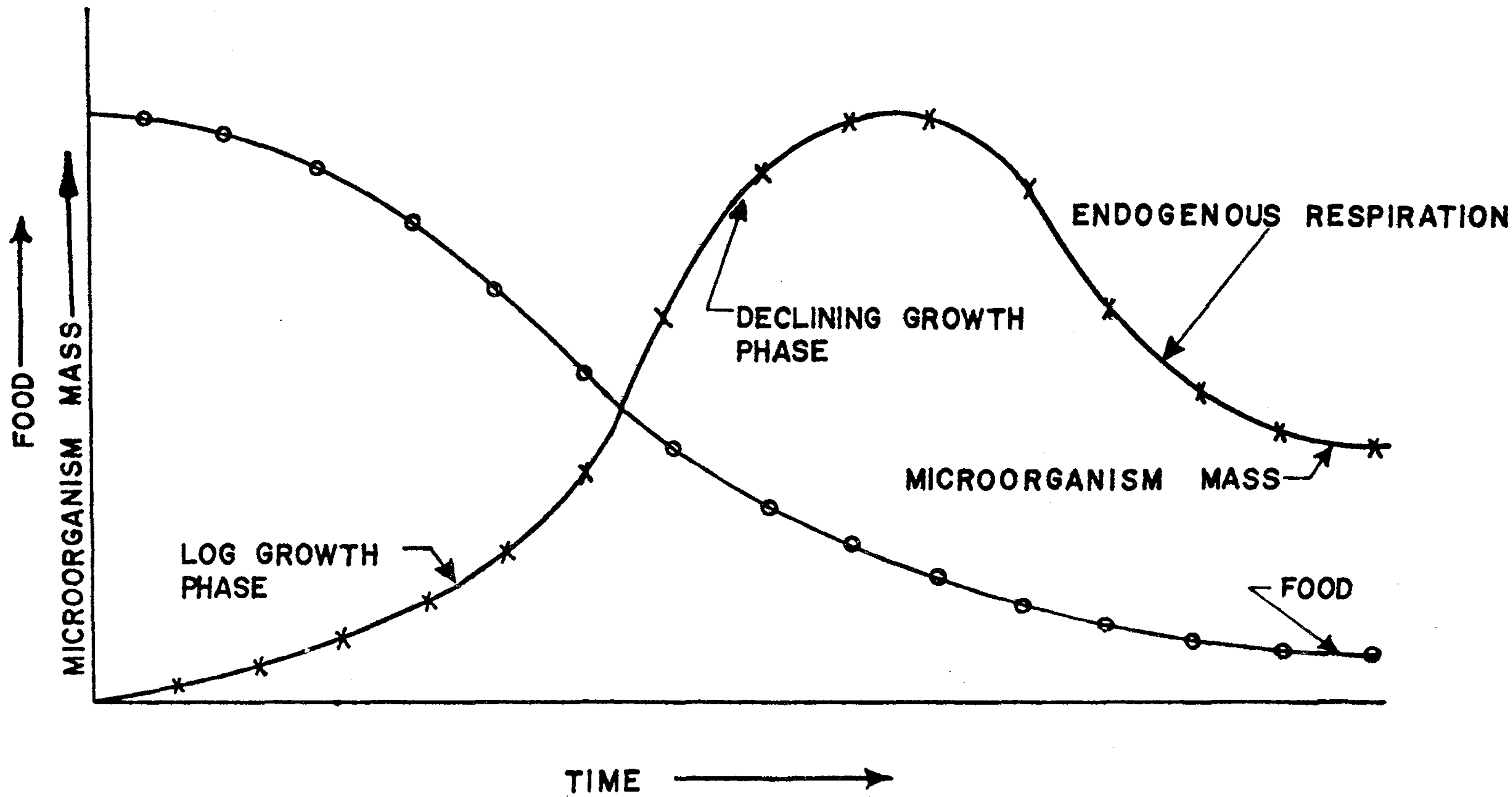

Figure 9-1. Phases of growth and relationship between microorganism mass and available food.

bacteria responsible for removal of organic carbon in that they utilize inorganic carbon (carbon dioxide, carbonates, etc.) as their source of carbon for synthesis of cells. They are known as autotrophs. Autotrophs cannot produce sufficient energy to compete with active heterotrophs. Therefore, nitrification in activated sludge systems occurs only when the level of organic carbon has been reduced and the autotrophic bacteria become active. The growth rate of the nitrifying bacteria is very slow; therefore, the microbial mass, i.e., the mixed liquor, must be retained in the system for a period sufficiently long for their reproduction. The length of time sludge is retained in an activated sludge system is known as sludge age or sludge retention time. In activated sludge systems nitrification may require a sludge age of 6-10 days. However, the nitrifiers are very sensitive to changes in their environment, sometimes requiring an even longer design sludge age to insure nitrification. For optimum nitrification, the dissolved oxygen should be greater than 2 mg/ℓ and the pH in the range between 7.6-8.4; decreasing temperature decreases the reaction; and toxic materials such as heavy metals are detrimental even at low concentrations. The effects of temperature and pH on the nitrification reaction are shown graphically in Figures 9-2 and 9-3, respectively. The degree of nitrification required by the regulatory agency will determine whether these environmental conditions should be maximized or not. With long detention times and long sludge ages, nitrification can occur in single-stage systems such as extended aeration. Another method of nitrification is to follow an activated sludge unit that is strictly designed for organic carbon removal with a second aeration tank and clarifier for the nitrification reactions (two-stage nitrification).

An important point to consider in designing extended aeration systems in which nitrification may occur is the additional oxygen required by the conversion of ammonia to nitrite and nitrate. Stoichiometric ratios derived from the chemical reactions given in Equations 9-4 and 9-5 show that 4.57 mass units of oxygen as O_2 are required for each unit of ammonia as nitrogen removed. This oxygen, which is in addition to that required for removal of carbonaceous BOD, will increase the size

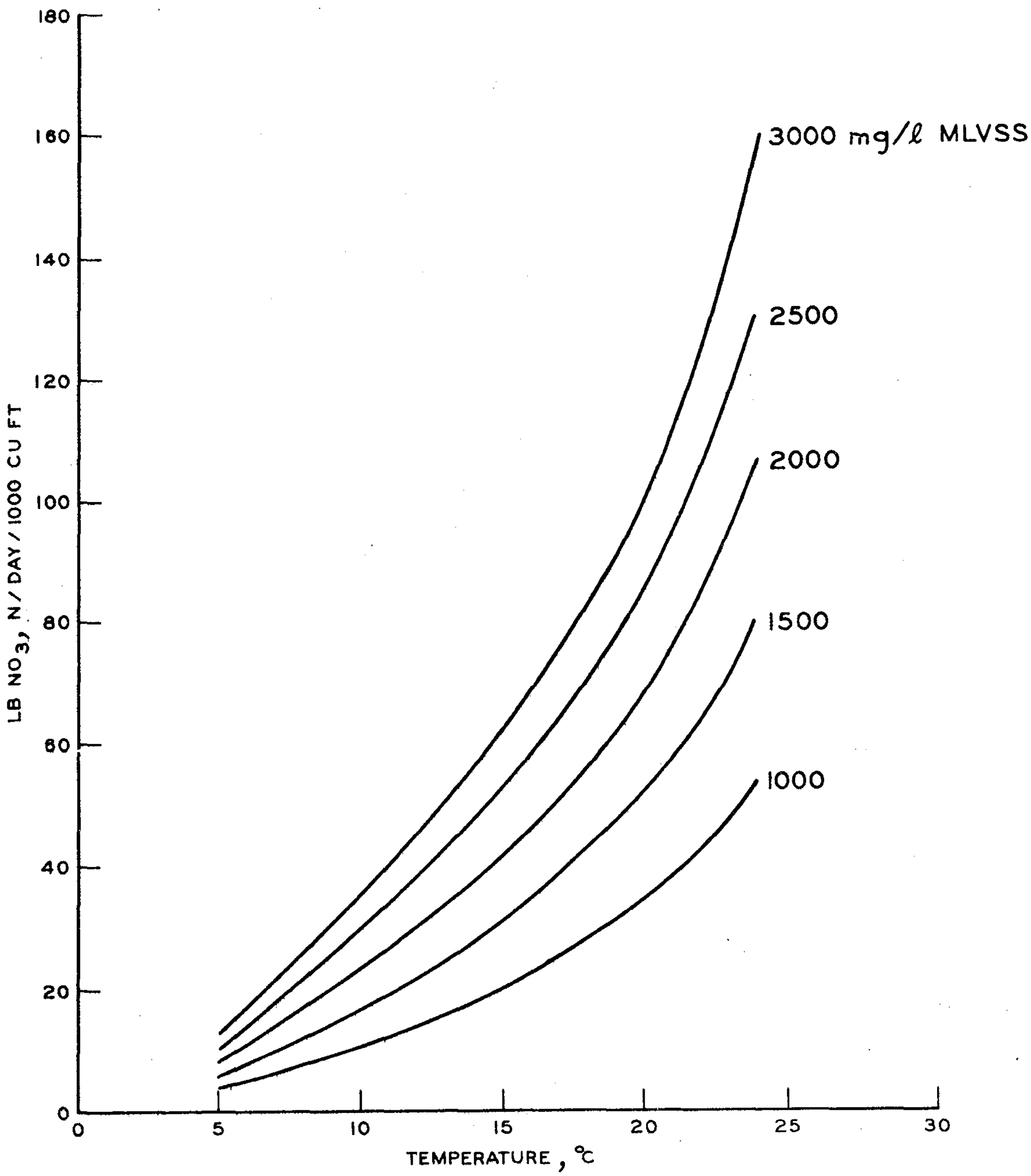

Figure 9-2. Effect of temperature on nitrification.

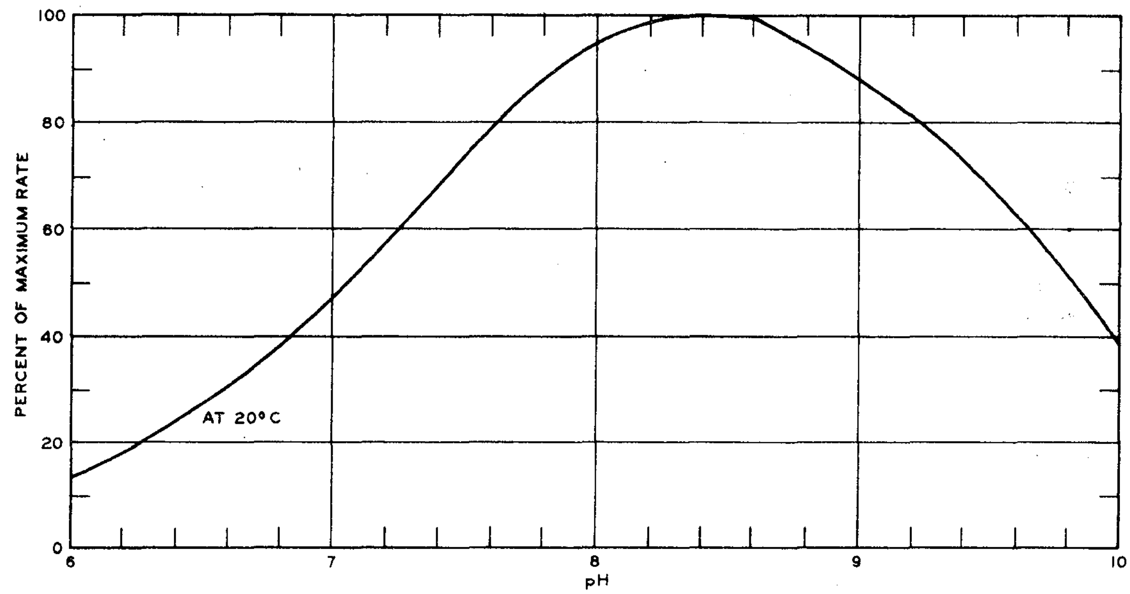

Figure 9-3. Effect of pH on nitrification.

of the aeration system for the extended aeration plant.

If a percentage removal of total nitrogen is required by the regulatory agency, then the nitrates may be biologically converted to nitrogen gas by optimizing the following denitrification reaction:

$$NO_3^- \rightarrow NO_2^- \rightarrow N_2 \text{ or } N_2O \tag{9-6}$$

The bacteria responsible for denitrification are facultative heterotrophs and require an organic carbon source for energy. A system designed for denitrification normally provides a third tank with no aeration and addition of supplemental carbon, although some carbon may be obtained through endogenous respiration. Denitrification often occurs in final clarifiers in which the mixed liquor is highly nitrified. Nitrogen gas is produced in the sludge blanket, causing the sludge to float to the surface and produce a solids loss over the clarifier weir. This is a significant operational problem for many extended aeration activated sludge systems.

Activated sludge process modifications may operate in any one or more of the growth phases, depending on the mass of food present with respect to the mass of microorganisms present. The ratio between these masses is termed the food-to-microorganism (F/M) ratio and is an important parameter in the design of activated sludge treatment systems because of its effect on the kinetics of removal and sludge settleability. The extended aeration system, the primary modification of activated sludge in use at rest areas, is characterized by a low F/M ratio and, therefore, operates in the endogenous growth phase. In order to maintain a desired F/M ratio, the mass of microorganisms, commonly measured as mixed liquor volatile suspended solids (MLVSS), must be controlled. This is done by returning the necessary portion of sludge from the clarifier to the aeration tank and wasting the excess. Because of the significant cost of sludge disposal, the amount wasted should be carefully evaluated in design procedures. As a practice sludge is wasted only once a year from the system.

Microbial waste reduction reactions in the aeration tank are affected by the mixing of the microorganism/food mass. Activated sludge systems may be completely mixed flow or plug flow. In completely mixed systems, influent wastewater (food) and recycled sludge (microorganisms) are discharged separately into the aeration tank where they lose their individual identity and are completely mixed with the contents of the tank. F/M ratio and oxygen uptake rate are considered constant throughout the aeration tank. Plug flow systems are characterized by long narrow aeration tanks and mixing of return sludge with influent wastewater at the front of the tank. The F/M at the influent end of the tank is high, the microorganisms are in the log growth phase, and oxygen consumption is great. As the wastewater flows toward the tank effluent, oxygen requirements decrease, and food concentration gradually decreases until the bacteria undergo endogenous respiration. The plug flow system may be used to optimize nitrification.

Extended aeration is the activated sludge modification usually used at rest areas. The number of extended aeration plants at rest areas ranks second only to septic tank-leach fields. Extended aeration systems normally do not include a primary clarifier as do conventional activated sludge systems. After comminution and possibly grit removal, wastewater enters the aeration tank. Aeration and mixing are usually provided by diffused aeration systems. In small plants the mixing regime is usually complete mix rather than plug flow. The clarifier is normally separate from the aeration tank, although some plants may have a settling compartment separated from the aeration tank only by an underflow baffle. For separate clarifiers, sludge is returned to the aeration tank by airlift pumps; whereas, in baffled tanks sludge is returned by gravity. Scum and floating solids are removed from the surface of the clarifier with an airlift scum return pump. Most plants include an aerated sludge holding tank to minimize the frequency of sludge disposal. The size of many extended aeration plants installed at rest areas (<15,000 gal) enables fabrication and assembly at the factory. These so-called "package plants" are shipped to the rest area site by truck ready to be installed. The plants are available in a variety of shapes and configurations depending on the manufacturer. Installation may be above or below the ground, with the units below ground offering additional protection from low temperatures and usually being more aesthetic.

About 98 percent of the influent BOD is either oxidized or converted to cellular mass in the extended aeration tank. This results in a consistently low soluble effluent BOD_5. However, the biological solids in the effluent contribute more to the total effluent BOD concentration than does the soluble BOD. Hence, efficient solids removal in the clarifier is a prime consideration in design of an extended aeration system. Most package plant clarifiers have a 4-hr detention time based on influent flow and an overflow rate less than 300 gal/ft^2/day. Operational procedures used in the clarifier determine the actual performance of the clarifier.

Problems in sludge settling may be caused by sludge bulking, denitrification, and production of nonflocculent solids. Sludge bulking, caused by filamentous bacteria, usually results from an overloaded system and low oxygen levels and generally will not be a problem in properly designed and operated extended aeration systems at rest areas. Denitrification, forming nitrogen gas bubbles and rising sludge, probably is the most significant problem with extended aeration plants at rest areas. This problem may be minimized by limiting the time that settled solids remain in the clarifier bottom. Dispersed flocs are produced at low organic loadings and lack the ability to flocculate in the clarifier. Studies by Pfeffer[1] showed that suspended solids removal effectiveness decreased with loadings less than 15 lb BOD_5 per 1000 cu ft of aeration. The clarifier cannot perform efficiently when subjected to significant increases in hydraulic loadings. Equalization of flow into the plant or controlled flow from the aeration tank to the clarifier may be necessary for optimum performance. More information regarding flow equalization is given in Section 6.

An advantage of the extended aeration system is the minimal sludge wasting required. For small systems, such as those at rest areas, there is an added advantage of providing sufficient aeration capacity to handle fluctuating loads. Sludge wasting is minimized by optimizing conditions for endogenous respiration. One such condition is a low F/M ratio (0.05-0.15). F/M for a given substrate is directly proportioned to the detention time and MLVSS. Most package extended aeration plants are designed for a detention time of 24 hr. MLVSS is an operational parameter and can be selected based on the design F/M. When extended aeration was first developed, it was thought that complete oxidation of the sludge produced was possible. Later investigations on municipal wastewater indicated that 23 percent of the biological solids produced cannot be further degraded by the microorganisms. Since the system could not ultimately handle a continuous buildup of solids, solids should be wasted or they will be discharged in the effluent. Obviously, excessive solids in the effluent are not acceptable because of suspended solids limitations and because of their contribution to the effluent BOD. Inert solids are generally more dense and settle better than active biological solids. Therefore, a significant portion of the solids discharged over the clarifier weir are biodegradable and exert a significant BOD in the effluent.

Sludge wasted from the system requires disposal. On-site disposal can be accomplished on sand drying beds for remote locations. Where the rest area is near a municipal system, the sludge may be trucked to the municipal wastewater-treatment plant. Wasting from a rest area plant may occur only a few times per year and should not be a significant problem.

9-2. PERFORMANCE

Extended aeration package plants at rest areas are capable of meeting PL 92-500 secondary treatment requirements for BOD_5 and total suspended solids (TSS) removal when the plants are properly designed, operated, and maintained. In addition, greater than 50 percent nitrification can be achieved in extended aeration plants where wastewater temperatures are greater than 15°C and the system is designed to promote nitrification. The effluent will require disinfection to meet fecal coliform limitations. Literature studies for extended aeration plants are usually based on performance of plants serving residential areas. Little comprehensive operating data were available for rest areas when this study was initiated.

Therefore, it was determined that there was a need to evaluate the operating efficiency of an extended aeration package treatment plant operating at a rest area. The plant monitored in this study was at a rest area on the west-bound lane of Interstate 20 east of Meridian, MS. The rest area is commonly referred to as the Lauderdale County Welcome Center and is equipped with attended information facilities, parking, picnic, drinking, trailer-parking, trailer-dumping, and wastewater-treatment facilities. The Welcome Center is illustrated in Figure 9-4. The main facility contains both men's and women's comfort facilities as well as a manned welcome center counter where free coffee and soft drinks are dispensed.

Water usage was monitored at the well located adjacent to the main welcome center building by installing a 1/2-inch Neptune Trident water meter in the main water line from the well. The water meter was equipped with an Easterline-Argus Strip Chart recorder that showed water usage in 50 gallon increments versus time. In this manner hourly and daily water usage were obtained.

Both interstate highway traffic and rest area traffic were

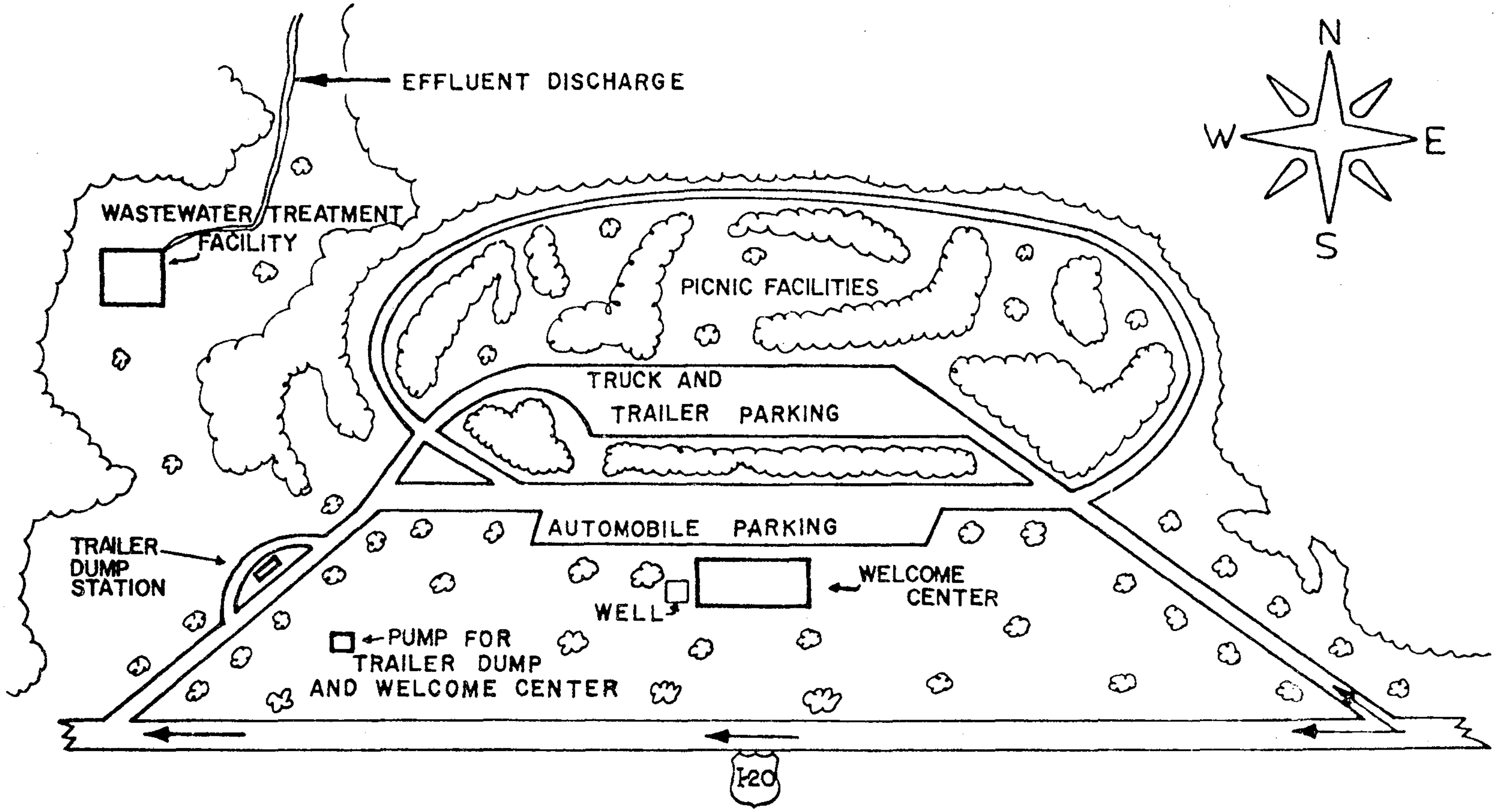

Figure 9-4. Lauderdale County Welcome Center.

monitored throughout the study by the Mississippi State Highway Department. In this manner it was possible to equate water usage with vehicles entering the rest area.

Wastewater is generated both at the main welcome center building and the trailer dumping station. Wastewater from both areas flowed by gravity to a lift station located between the welcome center egress road and Interstate 20. The wastewater lift station is equipped with a bar rack for trapping large objects that may have entered the wastewater lines and with 2 centrifugal pumps. Each pump is equipped with a macerator and is capable of pumping the wastewater at the rate of 80 gallons per minute. The pumps may be operated either manually or automatically. When operating automatically the pumps are activated by float switches so that they operate only when there is wastewater in the lift station. From the lift station the wastewater is pumped to the extended aeration activated sludge package plant which is located on the access road behind the recreational vehicle disposal facility (sewage dumping station).

In the treatment plant at the Lauderdale County Welcome Center the influent wastewater (which has already been macerated at the lift station) passes through a comminutor and then enters the first of three aeration chambers that are operated in series. Each of the three aeration chambers has a capacity of 5000 gallons and is equipped with air diffusers. The plant has a 2500-gal settling tank with sludge return, a 1125-gal chlorine contact chamber and a post aeration chamber equipped with diffused aerator. In this study wastewater sampling points were located at (1) the influent line to the comminutor, (2) the settling tank just prior to the weir, (3) the chlorine contact chamber prior to the weir, (4) inside each aeration chamber, and (5) in the sludge recycle line. A schematic of the treatment plant showing sampling points is shown in Figure 9-5.

Flow measurement was taken in the chlorine contact chamber by use of both a Manning Dipper Flowmeter and a Leopold Stevens type F recorder. Thus it was possible to contrast water usage with wastewater production. Composite samples proportional to flows were collected by a Manning S-4000 portable wastewater sampler on the effluent line just prior to

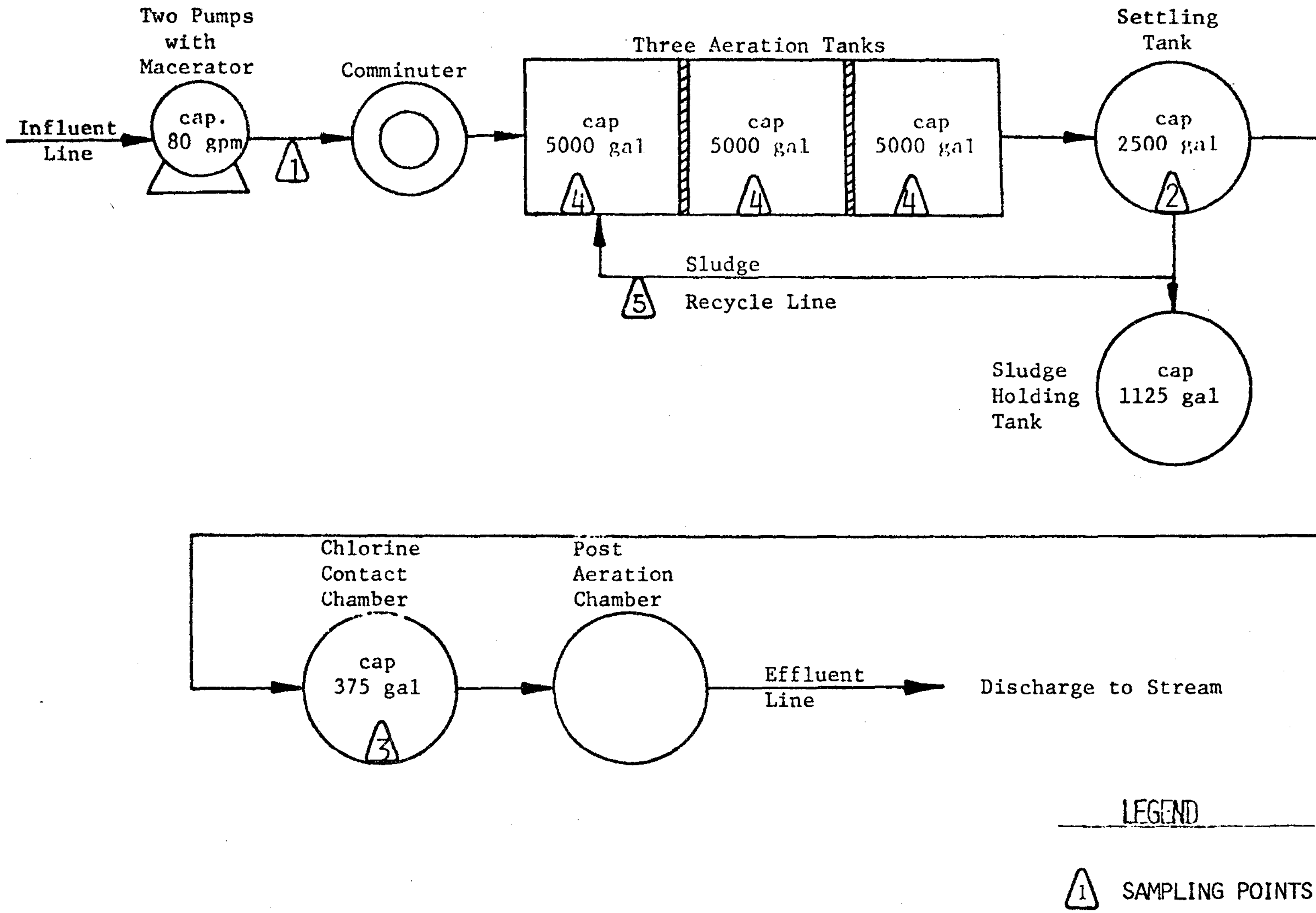

Figure 9-5. Schematic of treatment plant.

comminution. All other samples were grab samples taken manually including the grab samples taken at the influent line. Samples collected at the influent line and the chlorine contact chamber were analyzed for ammonia, nitrate, phosphate, pH, total solids, settleable solids, suspended solids, dissolved solids, and volatile suspended solids, 5-day biochemical oxygen demand, and fecal coliform count. All analyses were performed in accordance with procedures set forth in the 13th edition of Standard Methods (1971) with the exception of the dissolved oxygen uptake rates which were performed in accordance with the 14th edition of Standard Methods (1975).

A plot of water usage versus number of vehicles using the rest area (Figure 9-6) shows a direct correlation. The average water usage was 20.0 gal/veh as compared with the 6.7 gal/veh predicted by Zaltzman. Certain facts, however, must be pointed out. These are: (1) there is a high water usage at this particular rest area for irrigation, (2) there is a high water usage for cleaning purposes, and (3) there is a significant amount of water used daily for preparation of coffee. Wastewater production was also plotted versus number of vehicles entering the rest area (Figure 9-7). Average wastewater production was 7.0 gal/veh or slightly higher than the 5.0 gal/veh used by Zaltzman but well below the 15.5 gal/veh (5 gal/cap at 3.1 persons/veh) previously used by many state highway departments. It must be pointed out that even under peak flow the plant was receiving only 33-1/3% (5000 gals) of design flow and had a detention time of three days. Under average conditions the plant was receiving only 25% of design flow or less and had a detention time of four or more days.

A tabulation of the influent data collected is given in Table 9-1. Table 9-2 shows the effluent data collected. The graph of influent and effluent BOD_5 (Figure 9-8) shows the large variation in concentration that may occur in a rest area wastewater (note the presence of the trailer dump waste, 1965). It also shows the high efficiency of the plant investigated with an average influent BOD_5 of 124 mg/ℓ and an average effluent of less than 3 mg/ℓ BOD_5 or an overall BOD_5 removal efficiency of 98%. The requirements of PL 92-500 are for a

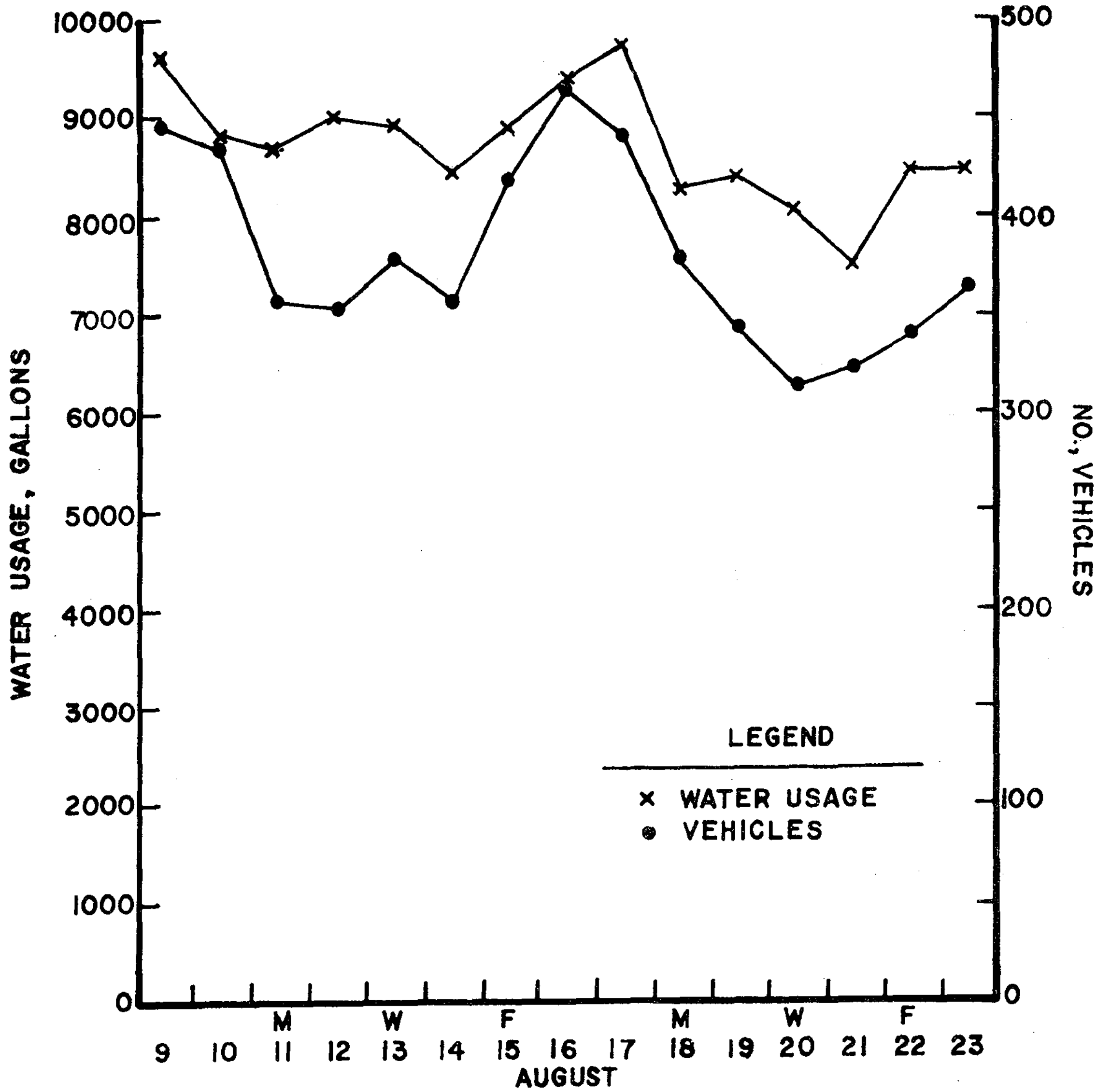

Figure 9-6. Water usage versus rest area traffic.

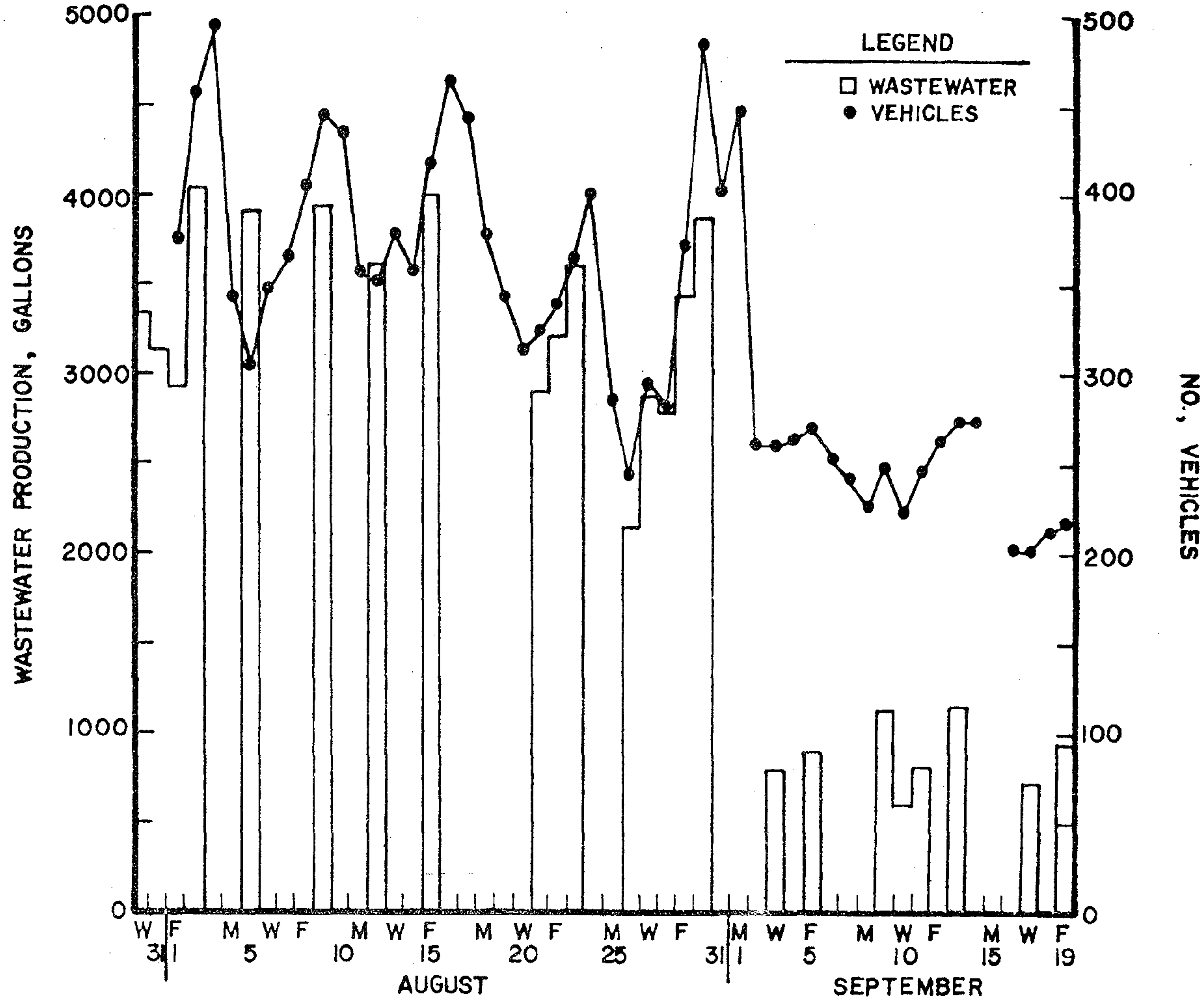

Figure 9-7. Wastewater production versus rest area traffic.

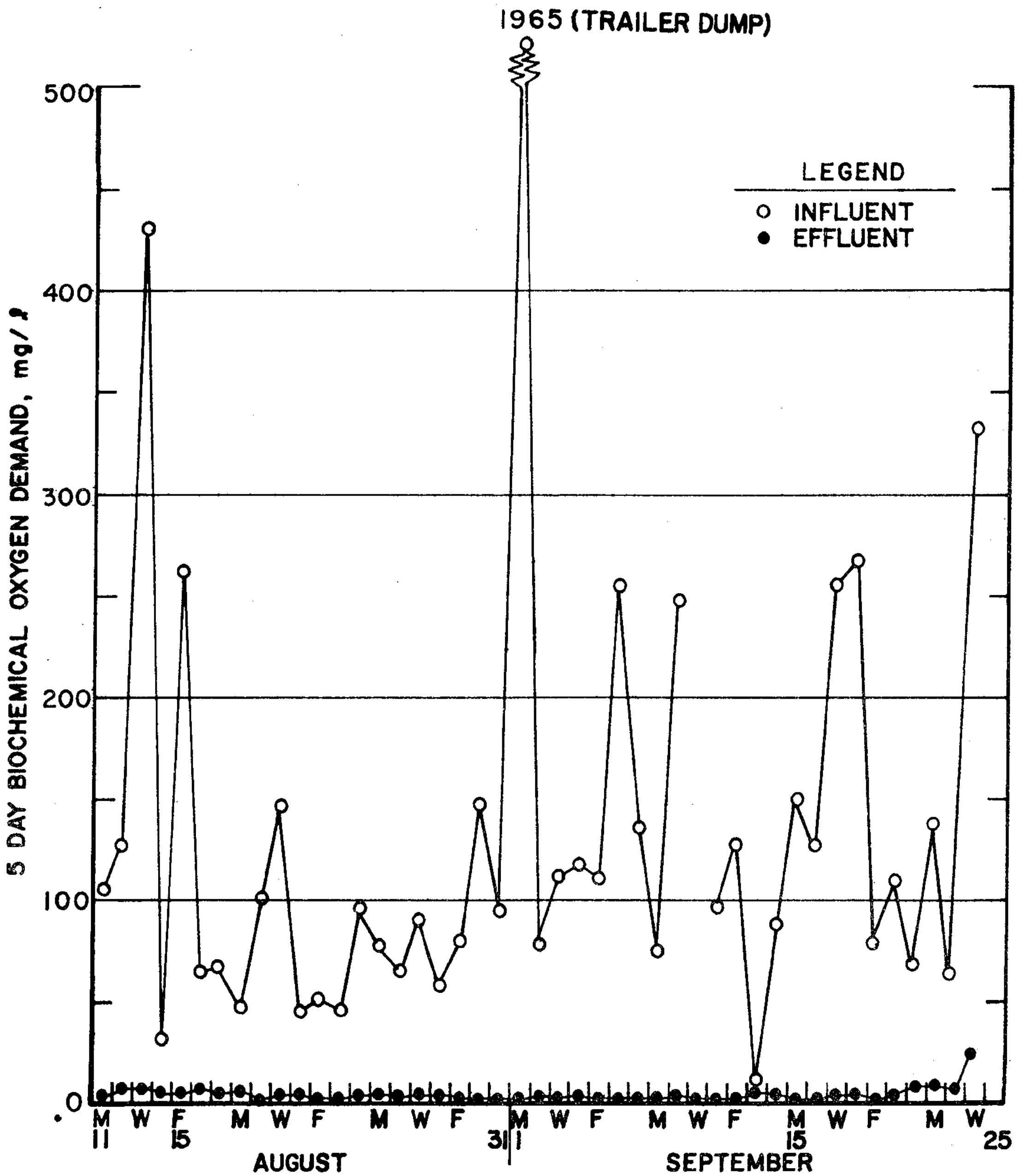

Figure 9-8. Influent and effluent BOD_5.

Table 9.1 Influent Data, Lauderdale County Welcome Center.

Date Influent	Time	BOD_5 mg/ℓ	F Coli × 10^6	TS mg/ℓ	Settleable Solids mg/ℓ	SS mg/ℓ	DS mg/ℓ	VSS mg/ℓ	pH	Ammonia	TKN	Total Phosphate	NO_3-N
8-11	1000	105	2.2	327	12.0	95	232	84	7.5	--	--	--	--
8-12	0800	128	13.0	418	9.0	125	293	115	8.0	--	--	--	--
8-13	0800	432	14.0	877	41.0	839	38	735	7.1	--	--	--	--
8-14	0900	32	0.95	216	4.5	49	167	43	7.1	--	--	--	--
8-15	0900	264	3.6	576	7.0	91	485	81	7.7	--	--	--	--
8-16	1000	65	1.1	310	8.0	91	219	85	7.7	--	--	--	--
8-17	1000	69	4.0	319	7.0	71	248	68	7.7	--	--	--	--
8-18	1100	48	2.7	284	8.0	91	193	88	7.7	--	--	--	--
8-19	1100	105	0.080	420	19.0	134	286	126	6.7	--	--	--	--
8-20	1200	148	0.29	422	24.0	178	244	172	7.6	--	--	--	--
8-21	1200	45	0.040	520	12.0	63	457	62	8.1	--	--	--	--
8-22	1130	54	0.20	469	27.0	314	155	249	8.4	--	--	--	--
8-23	1300	46	2.3	288	6.0	70	218	52	8.2	--	--	--	--
8-24	1300	96	0.37	359	15.0	133	226	122	7.9	--	--	--	--
8-25	1430	78	0.24	239	8.0	96	143	90	7.8	--	--	--	--
8-26	1400	66	0.14	276	8.0	59	217	58	7.6	--	--	--	--
8-27	1515	92	0.040	296	10.0	67	229	63	7.7	--	--	--	--
8-28	1500	58	0.040	285	8.0	67	218	61	7.7	--	--	--	--
8-29	1700	80	0.010	295	14.0	79	226	64	7.5	--	--	--	--
8-30	0845	148	7.2	408	20.0	176	232	154	7.6	--	--	--	--
8-31	0900	95	0.56	388	10.0	122	266	107	7.8	--	--	--	--
9-2	1245	78	9.2	270	4.0	87	183	81	7.3	18	26.0	18	1.0
9-3	1515	114	0.30	360	11.0	106	254	99	7.9	20	22.5	19	3.0
9-4	1430	118	1.0	334	7.0	96	238	85	8.2	20	22.5	17	0.4
9-5	1550	114	0.80	347	11.0	130	217	126	6.8	20	22.5	18	2.0
9-6	1330	255	0.20	739	50.0	557	182	512	8.1	20	22.5	22	1.0
9-7	1730	136	0.12	419	12.0	197	222	186	8.6	25	27.5	18	1.0
9-8	1600	75	53.0	386	11.0	4	382	3	8.8	23	24.0	17	1.0
9-9	1830	249	2.0	565	15.0	198	367	184	8.2	--	--	--	--
9-10	--	--	--	--	--	--	--	--	--	--	--	--	--
9-11	1400	96	60.0	258	1.5	63	195	48	7.3	--	--	--	--
9-12	1600	128	30.0	438	17.0	132	306	124	7.9	--	--	--	--
9-13	1700	12	4.0	251	0.2	19	232	15	7.5	--	--	--	--
9-14	1600	88	0.8	630	40.0	364	266	343	9.1	--	--	--	--
9-15	1300	152	0.8	410	20.0	149	261	147	8.9	--	--	--	--
9-16	1000	126	40.0	354	10.0	93	261	92	7.9	--	--	--	--
9-17*	1300	255	30.0	660	15.0	118	542	110	7.9	--	--	--	--
9-18	1400	267	20.0	454	17.0	198	256	183	7.2	--	--	--	--
9-19	1530	78	10.0	296	8.0	75	221	69	8.2	--	--	--	--
9-20	1300	110	--	406	10.0	113	293	106	8.7	--	--	--	--
9-21	1030	68	20.0	252	3.0	44	208	40	7.6	--	--	--	--
9-22	1600	138	0.6	389	18.0	141	248	130	8.9	--	--	--	--
9-23	0930	63	20.0	229	35.0	55	174	47	7.5	--	--	--	--
9-24	1530	333	50.0	310	4.0	66	244	60	7.7	--	--	--	--

* Distinct green tint.

Table 9-2. Effluent Data, Lauderdale County Welcome Center.

Date Effluent	Time	BOD_5 mg/ℓ	F Coli per 100 mℓ	TS mg/ℓ	Settleable Solids mℓ/ℓ	SS mg/ℓ	DS mg/ℓ	VSS mg/ℓ	pH	Ammonia	TKN	Total Phosphate	NO_3-N
8-11	1000	2	<1	220	0	7	213	5	7.2	--	--	--	--
8-12	0800	6	<1	243	0	8	235	5	7.5	--	--	--	--
8-13	0800	6	14	256	0	14	242	8	7.7	--	--	--	--
8-14	0900	3	100	246	0.1	8	238	5	7.0	--	--	--	--
8-15	0900	3	11	272	0	7.3	265	6	7.0	--	--	--	--
8-16	1000	6	70	286	0.1	15	271	12	7.5	--	--	--	--
8-17	1000	3	12	258	0	8	250	7	7.4	--	--	--	--
8-18	1100	4	<1	216	0	7	209	6	8.0	--	--	--	--
8-19	1100	1	6	216	0.1	8	208	6	6.4	--	--	--	--
8-20	1200	3	10	280	0.1	7	273	6	7.3	--	--	--	--
8-21	1200	3	80	224	0	28	196	26	7.8	--	--	--	--
8-22	1130	1	<1	212	0.1	15	197	7	8.1	--	--	--	--
8-23	1300	1	1,000	213	0	5	208	4	7.6	--	--	--	--
8-24	1300	2	1,000	183	0	6	177	4	7.5	--	--	--	--
8-25	1430	3	1,500	180	0	6	174	5	7.6	--	--	--	--
8-26	1400	2	1,200	198	0	4	194	3	7.5	--	--	--	--
8-27	1515	3	240	208	0	6	202	5	7.4	--	--	--	--
8-28	1500	2	<1	446	0.1	8	438	7	7.4	--	--	--	--
8-29	1700	1	3,600	165	0	8	157	6	7.4	--	--	--	--
8-30	0845	1	30	172	0	4	168	4	7.6	--	--	--	--
8-31	0900	1	40	182	0	5	177	4	7.5	--	--	--	--
9-1	1000	1	<2	182	0	6	176	4	7.2	--	--	--	--
9-2	1245	2	80	183	0	5	178	3	7.6	8	9	2.7	1
9-3	1515	1	1,100	191	0	6	185	4	7.6	10	11	0.8	2
9-4	1430	2	1,200	194	0	6	188	4	7.3	10	10.5	1.0	3
9-5	1550	1	1,600	200	0	5	195	3	7.0	10	10.5	0.5	3
9-6	1330	<1	<1	192	0	6	186	4	7.0	5	5.5	0.9	5
9-7	1730	1	10	207	0	102	105	98	7.0	1	1.5	3.0	6
9-8	1600	1	1	208	0	6	202	5	7.2	4	4.5	2.8	6
9-9	1830	190	190	232	0.1	4	228	4	7.3	--	--	--	--
9-10	--	--	--	--	--	--	--	--	--	--	--	--	--
9-11	1400	<1	4	216	0.1	14	202	5	7.2	--	--	--	--
9-12	1600	1	7	209	0.2	4	205	3	7.0	--	--	--	--
9-13	1700	1	1,000	208	0	4	204	4	7.2	--	--	--	--
9-14	1600	5	700	230	0	4	226	3	7.2	--	--	--	--
9-15	1300	2	1	250	0	4	246	3.5	7.1	--	--	--	--
9-16	1000	1	<1	210	0	5	205	4	7.2	--	--	--	--
9-17	1300	1	400	222	0	7	215	6	7.3	--	--	--	--
9-18	1400	2	100	190	0	6	184	3	7.0	--	--	--	--
9-19	1530	1	200	250	0	5	245	4	7.0	--	--	--	--
9-20	1300	2	--	232	0	5	227	3	7.5	--	--	--	--
9-21	1030	7	4	209	0	4	205	3	7.4	--	--	--	--
9-22	1600	8	500	223	0	6	217	4	7.3	--	--	--	--
9-23	0930	6	<1	218	0	4	214	3	7.4	--	--	--	--
9-24	1530	24	<1	553	1	14	539	12	7.4	--	--	--	--
9-25*	--	8	8,000	276	0.1	10	266	7	7.2	--	--	--	--
9-26*	--	8	20,000	347	0	11	336	7	6.8	--	--	--	--

* Settling tank.

final BOD_5 of 30 mg/ℓ or 85% removal, therefore this plant is in compliance. A graph of influent and effluent suspended solids (SS) concentration (Figure 9-9) again shows the large variation in influent concentration and the efficiency of the plant (93% removal of SS). Once again note the very low level of SS in the effluent (average less than 10 mg/ℓ), while the influent SS averaged 140 mg/ℓ. The requirements of PL 92-500 are 30 mg/ℓ for SS or 85% removal, so this plant is in compliance. Similar plots of volatile suspended solids (Figure 9-10) and total solids (Figure 9-11) show the same pattern holding true. The average of volatile suspended solids in the influent was 128 mg/ℓ with the average effluent being less than 8 mg/ℓ.

The average of total solids (dissolved and suspended) entering the plant was 390 mg/ℓ with the average effluent being 230 mg/ℓ.

A graph of fecal coliforms entering the plant (Figure 9-12) again shows the wide range that may be expected in rest area wastewaters. Note that the scale is No./100 mℓ × 10^6, or millions of organisms. A plot of fecal coliforms in the effluent (Figure 9-13) show the marked reduction possible with chlorination (the previous figure used a scale of No./100 mg/ℓ × 10^6 and this one uses a scale of No./100 mℓ). Upon analysis of the effluent fecal coliforms a check of the plant showed the chlorinator to be malfunctioning. This malfunction has since been rectified.

The maximum, minimum, mean, and standard deviations of the wastewater characteristics analyzed are presented in Table 9-3. From Tables 9-1 through 9-3 and Figures 9-8 through 9-13 it is seen that the influent wastewater characteristics were similar to those previously discussed in Chapters 2 and 4 with the average 5-day biochemical oxygen demand of 123 mg/ℓ and an average suspended solids content of 140 mg/ℓ. It may also be seen from these tables and figures that the extended aeration package plant investigated was producing an effluent with very low levels of wastewater constituents. The effluent values for this plant were investigated for compliance or noncompliance with the requirements set forth in PL 92-500. A tabulation of the 7-day compliance data appears in Table 9-4 and the 30-day data appears in Table 9-5.

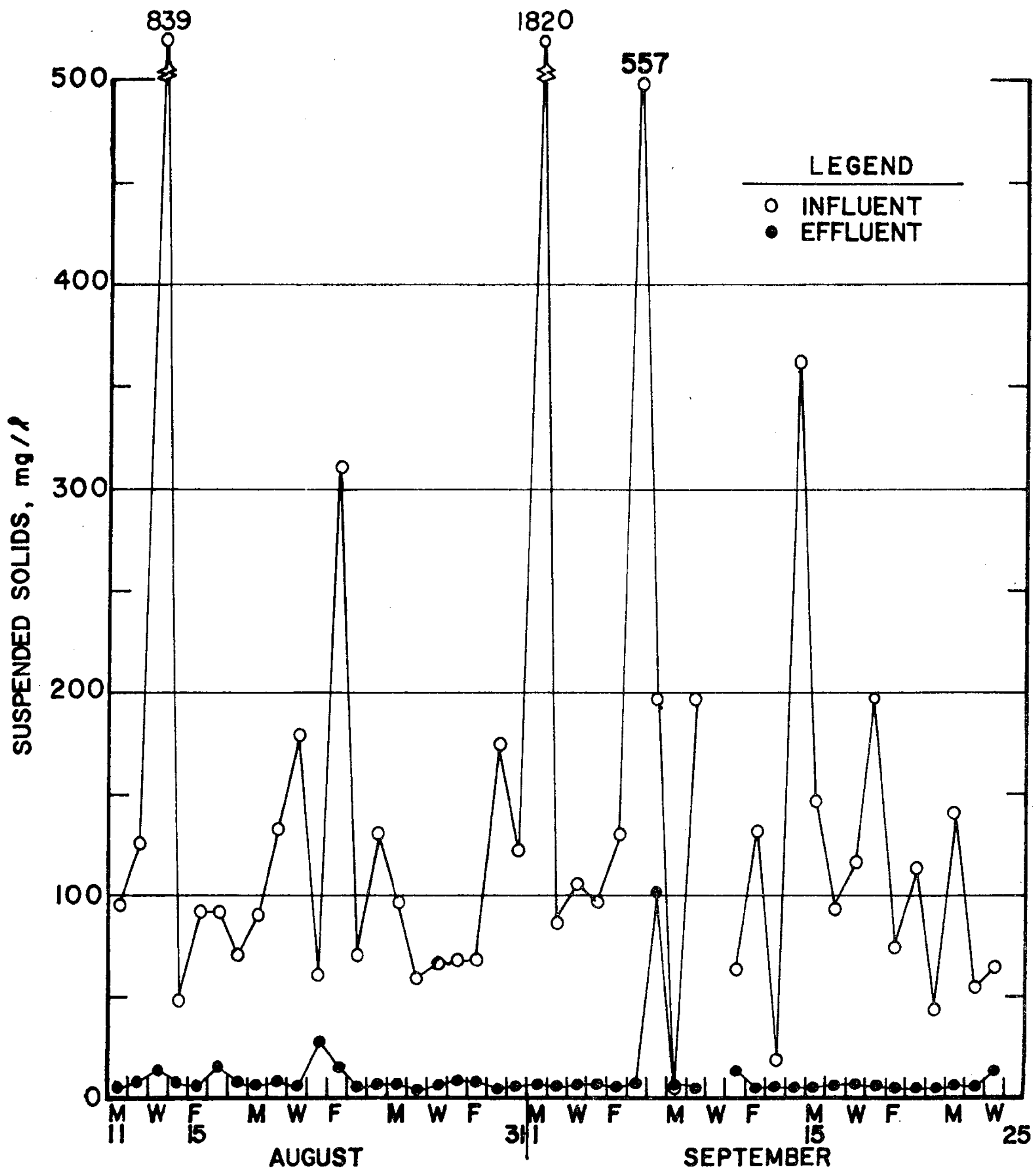

Figure 9-9. Influent and effluent suspended solids.

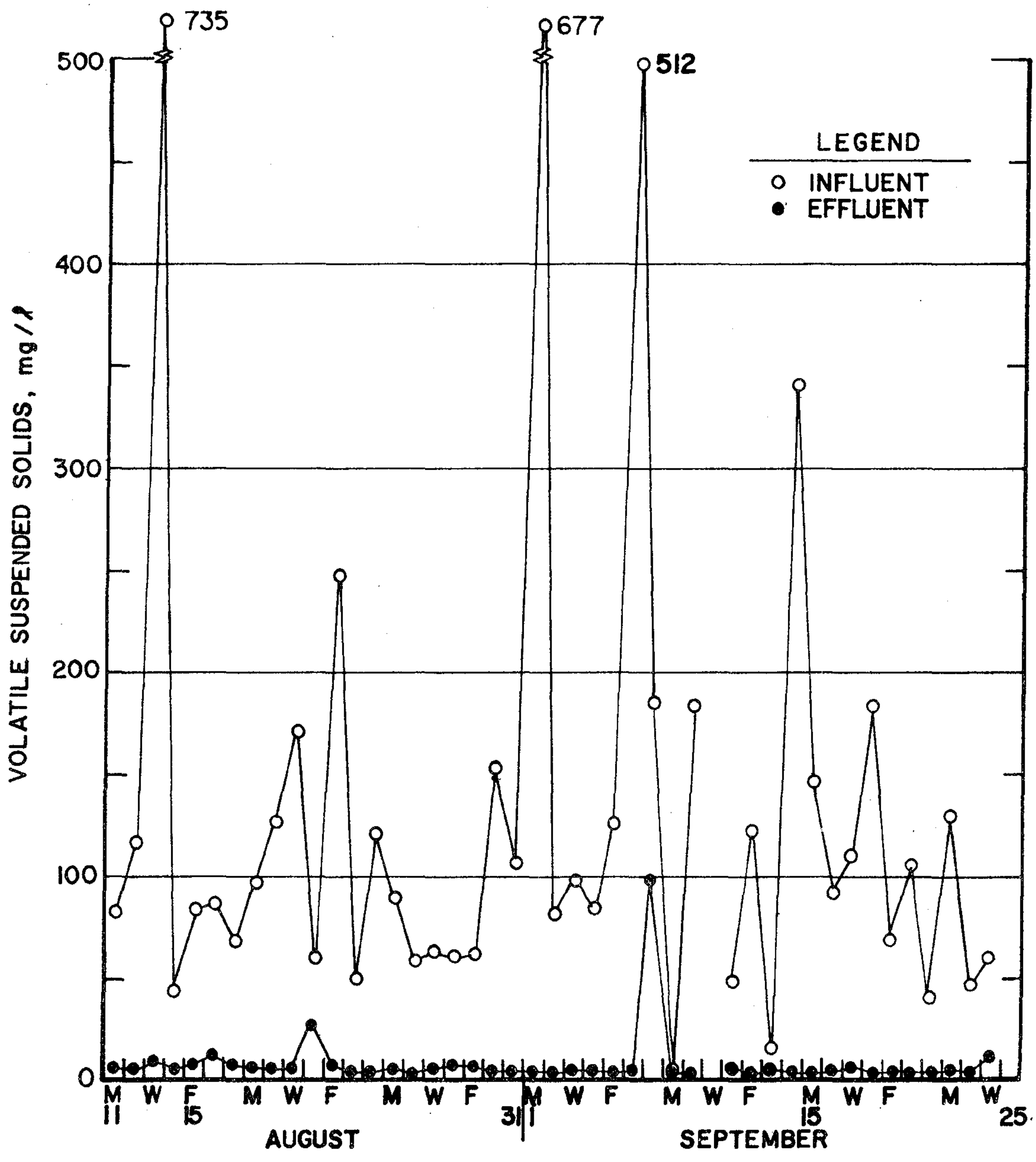

Figure 9-10. Influent and effluent volatile suspended solids.

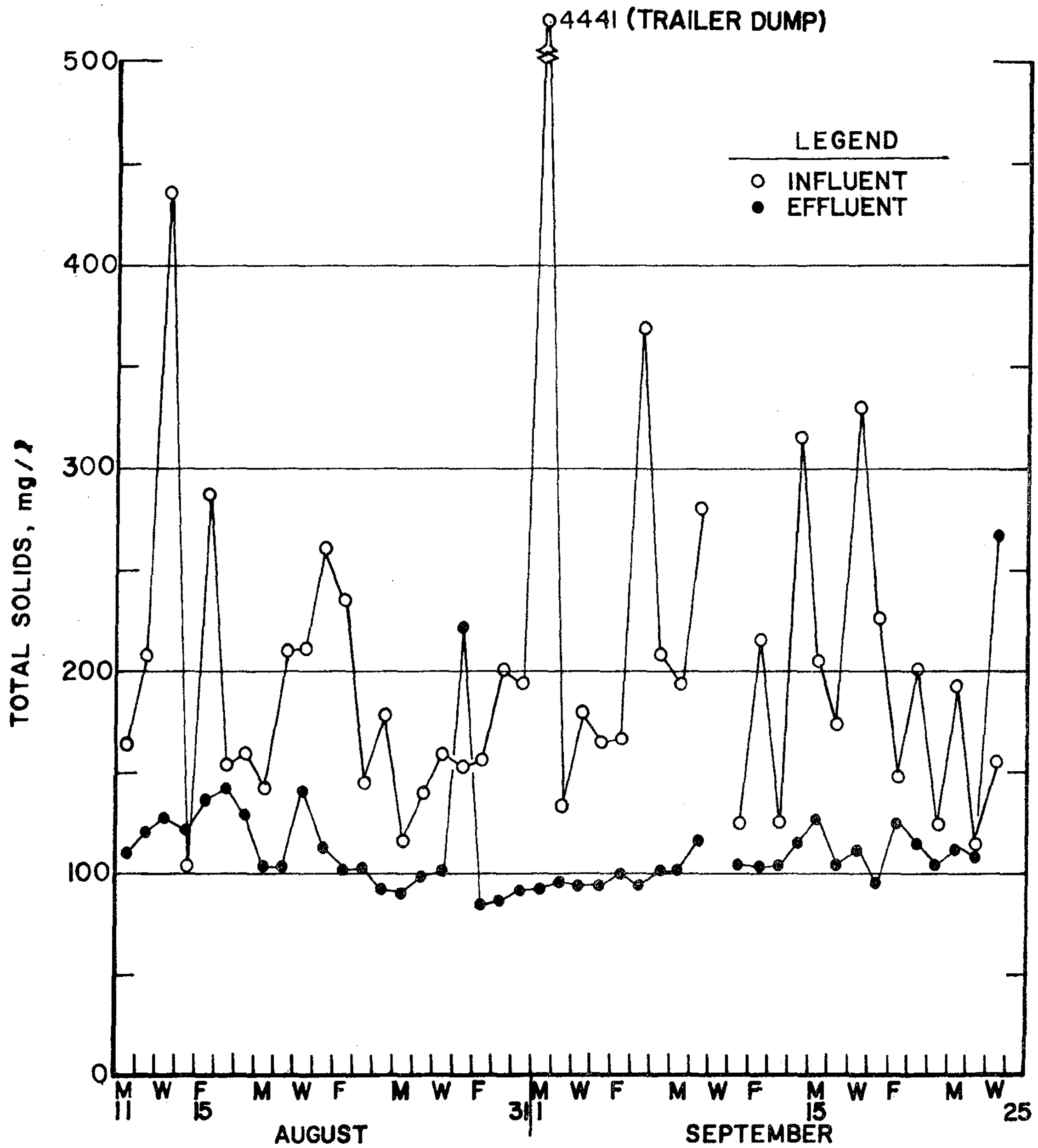

Figure 9-11. Influent and effluent total solids.

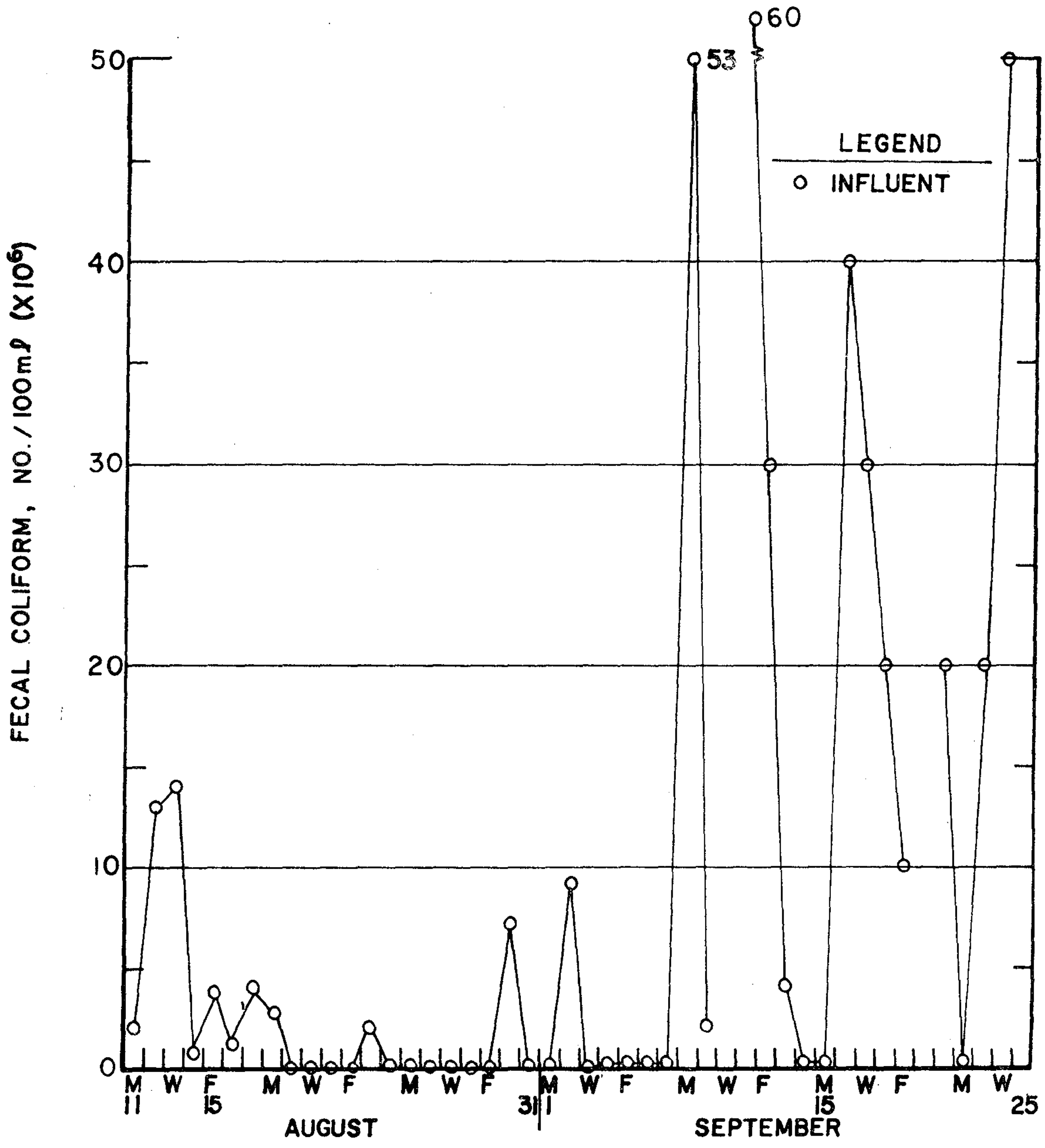

Figure 9-12. Influent fecal coliform count.

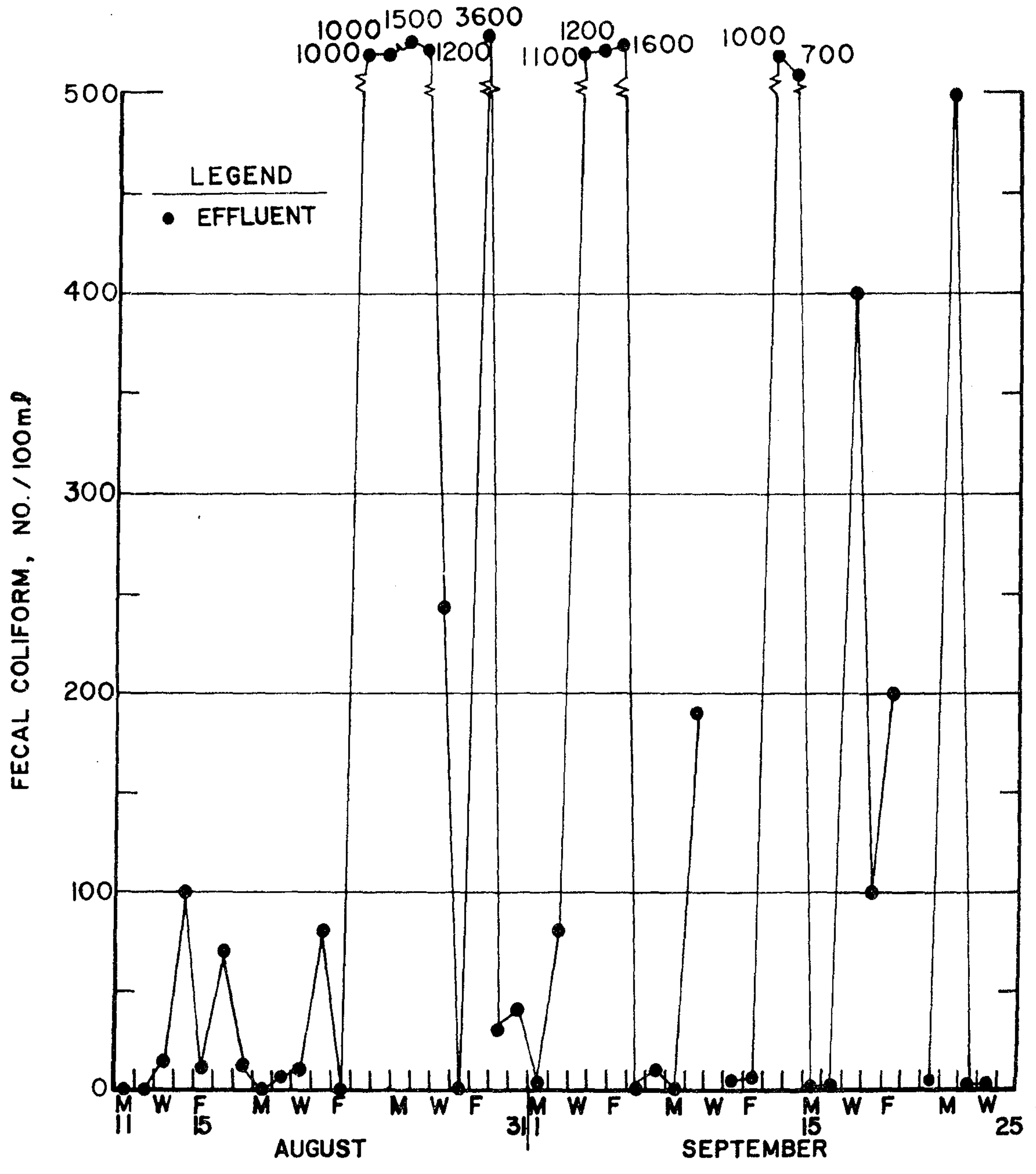

Figure 9-13. Effluent fecal coliform count.

Table 9-3. Wastewater Characteristics at a Mississippi Rest Area.

	BOD_5 mg/ℓ	TS mg/ℓ	Settleable Solids mg/ℓ	SS mg/ℓ	DS mg/ℓ	VSS mg/ℓ	Ammonia	TKN	Total Phosphate	NO_3-N
					Influent					
Maximum	432	877	50	839	542	735	25	27.5	22	3
Minimum	12	216	0.2	4	38	3	18	22.5	17	0.4
Mean	123.4	389.5	14.0	139.7	249.9	127.2	20.9	23.9	18.4	1.3
Standard Deviations	86.3	141.4	10.7	145.4	88.4	129.3	2.3	2.0	1.7	0.9
Observations	43	43	43	43	43	43	7	7	7	7
					Effluent					
Maximum	24	553	1	102	539	98	10	11	3	6
Minimum	1	165	0	4	105	3	1	1.5	0.5	1
Mean	2.9	229.2	0.1	9.5	219.8	7.5	6.9	7.5	1.7	3.7
Standard Deviation	3.8	66.9	0.2	14.9	68.0	14.5	3.6	3.7	1.1	1.9
Observations	44	44	44	44	44	44	7	7	7	7

Table 9-4. Effluent PL 92-500, Toomsuba Rest Area.

Sampling Period days	7-Day Arithmetic Mean BOD$_5$ mg/ℓ	7-Day Arithmetic Mean Suspended Solids mg/ℓ	7-Day Geometric Mean Fecal Coliform No./100 m	pH Range max	pH Range min
1-7	4	10	10	7.7	7.0
2-8	4	10	10	8.0	7.0
3-9	4	10	13	8.0	6.4
4-10	3	9	13	8.0	6.4
5-11	3	11	12	8.0	6.4
6-12	3	13	9	8.1	6.4
7-13	2	11	13	8.1	6.4
8-14	2	11	24	8.1	6.4
9-15	2	11	82	8.1	6.4
10-16	2	10	170	8.1	7.3
11-17	2	10	280	8.1	7.4
12-18	2	7	150	8.1	7.4
13-19	2	6	470	7.6	7.4
14-20	2	6	290	7.6	7.4
15-21	2	6	180	7.6	7.4
16-22	2	6	59	7.6	7.2
17-23	2	6	40	7.6	7.2
18-24	1	6	50	7.6	7.2
19-25	1	6	140	7.6	7.2
20-26	1	5	120	7.6	7.0
21-27	1	6	75	7.6	7.0
22-28	1	19	62	7.6	7.0
23-29	1	19	56	7.6	7.0
24-30	1	19	63	7.6	7.0
25-31	1	22	39	7.3	7.0
26-32	1	23	15	7.3	7.0
27-33	1	23	6	7.3	7.0
28-34	1	22	19	7.3	7.0
29-35	2	6	39	7.3	7.0
30-36	2	6	39	7.3	7.0
31-37	2	6	16	7.2	7.0
32-38	2	6	26	7.3	7.0
33-39	2	5	41	7.3	7.0
34-40	2	5	66	7.3	7.0
35-41	2	5	42	7.5	7.0
36-42	2	5	18	7.5	7.0
37-43	3	5	50	7.5	7.0
38-44	4	5	50	7.5	7.0
39-45	7	6	18	7.5	7.0

Table 9-5. Effluent PL 92-500, Toomsuba Rest Area.

Sampling Period days	30-Day Arithmetic Mean BOD$_5$ mg/ℓ	30-Day Arithmetic Mean Suspended Solids mg/ℓ	30-Day Geometric Mean Fecal Coliform No./100 m	pH Range max	pH Range min
1-30	2	11	38	8.1	6.4
2-31	2	11	43	8.1	6.4
3-32	2	11	45	8.1	6.4
4-33	2	11	44	8.1	6.4
5-34	2	11	48	8.1	6.4
6-35	2	11	55	8.1	6.4
7-36	2	10	48	8.1	6.4
8-37	2	10	44	8.1	6.4
9-38	2	10	54	8.1	6.4
10-39	2	10	59	8.1	7.0
11-40	2	10	66	8.1	7.0
12-41	2	9	65	8.1	7.0
13-42	2	9	69	7.6	7.0
14-43	2	9	67	7.6	7.0
15-44	2	9	52	7.6	7.0
16-45	3	9	39	7.6	7.0

In summary the following points should be noted:

a. This extended aeration activated sludge package plant was hydraulically overdesigned by a factor of 3 to 5.

b. All effluent BOD_5 grab and composite samples met the requirements of PL 92-500. All 7-day mean BOD_5's met requirements of PL 92-500. All 30-day BOD_5's met requirements of PL 92-500. All 7-day mean SS met requirements of PL 92-500. All 30-day mean SS met requirements of PL 92-500. All pH samples met requirements of PL 92-500.

c. Overall plant efficiency (based on average of grab samples) was 98% removal of BOD_5 and 93% removal of SS.

Temperature affects the removal efficiency for carbonaceous BOD in an extended aeration package plant. The BOD removal rate constant is related to temperature by

$$K_T = K_{20}(1.03)^{T-20} \tag{9-7}$$

where

K_T = removal rate constant, ℓ/mg/day, at temperature T, °C

K_{20} = removal rate constant, ℓ/mg/day, at 20°C

Use of the factor above in the BOD removal equation will allow determination of temperature for organic removal. Temperatures less than 15°C will reduce significant nitrification. However, regulatory agencies recognize this and in some cases do not require biological systems to remove TKN during the winter months. Removal of TKN is not as important during the winter since nitrification and ammonia toxicity in the stream are less in the winter months, and greater dilution of wastewater is provided in streams if it rains in the winter. If it is desired that nitrification take place during the winter the extended aeration package plant may be covered or enclosed in a shelter or building. Enclosure of the plant would enable the plant to operate at a higher temperature than if the plant were exposed to the weather. It may also be deemed necessary to heat the plant enclosure to ensure an adequate temperature for efficient plant operation.

Extended aeration package plants are capable of withstanding the fluctuations in organic (BOD) loading normally encountered at a rest area. Completely mixed plants are better than plug flow systems in this regard. The detention time in the aeration tank is of sufficient capacity for the microorganisms to respond to periodic increases in organic concentration. The only foreseeable problem would occur where trailer dumping facilities are provided and no provision is made for equalizing this loading to the system.

Hydraulic (Q) shock loadings from rest areas may have an adverse effect on the performance of an extended aeration plant. The detention time in the aeration tank is decreased, but more importantly, the increase in overflow rate and the decrease in detention time in the clarifier may result in carryover of solids in the effluent. Additionally, the loss of solids may reduce the MLVSS concentration to a point where soluble BOD removal decreases. Because of this problem, clarifier overflow rates for small extended aeration plants are 300 gpd/ft^2 rather than 600 gpd/ft^2 as for larger systems. Equalization of flow either in an auxiliary surge tank (prior to the aeration tank) or in the aeration tank (by oversizing the aeration tank) may be required to dampen the peak flows. The fluctuating hydraulic loads occur due to fluctuating use of the rest area, but the actual flow to the plant is often governed by design and operation of the wet well and pumping station to the plant. Oversizing of these pumps results in short surges of flow many times greater than the design flow for the clarifier. Therefore, pumps should be designed in accordance with actual flows.

The organic and hydraulic overdesign of extended aeration plants is a result of high estimates of water usage and organic concentration, as well as design of systems for a predicted future rate of usage. An organically underloaded plant operates at such a low F/M that the concentration of active biological mass in the aeration tank is not great enough for the system to operate as designed. The solids produced are not flocculent, but rather they are dispersed and do not settle well. Pfeffer found that a minimum BOD level of 9.0 lb per 1000 cu ft was necessary to achieve 95 percent removal of suspended solids in the clarifier.[1] Underloading conditions usually optimize the nitrification reaction. With longer detention time in the clarifier, denitrification accompanied by rising sludge will also be a greater problem.

In an effort to provide an extended aeration package plant that will function efficiently both when installed and at the end of design life many rest areas and other institutions where low (<50,000 gpd) predominate utilize a type of treatment plant called "modular construction." Modular construction plants are extended aeration package plants with more than one aeration chamber. These plants can be operated with the aeration chambers arranged in series or in parallel. As an example a modular construction plant consists of three aeration chambers and one clarifier. At the start up of the plant only one aeration chamber and the clarifier is required to produce the desired effluent. This plant is reevaluated within the first 5 years of service and periodically (not to exceed 5 years) thereafter at which time flow to the plant may have increased sufficiently to require that two aeration chambers and the clarifier are needed to produce the desired effluent. After another 10 years of service it may be found that all three aeration chambers and the clarifier are required to produce the desired effluent. At this point the modular construction plant is operating at maximum capacity and any future increase in flow will lower detention time and thus efficiency. However, because the plant was modular it has been operated at design detention time for 15 years; i.e. the plant has not had to be upgraded or redesigned. Modular construction of extended aeration package plants is therefore a viable method of treating rest area generated wastewater.

It is felt by the authors of this report that modular construction extended aeration package plants with at least two aeration chambers are better suited for use at rest areas than with only one aeration chamber. It is also felt that these plants should be reevaluated at least every five years to determine if one, two, or three section tanks are necessary to provide the design detention time or if additional clarifiers are necessary.

Overloading an extended aeration system may be less likely to create problems than severe underloading. Pfeffer loaded an extended aeration system at 40 lb BOD_5 per 1000 cu ft and achieved 90 percent BOD removal but only 80 percent suspended solids removal.[1] Overloading the system causes an increase in solids production and oxygen uptake of the mixed liquor. The volume of sludge wasted would have to be increased, and aeration equipment would have to be capable of transferring more oxygen to the mixed liquor. The solids load to the clarifier may be beyond its settling capability.

Extended aeration plants at rest areas may be upset by introduction of toxic materials to the system. Normal rest area waste is not expected to contain toxic materials provided that comfort station cleaning agents are carefully selected for their biodegradability. If facilities are provided for trailer dumping, then this potentially toxic wastewater should be slowly added to the system in order to allow sufficient dilution in the aeration tank. It may be necessary either to treat trailer dumping wastes seperately or to provide an equalization tank prior to the extended aeration tank to thoroughly mix the trailer dumping wastes with the wastes from the comfort station. Toxic materials would destroy the active microbial mass in the system or result in production of a sludge with poor settleability. Nitrification may be severely inhibited by toxic materials. The pH of the mixed liquor can be reduced by addition of toxic materials by carbonic acid produced from carbon dioxide (CO_2) and water, or by nitric acid produced when nitrification produces significant concentrations of nitrate (NO_3).

9-3. FLEXIBILITY

Typical extended aeration systems with one aeration tank only lack the flexibility necessary for expansion to accommodate an increased hydraulic load. Again this problem is primarily related to the final clarifier. The most expensive part of an extended aeration system is the tank including aeration tank and clarifier. Enlargement of the clarifier would require an additional tank or significant modification of the existing tank. In contrast to the clarifier, the aeration tank could accept increased loadings, hydraulically and organically. The F/M could be increased by a factor of four and still be in the range acceptable for activated sludge systems. Aeration and sludge handling facilities could be expanded to accommodate this loading with a modest investment. Some manufacturers build extended aeration systems with modular aeration tanks so that their capacity may be increased as use of the facility increases. Seasonal loadings could be accommodated by this arrangement, as well as future increases in loading.

Another method for providing flexibility in an extended aeration system is modular construction. With more than one aeration tank included in the extended aeration plant it is possible to maintain the design detention time through the plant even if the hydraulic load has been increased through the life of the plant.

A higher quality effluent can be achieved by modification of operation or by adding additional unit processes to the extended aeration system. Operational changes can be made to enhance ammonia removal. Nitrification-denitrification units may be added to an existing system for removal of total nitrogen. Physical-chemical processes may be added to achieve phosphorus removal. Suspended solids removal may be upgraded by adding a filter.

9-4. RELIABILITY

To achieve reliable operation from an extended aeration system, frequent operator inspections are necessary. The mechanical equipment provided on package plants generally operates for long periods without breakdown, but when a failure does occur, the biological system may be destroyed. Frequent operational problems encountered will be described in paragraph 9-5 below. Power failure could completely disrupt the system and result in untreated effluent and nuisance conditions. If the plant is frequently inspected, any malfunction can generally be corrected in a short time. However, restoration of the biological system may require several days. When operating as designed, an extended aeration system will consistently provide an effluent that will meet the secondary treatment requirements of PL 92-500.

9-5. OPERATION

The operation of an extended aeration package plant is more complex than operation of most of the other secondary treatment systems used at rest areas. However, a technical background and significant training are not prerequisites for learning to operate an efficient extended aeration plant. An operator can be taught the basic principles of activated sludge, the mechanics of operation, and the indicators of microbiological problems requiring technical assistance with a minimum amount of formal training. If chemical analyses of the plant effluent are performed at the plant, more training in regard to analytical procedures would be required.

Most of the operational problems associated with extended aeration systems involve maintaining good sludge settleability. The most commonly occurring problem at rest areas is likely to be rising sludge in the clarifier caused by denitrification. This problem may be minimized by adjusting the sludge return rate so that the sludge blanket is not allowed to remain in the clarifier longer than is necessary for sludge settleability. Daily removal of the scum that is formed on the clarifier surface can help prevent the solids from being discharged in the effluent. Airlift pumps for scum return generally are operated most efficiently when an operator is present so that the solids can be moved manually near the vicinity of the pump intake. Periodic (twice daily) scraping of the walls of the clarifier's entire depth will improve the efficiency of the clarifier by removing solids buildup that will denitrify and eventually float to the surface.

Control of sludge age is also important to efficient operation of the biological mass and to good sludge settleability. Sludge age for an extended aeration system is usually 20-30 days. The sludge age is controlled by wasting solids either from the sludge recycle line or from the aeration tank. Wasting from the recycle line has the advantage of having a higher concentration of solids than the mixed liquor. However, in order to evaluate the volume to be wasted, the solids concentration of the sludge recycle must be known. On the other hand, wasting from the aeration tank can be easily controlled by wasting a fraction of the mixed liquor or aeration tank volume on a regular schedule, assuming loss of solids in the effluent is negligible. For example, if a sludge age of 25 days is required, this could be accomplished by wasting a volume of mixed liquor equal to 4 percent of the aeration tank volume each day. If the loss of solids in the effluent is significant, the volume wasted may need to be reduced correspondingly. For extended aeration systems, weekly wasting would be more applicable for the small volumes involved. The mixed liquor wasted, being relatively low in solids, should be allowed to settle in a sludge decant unit or an unaerated sludge holding tank in order to concentrate the solids and minimize the volume sludge for disposal. The supernatant must be returned to the aeration tank.

The dissolved oxygen in the aeration tank must be maintained at concentrations greater than 2.0 mg/ℓ. Lower concentrations favor the growth of filamentous organisms (Spaerotilus) rather than the growth of floc-forming bacteria. Filamentous growth produces a bulking sludge that cannot be clarified. Introduction of toxic materials to the aeration tank may destroy the active bacterial mass and result in a high F/M ratio that will also favor filamentous growth. Bulking sludge may be destroyed by holding the sludge in an anaerobic environment for at least 6 hr.

Foaming in the aeration tank is often a problem at extended aeration plants. Some plants are equipped with sprays to control this problem. Foaming deposits solids on the walls of the aeration tank above the liquid level. If not removed, these solids may cause odors and detract from the aesthetics of the plant. Daily removal of foam can be accomplished by spraying the surface with water.

Mechanical problems for extended aeration plants can be minimized by daily cleaning and inspection. Clogging of airlift pump lines for sludge and scum return is a frequent problem; therefore, these lines should be accessible for cleaning. Where diffusers are used for aeration, they must be periodically pulled out of the aeration tank and cleaned. Debris should be removed daily from bar screens, and comminutors should be kept in good working order. Mechanical equipment should be lubricated in accordance with the manufacturer's instructions.

An extended aeration system is dependable and will provide satisfactory service if it is properly maintained and operated. For rest areas daily cleaning and inspection should be required. It is estimated that this can be accomplished with less than one man-hour per day.

9-6. PRELIMINARY DESIGN

Design of an extended aeration system is based on mathematical expressions derived from microbial kinetics in a completely mixed reactor, i.e., the aeration tank. McKinney, Eckenfelder, Goodman, and Lawrence and McCarty, and others have developed models for design of activated sludge systems. The process design parameters derived from the various models or design procedures generally do not differ significantly. Typical design parameters for an extended aeration system are given in Table 9-6.

Table 9-6. Extended Aeration Design Criteria.

Aeration Tank	
Detention time, hr (t)	18-36
Sludge age, days (t_s)	20-30
Food-to-microorganisms ratio, lb BOD_5/lb MLVSS-day (F/M)	0.05-0.15
BOD_5 loading, lb BOD_5/1000 cu ft aeration tank	10-25
MLSS, mg/ℓ	3000-6000
Sludge return rate, % of influent flow	50-200
Air required, lb O_2/lb BOD_5-day	>1.5
MLVSS, mg/ℓ (X_v)	2100-4200
Recycle flow/average flow (R)	0.50-2.0
θ	1.0-1.03
Clarifier	
Overflow rate, gpd-sq ft (OFR)	100-300
Detention time, hr	4
Solids loading, lb/sq ft-hr (SLR)	0.5-1.25
Weir loading, gpd-ft	10,000

Manufacturers of package extended aeration plants have designed their plants to treat domestic wastewaters with average characteristics, and the design parameters calculated for such plants are generally in agreement with the values given in Table 9-6. Buyers for package plants, therefore, often need to determine design flow only and choose the plant designed by the manufacturer to efficiently treat that volume of wastewater. However, problems may arise in using the manufacturer's design for a plant receiving wastewater that is not typical of average domestic wastewater in some respects. Carbonaceous BOD_5 is lower for rest area wastewaters than for domestic; therefore, the package plant selected for a given flow would be oversized from the standpoint of BOD loading. The concentration of ammonia nitrogen in rest area wastewater should be a concern to design engineers because of the increased oxygen demand and the sludge settling problems associated with densification of a nitrified mixed liquor in the final clarifier. The effect of shock hydraulic loadings produced at rest areas should also be evaluated since some package plants may not be designed to accommodate such fluctuations. Failure to utilize the basic fundamentals of biological wastewater treatment in selecting and operating extended aeration package plants often results in ineffective systems. In an effort to enable the rest area engineer to evaluate the package plant design and to determine the effect of parameters such as MLVSS, sludge age, sludge recycle, and oxygen uptake on the operation of the plant, design procedures developed from Eckenfelder's formulations are presented below:[2]

a. Inputs required.

(1) Wastewater flows and characteristics.

Q = average daily flow, gpd

P = peak BOD_5/average BOD_5 = 2 (normal for rest area wastewater)

S_o = influent BOD_5, mg/ℓ

SS_i = influent suspended solids, mg/ℓ

VSS_i = influent volatile suspended solids, mg/ℓ

TKN_i = influent total Kjeldahl nitrogen, mg/ℓ

(2) Temperatures, winter and summer.

Design constants (values given are literature values for domestic wastewater. Laboratory tests are needed to determine exact design constants for rest area wastewater.)

a = fraction of BOD_5 synthesized = 0.73

a_o = fraction of BOD_5 synthesized to degradable solids = 0.77a = 0.56

a' = fraction of BOD_5 oxidized for energy = 0.52

b = endogenous respiration rate in BOD_5 equivalents = 0.075/day

b' = fraction of VSS oxidized/day = 0.15/day

f = nonbiodegradable fraction of VSS in influent = 0.40

f' = degradable fractions of MLVSS = 0.53

K_{20} = BOD removal rate constant at 20°C = 0.0007-0.002 ℓ/mg-hr (use 0.001 ℓ/mg-hr)

θ = temperature coefficient = 1.02-1.10 (use 1.03)

b. Design calculations.

(1) Calculate the volume of the aeration tank using a detention time of 24 hours (1 day).

$$V = Qt$$

where

V = volume of aeration tank, gal

Q = flow, gpd

t = detention time, days

(2) Calculate the MLVSS in mg/ℓ (X_v).

$$X_v = \frac{a_o S_r}{bf'(t)}$$

where

X_v = MLVSS, mg/ℓ

a_o = design constant = 0.56

$S_r = S_o - S_e$

b = design constant = 0.075/day

f' = design constant = 0.53

t = detention time, days = 1 day

The term $(S_o - S_e)$ is the BOD_5 removed by the system excluding the effluent BOD_5 contribution from the effluent suspended solids and is noted S_r . An extended aeration plant can produce an effluent soluble BOD_5 (S_e) of less than 5 percent of the total influent BOD_5. Therefore, S_e is assumed for design purposes (5 mg/ℓ).

(3) Calculate the food to microorganism ratio (F/M) and check against standard values (Table 9-1).

$$F/M = \frac{S_o}{X_v t}$$

where

F/M = lb BOD_5/lb MLVSS per day
S_o = influent BOD_5, mg/ℓ
X_v = MLVSS, mg/ℓ
t = detention time, days

(4) Assume a temperature constant (K_{20}) and adjust for mean winter temperature (T).

$$K_T = K_{20}\theta^{T-20}$$

where

K_T = adjusted temperature constant
K_{20} = BOD removal rate constant = 0.001 ℓ/mg-hr
θ = temperature coefficient = 1.03
T = mean winter temperature, °C

(5) Check effluent soluble BOD_5 (S_e).

$$S_e = \frac{S_o}{1 + K_T X_v t}$$

where

S_e = effluent soluble BOD_5, mg/ℓ
S_o = influent BOD_5, mg/ℓ
K_T = K_{20} adjusted for temperature T
X_v = MLVSS, mg/ℓ
t = detention time, days

Check calculated S_e against assumed S_e in (2) and redo steps 2-5 with new S_e if necessary.

(6) Calculate the oxygen required per day (O_2 lb/day). In order for the extended aeration system to function as designed, adequate oxygen for the microorganisms must be supplied. Oxidation of influent BOD_5 for energy, oxidation of endogenous mass, and nitrification of organic and and ammonia nitrogen are all sinks for oxygen in the aeration tank. These requirements are described mathematically in the equation

$$O_2(\text{lb/day}) = \left[a'S_r + N_f(4.57)TKN_i\right]QP(8.34 \times 10^{-6}) + b'X_vV(8.34 \times 10^{-6})$$

where

O_2 = pounds of oxygen required per day

a' = constant = 0.52

$S_r = S_o - S_e = BOD_5$ removed

N_f = fraction TKN removed = 0.6

4.57 = conversion factor

TKN_i = total Kjeldahl nitrogen of influent

Q = flow, gpd

P = peak organic load/average organic load = 2

8.34×10^{-6} = conversion factor

b' = constant = 0.16/day

X_v = MLVSS, mg/ℓ

V = volume aeration tank, gal

(7) Check the pounds of oxygen supplied per pound BOD_5 removed per day

$$\frac{\text{lb } O_2}{\text{lb } BOD_r} = \frac{O_2(\text{lb/day})}{QS_r(8.34 \times 10^{-6})}$$

where

lb O_2/lb BOD_r = pounds of oxygen per pound BOD removed

O_2(lb/day) = pounds of oxygen per day (from (f))

Q = flow, gpd

$S_r = S_o - S_e = BOD_5$ removed

8.34×10^{-6} = conversion factor

Check against standard values (Table 9-1).

(8) Design the aeration system (diffused) and adjust the standard transfer efficiency (STE) to operating conditions.

$$OTE = STE \frac{\left(C_{s_m} \beta p - C_L\right)}{9.17} \alpha (1.024)^{T-20}$$

where

OTE = operating transfer efficiency

STE = standard transfer efficiency, 5-8%

β = constant = 0.9

p = correction factor for pressure = 1.0

C_L = minimum dissolved oxygen = 2.0 mg/ℓ

9.17 = conversion factor

α = constant = 0.9

1.024 = conversion factor

T = temperature, °C

and

$$C_{s_m} = C_s\left(\frac{p_b}{29.4} + \frac{O_t}{42}\right)$$

where

C_{s_m} = oxygen saturation for aeration tank at middepth

C_s = oxygen saturation (from Table E3 in Appendix E)

p_b = pressure at tank bottom, psig, use 18.2 psig

O_t = oxygen exit gas, %, use 18%

(9) Calculate the required airflow, standard cubic feet per minute (scfm).

$$\text{scfm} = \frac{O_2(\text{lb/day}) \times 100}{OTE(\%)(0.0174)\ 1440}$$

where

scfm = standard cubic feet per minute, required airflow

O_2 = oxygen required per day (from (f))

100 = conversion factor

OTE = oxygen transfer efficiency,%

0.0174 = conversion factor

1440 = conversion factor

(10) Calculate scfm for mixing. This requires 20-30 scfm per 1000 cu ft of tank volume.

$$\frac{\text{scfm}}{1000 \text{ cu ft}} = \frac{\text{scfm}}{V} 7.48 \times 10^3$$

where

scfm/1000 cu ft = scfm required for mixing

scfm = standard cubic feet per minute, required air flow

V = volume of aeration tank in gallons

7.48×10^3 = conversion factor

Note: If scfm/1000 cu ft is <20, use 20 scfm.

(11) Calculate the amount of sludge to be wasted (Q_w) from the aeration tank neglecting solids lost in effluent. Note: sludge is usually wasted only once a year.

$$Q_w = \frac{V}{t_s}$$

where

Q_w = sludge to be wasted, gpd

V = volume of aeration tank, gallons

t_s = sludge age, days (20-30 days)

(12) Calculate the sludge recycle ratio (R) needed to maintain the correct mixed liquor suspended solids content ratio. Note: after operation commences the plant operator will adjust the recycle to maintain proper performance.

$$R = \frac{Q_r}{Q} = \frac{\text{MLSS}}{X_r - \text{MLSS}}$$

where

R = recycle ratio

Q_r = recycle flow, gpd

Q = influent glow, gpd

MLSS = 1.43 MLVSS = $1.43X_v$

X_r = suspended solids concentration in return sludge line = 8000 mg/ℓ

(13) Calculate total effluent BOD_5.

$$\left(BOD_5\right)_e = S_e + (TSS)_e 0.3$$

where

$\left(BOD_5\right)_e$ = total effluent BOD_5, mg/ℓ

S_e = effluent soluble BOD_5, mg/ℓ from (5)

$(TSS)_e$ = total suspended solids effluent, mg/ℓ (20 mg/ℓ)

0.3 = design constant

(14) Design final clarifier.

(a) Calculate surface area (SA)

$$SA = \frac{Q\left(1 + \frac{Q_r}{Q}\right)}{OFR}$$

where

SA = surface area, sq ft

Q = influent flow, gpd

Q_r = recycle flow, gpd

OFR = overflow rate, gpd/sq ft (300 gpd/sq ft)

(b) Check the solids loading rate (SLR) with Table 9-1.

$$SLR = \frac{MLSS\left(1 + \frac{Q_r}{Q}\right)Q}{SA\ 24}\ 8.34 \times 10^{-6}$$

where

SLR = solids loading rate, lb/sq ft/day

MLSS = 1.43 MLVSS = $1.43X_v$

Q_r = recycle flow, gpd

Q = influent flow, gpd

SA = surface area, sq ft

8.34×10^{-6} = conversion factor

24 = conversion factor

c. Output obtained.

V = volume aeration tank, gal

X_v = operating MLVSS, mg/ℓ

F/M = food to microorganism ratio

K_T = adjusted temperature constant

S_e = effluent soluble BOD_5, mg/ℓ

O_2 = pounds of oxygen required per day, lb/day

lb O_2/lb BOD_r = pounds of oxygen per pound BOD removed

OTE = operating transfer efficiency, %

scfm = standard cubic feet per minute required airflow

scfm/1000 cu ft = scfm required for mixing

Q_w = sludge to be wasted, gpd

R = recycle ratio

$\left(BOD_5\right)_e$ = total effluent BOD_5 mg/ℓ

SA = surface area of clarifier, sq ft

SLR = solids loading rate to clarifier, lb/sq ft/day

d. Example calculations.

The following parameters are known:

Q = flow = 6000 gpd

S_o = influent BOD_5 = 165 mg/ℓ

S_e = 5 mg/ℓ

P = peak BOD_5/avg. BOD_5 = 2

T = temperature = 15°C (mean winter temperature)

SS_i = influent suspended solids = 190 mg/ℓ

VSS_i = influent volatile suspended solids = 170 mg/ℓ

TKN_i = total Kjeldahl nitrogen = 30 mg/ℓ

Also the following design constants are assumed

$a = 0.73$

$a_o = 0.56$

$a' = 0.52$

$b = 0.075$/day

$b' = 0.15$/day

$f = 0.40$

$f' = 0.53$

$K_{20} = 0.001$ ℓ/mg-hr

$\theta = 1.03$

$N_f = 0.6$

STE = 6%

$\beta = 0.9$

$p = 1.0$

$C_L = 2.0\ mg/\ell$

$\alpha = 0.9$

$P_b = 18.2$ psig

$O_t = 18\%$

$t_s = 25$ days

$X_r = 8000\ mg/\ell$

$(TSS)_e = 20\ mg/\ell$

(1) Calculate volume of aeration tank. Assume detention time (t) = 24 hr = 1 day.

$$V = Qt$$

$$V = 6000 \text{ gpd} \times 1 \text{ day}$$

$$\underline{V = 6000 \text{ gal}}$$

(2) Calculate the MLVSS in mg/ℓ (X_v).

$$X_v = \frac{a_o S_r}{bf't}$$

$$X_v = \frac{0.56(165 - 5)}{0.075/\text{day } 0.53 \text{ 1 day}}$$

$$\underline{X_v = 2254 \text{ mg}/\ell}$$

X_v checks with Table 9-1.

(3) Calculate the food to microorganism ratio (F/M).

$$F/M = \frac{S_o}{X_v t}$$

$$F/M = \frac{165 \text{ mg}/\ell}{2254 \text{ mg}/\ell}$$

$$F/M = \underline{0.07}$$

F/M checks with Table 9-1.

(4) Calculate K_T for mean winter 15°C.

$$K_T = K_{20}\theta^{T-20}$$

$$K_T = 0.001\ \ell/\text{mg-hr}\ 1.03^{15-20}$$

$$K_T = 0.00086\ \ell/\text{mg-hr}$$

(5) Check soluble effluent BOD_5 (S_e).

$$S_e = \frac{S_o}{1 + K_T X_v t}$$

$$S_e = \frac{165}{1 + (0.00086\ \ell/\text{mg-hr})2254\ \text{mg}/\ell(24\ \text{hr})}$$

$$\underline{S_e = 3.5\ \text{mg}/\ell}$$

S_e calculated is near enough to S_e assumed in step (2). Proceed.

(6) Calculate the oxygen required per day. (For conservative design use QP)

$$O_2(\text{lb/day}) = \left\{\left[a'S_r + N_f(4.57)TKN_i\right] QP(8.34 \times 10^{-6})\right\} + b'X_vV(8.34 \times 10^{-6})$$

$$O_2(\text{lb/day}) = \left\{\left[0.52(160\ \text{mg}/\ell) + 0.6(4.57)30\ \text{mg}/\ell\right] \times \left[6000\ \text{gpd}\ 2(8.34 \times 10^{-6})\right]\right\} + \left[0.15(2254\ \text{mg}/\ell) \times 6000\ \text{gal}\ (8.34 \times 10^{-6})\right]$$

$$\underline{O_2 = 33.5\ \text{lb/day}}$$

(7) Calculate the pounds of oxygen supplied per pounds of BOD removed per day.

$$\text{lb}\ O_2/\text{lb}\ BOD_r = \frac{O_2(\text{lb/day})}{QS_r 8.34 \times 10^{-6}}$$

$$\text{lb}\ O_2/\text{lb}\ BOD_r = \frac{33.5\ \text{lb/day}}{6000\ \text{gpd}\ 160\ \text{mg}/\ell\ 8.34 \times 10^{-6}}$$

$$\underline{\text{lb}\ O_2/\text{lb}\ BOD_r = 4.2}$$

This checks with value in Table 9-1.

(8) Design aeration equipment and adjust standard transfer efficiency (STE) to operating conditions. First adjust the oxygen saturation for the aeration tank C_{s_m} .

$$C_{s_m} = C_s \quad \frac{P_b}{29.4} + \frac{O_t}{42}$$

$$C_{s_m} = 10.15 \text{ mg/}\ell \quad \frac{18.2 \text{ psig}}{29.4} + \frac{18}{42}$$

$$\underline{C_{s_m} = 10.63 \text{ mg/}\ell}$$

$$\text{OTE} = \text{STE} \quad \frac{C_{s_m} \beta p - C_L}{9.17} \quad \alpha(1.024)^{T-20}$$

$$\text{OTE} = 6 \quad \frac{10.63 \text{ mg/}\ell(0.9)1.0 - 2 \text{ mg/}\ell}{9.17} \; 0.9(1.024)^{15-20}$$

$$\underline{\text{OTE} = 4\% \text{ efficiency}}$$

(9) Calculate the required airflow in standard cubic feet per minute (SCFM).

$$\text{SCFM} = \frac{O_2(\text{lb/day}) \times 100}{\text{OTE}(0.0174)1440}$$

$$\text{SCFM} = \frac{33.5(\text{lb/day})(0.9)100}{4\%(0.0174)1440}$$

$$\underline{\text{SCFM} = 30.0 \text{ cu ft/minute}}$$

(10) Calculate SCFM for mixing.

$$\text{SCFM/1000 cu ft} = \frac{\text{SCFM}}{V}\left(7.48 \times 10^3\right)$$

$$\text{SCFM/1000 cu ft} = \frac{30.0 \text{ cfm}}{6000 \text{ gal}}\left(7.48 \times 10^3\right)$$

$$\underline{\text{SCFM/1000 cu ft} = 37.5 \text{ cfm}}$$

This is above the 20-30 scfm necessary to ensure complete mixing and is therefore sufficient.

(11) Calculate the amount of sludge to be wasted from the aeration tank.

$$Q_w = \frac{V}{t_s}$$

$$Q_w = \frac{6000\ \text{gal}}{25\ \text{days}}$$

$$\underline{Q_w = 240\ \text{gpd}}$$

(12) Calculate the sludge recycle ratio.

$$R = \frac{Q_r}{Q} = \frac{\text{MLSS}}{X_r - \text{MLSS}}$$

$$R = \frac{Q_r}{6000\ \text{gpd}} = \frac{1.43X_v}{X_r - 1.43X_v}$$

$$R = \frac{Q_r}{6000\ \text{gpd}} = \frac{1.43(2254\ \text{mg}/\ell)}{8000\ \text{mg}/\ell - 1.43(2254\ \text{mg}/\ell)}$$

$$R = \frac{Q_r}{6000\ \text{gpd}} = 0.67$$

$$R = 67\%\ \text{recycle} \qquad \underline{Q_r = 4020\ \text{gpd}}$$

(13) Calculate the effluent BOD_5.

$$\left(BOD_5\right)_e = S_e + (TSS)_e 0.3$$

$$\left(BOD_5\right)_e = 3.5\ \text{mg}/\ell + 20\ \text{mg}/\ell(0.3)$$

$$\underline{\left(BOD_5\right)_e = 9.4\ \text{mg}/\ell}$$

(14) Design final clarifier.

(a) Calculate surface area assuming an overflow rate (OFR) of 300 gpd/sq ft.

$$SA = \frac{Q\left(1 + \frac{Q_r}{Q}\right)}{\text{OFR}}$$

$$SA = \frac{6000\ \text{gal}\left(1 + \frac{4020}{6000}\right)}{300\ \text{gpd/sq ft}}$$

SA = 33.4 sq ft

(b) Check the solids loading rate (SLR).

$$SLR = \left[\frac{MLSS\left(1+\frac{Q_r}{Q}\right)Q}{SA\ 24}\right](8.34 \times 10^{-6})$$

$$SLR = \left[\frac{1.43X_v\left(1+\frac{Q_r}{Q}\right)Q}{SA\ 24}\right](8.34 \times 10^{-6})$$

$$SLR = \left[\frac{1.43(2254\ mg/\ell)\left(1+\frac{4020}{6000}\right)6000\ gpd}{33.4\ sq\ ft\ 24\ hr/day}\right](8.34 \times 10^{-6})$$

SLR = 0.34 lb/sq/hr

This is slightly lower than the range given in Table 9-1 but it will not produce any problem in the clarifier.

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10. TRICKLING FILTERS

10-1. PROCESS DESCRIPTION

<u>10-1-1. Basic principles.</u> A trickling filter is a bed of relatively large sized rock or crushed stone, or, in some cases, plastic media or redwood slats arranged to provide as great a surface area as possible within the smallest volume. Rock beds are generally limited to 4-10 ft in depth, whereas plastic media filters may be built in towers of 15-25 ft in height because of their lighter weight and greater void space for ventilation, affording considerable space-saving economics. For this reason the following discussion and design procedure is limited to plastic media trickling filters.

Effluent from a primary clarifier is applied over this bed or media in such a manner that it trickles uniformly down through the bed, exposing all surface areas of the media to the waste flow. The plastic media provides a surface upon which millions of microorganisms such as bacteria, protozoa, fungi, and algae may attach themselves to form what is commonly called "zoogleal film." This slime growth, or the zoogleal film, absorbs and utilizes (for energy and cell growth) much of the suspended colloidal and dissolved organic matter from the wastewater as it passes over the growth in a thin film. Part of the absorbed material is utilized by the organisms, as food for production of new cells, while another portion is oxidized to carbon dioxide and water. Partially decomposed organic matter together with excess and dead film is continuously or periodically washed (sloughed) off and passed from the filter with the effluent. These solids are then further removed in the secondary clarifier following the filter.

For the oxidation (decomposition) process to be accomplished, the biological film requires a continuous supply of dissolved oxygen, which may be supplied from the air circulating through the filter voids (spaces between the media). Air may also be added to the trickling filter by installing blower motors in the filter to force air through the media voids. Adequate ventilation of the filter must be provided;

therefore, the voids in the filter media must be kept clean. Clogged voids can create operational problems, including ponding and reduction in overall filter efficiency. Consequently, for maximum efficiency, the slime growths on the filter media should be kept fairly aerobic. This can be accomplished by proper design of the wastewater collection system, proper operation of primary clarifiers, pretreatment of the wastewater by aeration, or addition of recycled filter effluent.

10-1-2. Operational features. The main operational features of a trickling filter include the primary clarifier, the filter media, the underdrain system, the distribution system, and the final clarifier (Figure 10-1).

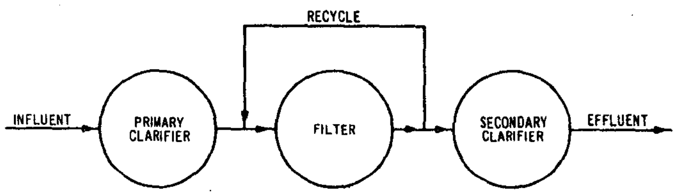

Figure 10-1. Common flow diagram for single-stage high-rate trickling filter.

10-1-2-1. Primary clarifier. Primary clarification of the rest area wastewater, prior to application to the trickling filter media, is desirable to remove the large, readily settleable solids from the wastewater. Primary clarification will also slightly reduce the organic (BOD) loading to the filter.

10-1-2-2. Filter media. As previously mentioned, the filter media provide a large surface area upon which a biological slime growth develops. This slime growth, normally called zoogleal film, contains millions of microorganisms that break down the organic material present in the wastewater, thus achieving the desired treatment. Consequently, loading capabilities of trickling filters are known to be related to the available surface area for biological slime growth.

The comparative physical properties of various trickling filter media are presented in Table 10-1. Two properties which are of interest are speciific surface area and percent void space. As shown in Table 10-1, plastic media have more surface area and more void space than do conventional stone media. Greater surface area permits a larger mass of biological slimes per unit volume, while increased void space allows for higher hydraulic loadings and enhanced oxygen transfer. Consequently, the use of plastic media in trickling filters has extended the range of hydraulic and organic loading well beyond the range of stone media.

<u>10-1-2-3. Underdrain system.</u> The underdrain system normally has a sloping bottom, leading to a central channel, which collects the filter effluent. The underdrain system also supports the media and permits airflow for ventilation.

<u>10-1-2-4. Distribution system.</u> The distribution system normally consists of a rotary distributor having four horizontal pipes supported a few inches above the filter media by a central column. The wastewater is fed from the column through the horizontal pipes and is distributed over the media through orifices located along one side of each of the horizontal pipes. Rotation of the arms is due to the "jetlike" or rotating water sprinkler reaction from wastewater flowing out of the orifices.

<u>10-1-2-5. Final clarifier.</u> The final clarifier is provided following the trickling filter to settle out "sloughings" from the filter media.

10-2. PERFORMANCE

The performance of synthetic media trickling filters can be influenced by several factors including wastewater characteristics, depth, hydraulic and organic loadings, recirculation, and temperature.

The organic matter in settled wastewater is mainly present in either soluble or colloidal forms. Generally, trickling filters are more efficient in removing colloidal material than in removing truly soluble organic matter.

The depth of synthetic media trickling filters may range from

Table 10-1. Comparative Physical Properties of Trickling Filter Media.

Packing	Nominal Size, in.	Units per cu ft	Unit Weight lb/cu ft	Specific Surface Area sq ft/cu ft	Void Space %
Plastic media	20 by 48	2 to 3	2 to 6	25 to 30	94 to 97
Del-Pak redwood	47-1/2 by 47-1/2 by 35-3/4	--	10.3	14	--
Granite	1 to 3	--	90.0	19	46
Granite	4	--	--	13	60
Blast-furnace slag	2 to 3	51	68	20	49

-- Information not available.

15 to 25 ft. Deep filter beds are made possible primarily because of the higher oxygen transfer capability even with the higher hydraulic loadings maintained in the synthetic media filters. Therefore, organic removal in such filters occurs throughout the depth of the filter. Deeper filter beds also allow for nitrification of the wastewater to take place.

Hydraulic and organic loadings are two of the most important parameters affecting the performance of a trickling filter. Because of the superior physical characteristics of synthetic media (more surfaces area and more void area space), such filters are capable of handling hydraulic loadings of 3000 gal/sq ft/day and an organic loading in excess of 150 lb BOD/1000 cu ft per day. However, organic loadings to achieve nitrification in plastic media filters should be limited to 25 lb BOD/1000 cu ft/day.

Recirculation is necessary to provide uniform hydraulic loading as well as to dilute the high strength wastewater. Thus, high recirculation rates will often be necessary in rest-area applications to dilute the high strength wastewaters generated at these facilities. Recirculation as applied to plastic media is normally based on the minimum wetting rates, i.e., a rate of flow per unit area which will induce a biological slime throughout the depth of the media. This minimum wetting rate typically ranges from 0.5 to 1.0 gpm/sq ft, depending on the geometric configuration of the media. Therefore, recirculation in plastic media filters is practiced to maintain the desired wetting rate for the particular media. Generally, increasing the hydraulic loading substantially above the minimum wetting rate decreases the BOD removal through the filter.

Like any other biological waste treatment process, the efficiency of plastic media trickling filters is affected by temperature changes. The effect of temperature (E_T) on filter performance is expressed by the following relationship:

$$E_T = E_{20}\theta^{(T-20)} \qquad (10\text{-}1)$$

where

E_{20} = the efficiency at 20°C

θ = a constant with values between 1.035 and 1.042

T = the temperature in °C

In summary, properly designed and operated plastic media trickling filters can produce effluents which meet the secondary treatment requirements. Nitrification can be achieved in such filters providing that the organic loading is limited to below 25 lb BOD/1000 cu ft/day particularly during the cold winter months.

10-3. RELIABILITY

Except for sensitivity to temperature changes, properly designed and operated plastic media trickling filters can produce effluents that consistently meet the secondary treatment requirements as defined by EPA. Fluctuations in hydraulic and organic loadings can be dampened by adjusting the rate of recirculation. Nitrification can be accomplished by limiting the organic loading below 25 lb BOD/1000 cu ft/day. Being a biological treatment process, plastic media trickling filters cannot handle toxic materials. Sludge production is minimum and the sludge can be easily concentrated, digested, and dewatered.

10-4. FLEXIBILITY

There are many possible flow configurations that may be used in a trickling filter plant. Thus such facilities provide maximum flexiibility in upgrading or expanding the treatment system. Flexibility in operation can also be provided by varying the rate of recirculation to allow for shock loadings or extremely low flows. A common flow diagram associated with trickling filter operation is presented in Figure 10-1.

10-5. OPERATION

Trickling filter is regarded as an ideal unit process for treating relatively small flows. Its popularity stems from its economy and relative simplicity of operation. Compared with activated sludge, controls and startup procedures are much simpler to accomplish, detect, or cure. Consequently, normal operation of such facilities would require much less attention than an activated sludge.

10-6. PRELIMINARY DESIGN

Three separate units are necessary for designing a trickling filter system; primary clarifier, filter, and secondary filter.

10-6-1. Primary clarifier. Primary clarifiers for use before trickling filters may be designed by selecting an overflow rate (OFR) and using the following formulation:

a.

$$SA = \frac{Q}{OFR} \tag{10-2}$$

where

SA = surface area, sq ft

Q = average daily flow, gpd

OFR = overflow rate, gal/sq ft/day (use 800)

b. A detention time is now selected and the volume of the clarifier calculated.

$$V = \frac{Qt}{24} \tag{10-3}$$

where

V = volume of clarifier, cu ft

Q = average daily flow, gpd

t = detention time, hours

24 = conversion factor

c. Calculate the side water depth (SWD) of the clarifier.

$$SWD = \frac{V}{SA7.48} \tag{10-4}$$

where

SWD = side water depth, feet

SA = surface area, sq ft

V = volume, gal

7.48 = conversion factor

10-6-2. Trickling filter. The design of the trickling filter is based on selecting an organic loading rate at which nitrification will take place. Formulation for filter design is as follows.

a. Determine the following parameters

Q = average daily flow, gpd

BOD_5 = 5-day BOD of wastewater

SS = suspended solids in wastewater

lb BOD_5/1000 cu ft = organic loading rate (20 lb BOD/1000 ft^3)

D = depth, 15-25 ft (use 20 ft)

Q_{min} = minimum hydraulic load (use 0.75 gpm/ft^2)

b. Calculate the organic load to the filter assuming a 25% BOD reduction in the primary clarifier.

$$\text{BOD(lb/day)} = \text{BOD(mg/}\ell\text{)}Q(0.75)8.34 \times 10^{-6} \qquad (10\text{-}5)$$

where

BOD(lb/day) = organic load to filter

BOD(mg/ℓ) = BOD in wastewater

Q = average daily flow, gpd

0.75 = 25% reduction in BOD through primary clarifier

8.34×10^{-6} = conversion factor

c. Calculate the volume of the filter media.

$$V_M = \frac{\text{BOD(lb/day) in wastewater}}{\text{BOD(lb/day) organic load rate}} 1000 \qquad (10\text{-}6)$$

where

V_M = volume of media, cu ft

BOD(lb/day) in waste = calculated BOD to filter

BOD(lb/day) rate = assumed organic loading rate (use 20)

d. Calculate surface area (SA) of filter.

$$SA = \frac{V_M}{D} \qquad (10\text{-}7)$$

where

SA = surface area, sq ft

V_M = volume of media, cu ft

D = depth of filter, ft (use 20)

e. Calculate the diameter (d) of the filter.

$$d = \sqrt{\frac{4SA}{\pi}} \qquad (10\text{-}8)$$

where

d = diameter of filter, ft

4 = conversion factor

SA = surface area, sq ft

π = pi

f. Calculate the recycle ratio (Q_r/Q) by calculating the minimum recycle flow (Q_r).

$$Q_r = Q_{min} SA1440 - Q \qquad (10\text{-}9)$$

where

Q_r = recycle flow, gpd

Q_{min} = minimum hydraulic flow (0.75 gpm/sq ft

SA = surface area, sq ft

1440 = conversion factor

Q = average daily flow

10-6-3. Secondary clarifier. The secondary clarifier, to be used after the trickling filter, is designed in the same manner as the primary clarifier. The only difference in the two designs is the overflow rate. The secondary clarifier should be designed with an overflow rate (OFR) of 600 gpd/sq ft.

10-6-4. Design example.

a. Primary clarifier.

(1) Calculate surface area.

$$SA = \frac{Q}{OFR}$$

where

$$SA = \frac{6000 \text{ gpd}}{800 \text{ gpd/sq ft}}$$

Q = 6000 gpd

OFR = 800 gpd/sq ft (assume)

SA = 7.5 sq ft

(2) Calculate volume.

$$V = \frac{Qt}{24}$$

where

$$V = \frac{6000(4)}{24}$$

$Q = 6000$ gpd

$t = 4$ hours (assume)

$\underline{V = 1000 \text{ gal}}$

(3) Calculate side water depth (SWD).

$$SWD = \frac{V}{SA(7.48)}$$

where

$$SWD = \frac{1000}{7.5(7.48)}$$

$V = 1000$ gal

$SA = 7.5$ sq ft

$\underline{SWD = 18 \text{ ft}}$

b. Trickling filter.

(1) Determine the following parameters.

$Q = 6000$ gpd

$BOD_5 = 165$ mg/ℓ

$SS = 190$ mg/ℓ

lb BOD_5/1000 cu ft = 20

$D = 20$ ft

$Q_{min} = 0.75$ gpm/ft^2

(2) Calculate BOD_5 removed by primary clarifier.

$$\text{BOD lb/day} = BOD_5(\text{mg/ℓ})Q(0.75)8.34 \times 10^{-6}$$

where

$\text{BOD lb/day} = 165 \text{ mg/ℓ } 6000 \text{ gpd}(0.75)8.34 \times 10^{-6}$

BOD_5 mg/ℓ = 165 mg/ℓ

$Q = 6000$ gpd

$\underline{\text{BOD lb/day} = 6.2 \text{ lb/day}}$

(3) Calculate volume of filter media (V_M).

$$V_M = \frac{BOD(lb/day)1000}{BOD\ application\ rate}$$

where

$$BOD\ lb/day = 6.2\ lb/day$$

$$BOD\ application\ rate = 20\ lb\ BOD_5/1000\ cu\ ft$$

$$V_M = \frac{6.2(lb/day)1000}{20\ lb\ BOD_5/1000\ cu\ ft}$$

$$\underline{V_M = 310\ cu\ ft}$$

(4) Calculate surface area of filter.

$$SA = \frac{V_M}{D}$$

where

$$SA = \frac{310\ cu\ ft}{20\ ft}$$

V_M = 310 cu ft

D = 20 ft

$\underline{SA = 15.5\ sq\ ft}$

(5) Calculate diameter of filter.

$$d = \sqrt{\frac{4SA}{\pi}}$$

where

$$d = \sqrt{\frac{4(15.5)}{\pi}}$$

SA = 15.5 sq ft

$\underline{d = 4.5\ ft}$

(6) Calculate recycle (Q_r) and recycle ratio $\frac{Q_r}{Q}$.

$$Q_r = Q_{min}SA(1440) - Q$$

where

$$Q_r = 0.75(15.5)1440 - 6000$$

Q_{min} = 0.75 gpm/sq ft

$$SA = 15.5 \text{ sq ft}$$

$$Q = 6000 \text{ gpd}$$

$$\underline{Q_r = 10{,}740 \text{ gpd}}$$

$$\frac{Q_r}{Q} = \frac{10{,}740}{6{,}000}$$

$$\frac{Q_r}{Q} = 1.8$$ <u>Design for 2.0</u>

c. Secondary clarifier.

(1) Calculate surface area.

$$SA = \frac{Q}{OFR}$$

where

$$SA = \frac{6000 \text{ gpd}}{600 \text{ gpd/sq ft}}$$

$$Q = 6000 \text{ gpd}$$

$$OFR = 600 \text{ gpd/sq ft}$$

$$\underline{SA = 10 \text{ sq ft}}$$

(2) Calculate volume.

$$V = \frac{Qt}{24}$$

where

$$V = \frac{6000 \text{ gpd } 4 \text{ hr}}{24 \text{ hr/day}}$$

$$Q = 6000 \text{ gpd}$$

$$t = 4 \text{ hr}$$

$$\underline{V = 1000 \text{ gal}}$$

(3) Calculate side water depth.

$$SWD = \frac{V}{SA 7.48}$$

where

$$SWD = \frac{1000 \text{ gal}}{10 \text{ sq ft } 7.48}$$

$$V = 1000 \text{ gal}$$

$$SA = 1000 \text{ sq ft}$$

$$\underline{SWD = 13.4 \text{ ft}}$$

10-7. ENVIRONMENTAL IMPACTS

The main operational problems which may cause undesirable environmental impacts include clogging and ponding of filter media, fly nuisance in the vicinity of filter, and filter odors.

Clogging and ponding may be corrected by closing the filter media with chlorine at a rate of 0.5 to 1.0 mg/ℓ for several hours a day during periods of low flow. Flies may be controlled by increasing the rate of recirculation to filter to wash fly larvae out of filter; flooding the filter for 24 hr, if possible, to prevent completion of life cycle of flies; applying a low dosage of chlorine; and maintaining grounds so as not to provide sanctuaries for flies.

Filter odors, if developed, can be corrected by aerating or prechlorinating incoming wastewater; clearing underdrain system of all stoppages; increasing recirculation rate to filter to increase dissolved oxygen and to slough off surface slimes; and maintenance of ground around filter.

It should be noted that the problems above could be totally eliminated by proper operation and maintenance of the treatment facility.

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11. ROTATING BIOLOGICAL FILTER

11-1. PROCESS DESCRIPTION

The rotating biological filter is a secondary biological wastewater-treatment system. It is a modification of the trickling filter process which utilizes biological films attached to rotating plastic discs or media. The media, which may be corrugated to increase surface area, are mounted on a horizontal shaft, and rotate slowly in a tank containing wastewater. The discs are normally designed to rotate 2 to 5 rpm and are submerged about 40 percent into the wastewater.

The rotation of the discs aerates the fixed film and provides contact between the biomass and the wastewater. As the wastewater comes in contact with the surface of the film, organic matter is utilized by the biomass as an energy source. Contact of the film of biomass and adhering wastewater with air maintains aerobic conditions in the film and the contents of the tank. Shearing forces exerted on the film as it passes through the waste cause excess biomass to slough from the media into the tank. The sloughed biomass is kept in suspension in the wastewater as it passes through the tank and may be considered to be analogous to the mixed liquor of the activated sludge process.

The rotating biological filter process usually consists of two to four stages in series. Since the first stage receives the highest organic load, it will have the heaviest growth. Film thickness normally varies between 1/8 inch in the first stage to 1/20 inch in the last stage.[1] Each stage operates as a completely mixed, fixed film biological reactor. Treated wastewater and sloughed biomass pass over weirs from stage to stage progressively, undergoing an increased degree of treatment by specific biological cultures in each stage that are adapted to the changing wastewater. The initial stages of media develop relatively heavy cultures of heterotrophic bacteria and fungi. "As the concentration of organic matter decreases in subsequent stages, nitrifying bacteria begin to appear, along with various types of protozoans, rotifers, and other predators."[2]

Excess biomass and treated wastewater leaving the last stage of media pass to a secondary clarifier in which the solids are separated for disposal. The excess sludge normally settles well, reaching a solids concentrations in the clarifier of 2 percent to 4 percent.

Under normal conditions, operation of the process is on a once-through basis. That is, there is usually no recycle of sludge or wastewater as there is in the activated sludge and trickling filter processes. However, at low flows the effluent may be conveniently recycled to keep the media wet and the biomass active.

Flow rates from most rest areas dictate a relatively small treatment system. Biodisc units for flows up to 80,000 gpd are available as factory-built package systems that can be transported to the rest area site ready for installation. Primary treatment for removal of suspended solids prior to entering the biodisc tank is recommended. Package systems often use septic tanks or Imhoff cones for this purpose. These anaerobic systems provide for storage and digestion of both primary solids and sludge removed in the secondary clarifier. For smaller systems, flow equalization is usually provided. A configuration of a biodisc package unit manufactured by Autotrol Corporation is given in Figure 11-1.

11-2. PERFORMANCE

Currently, no data are available on the performance of a rotating biological filter at a rest area. However, fixed film processes such as these are relatively well suited to the varying hydraulic loads encountered at rest areas. Antoine[3] reported that hydraulic shock loads resulting in liquid detention times as low as 3 min (usually 1 to 3 hr) did not result in measurable stripping of biomass from the discs. Although high hydraulic surges will result in temporary reduction of efficiency, recovery is relatively rapid, since the biomass remains on the discs.

Rest areas are subject to periods of very low or no flow. During extended periods of low flow, film thickness on the biodisc will decrease somewhat. However, treatment efficiency would be expected to be greater than usual during these periods. Efficiency would then fall for

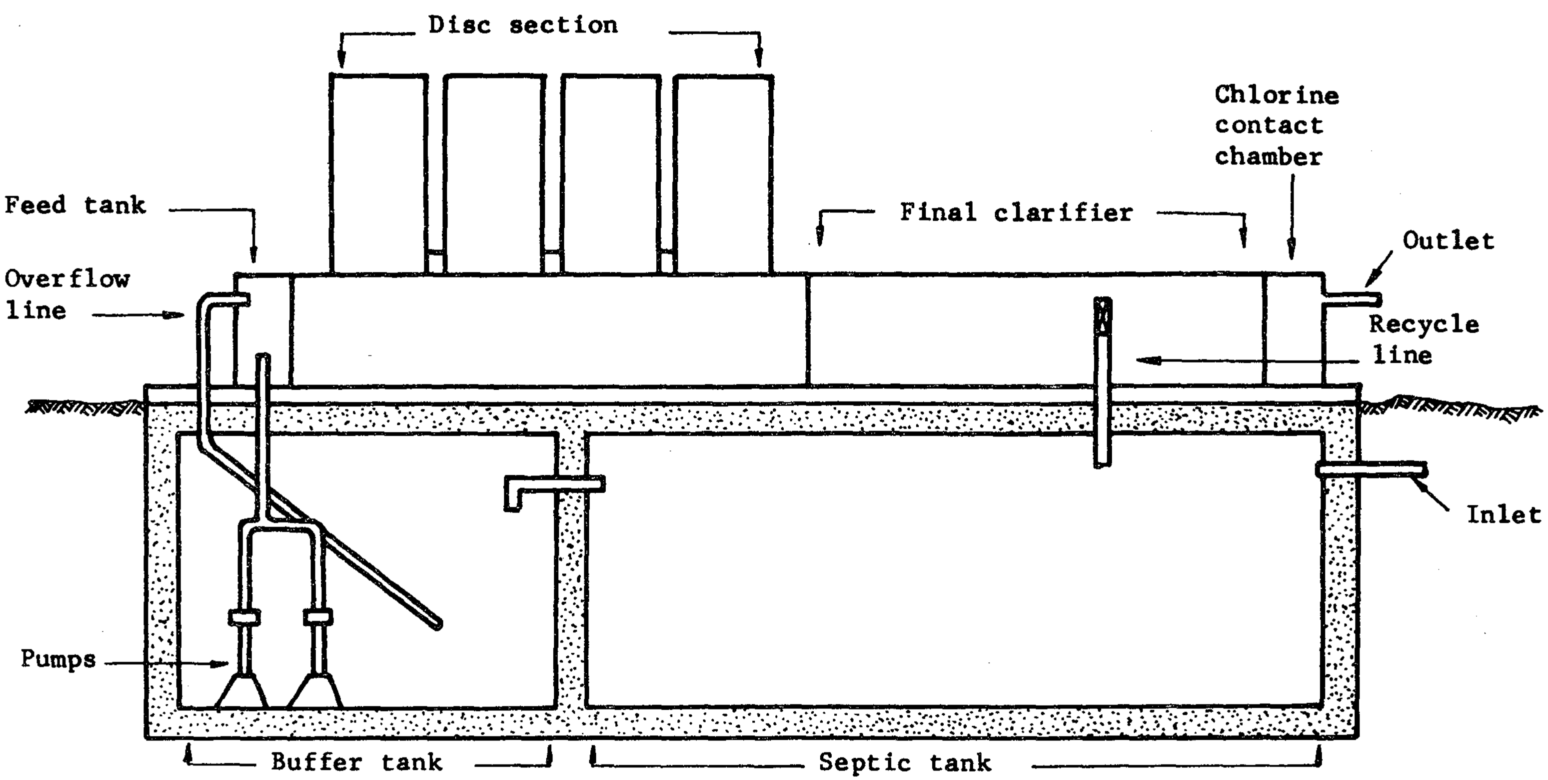

Figure 11-1. Biodisc package treatment unit.

a short time when high flows resume until the film thickness reaches its normal value. One study[4] of biodisc performance at a summer camp showed very low flows for 2 days on weekends followed by normal flows on weekdays. No observable effect on performance was noted due to the low weekend flows. At rest areas where no flow occurs for extended periods, effluent can be conveniently recycled through the system to provide some organic matter to maintain biological activity on the discs.

A staged or plug-flow type of process such as the rotating biological filter is well suited to the removal of ammonia nitrogen where this is required. Nitrifying bacteria, which oxidize ammonia to nitrate nitrogen are grown slower than the heterotrophs which remove BOD. Hence, the nitrifying organisms can successfully compete only when BOD is reduced to a low value such as that which occurs in the latter stages of the biodisc process. Studies[2,5,6] have shown that ammonia nitrogen removal begins when the BOD falls below 14-30 mg/ℓ and that high degrees of nitrification can be achieved as the BOD falls below 10 mg/ℓ when treating municipal sewage.[2] Since rest area waste has ammonia nitrogen values higher than municipal waste, significantly higher populations of nitrifiers would be expected to establish themselves on the discs. However, it is expected that the hydraulic loading rate will have to be kept low (less than 1 gpd/ft^2).

Wastewater temperature affects rotating biological filter performance just as it does all biological treatment processes. Temperatures as low as 55°F have essentially no effect on performance. However, below 55°F, both BOD removal and nitrification will decrease somewhat. Installations in southern climates do not need to be covered except for aesthetic reasons or for protection against vandalism. Wind and rain will have no significant effect on the media or biological film.

Enclosures for biodisc plants may be constructed of any suitable corrosion-resistant material. No heating or forced ventilation is necessary. One company has developed a molded plastic cover with thermal insulation. The cover generally follows the outline of the biodiscs and minimizes the area to be covered. Covers also provide protection against accumulation of leaves in the tanks and can enhance the aesthetics of the treatment plant.

11-3. FLEXIBILITY

A biodisc plant can be expanded to handle increased wastewater flows by adding additional stages to the unit. Two to four stages are usually constructed initially. Since BOD removal is primarily a function of surface area, the increased surface area of subsequent stages could handle additional BOD loadings. However, the surface area of the clarifier would also have to be increased to accommodate an increase in hydraulic loadings. Biodisc systems used at rest areas would be package-type systems. Significant modifications would be required to an existing package unit in order to provide additional surface area for both the biological disc and the clarifier. It is anticipated that the most economical solution would be to purchase a second package unit to be operated in parallel with the existing unit.

Upgrading a biodisc to meet more stringent effluent limitations may be accomplished with less difficulty than with most other systems. Nitrification can be achieved by adding stages to a biodisc system, as well as to other biological systems. If the additional stages are added after the clarification step of a secondary treatment process, then further clarification may not be required because of the low sludge production rate of the nitrifying organisms.[2] Denitrification units are available for removal of total nitrogen. These units consist of submerged biodiscs in an anaerobic environment with the addition of a supplemental carbon source, usually methanol, for the heterotrophic denitrifying bacteria. Since the biological growth is attached to the discs, carbonaceous BOD removal, nitrification, and denitrification can occur in these successive stages with a clarifier only at the end of the process.

Phosphorus removal requires chemical addition as with other biological processes. Since primary settling is usually provided, phosphorus removal may be achieved in this tank.

Higher overflow rates in the clarifier may be possible if mircoscreening or filtration is provided for removal of suspended solids. This would allow expansion of the biodisc surface area without requiring additional clarification.

11-4. RELIABILITY

The reliability of the biodisc process in a rest area is difficult to assess since no operational data are available. However, use of biodisc is increasing in the United States and has been widely used in Europe. The biodisc system should provide an effluent of consistent quality because of the equalized flow conditions usually provided with biodisc units.

11-5. OPERATION

The major cause of unsatisfactory effluents from small package plants is poor operation. It is reasonable to assume that processes requiring a minimum of operator attention will be the most likely to produce consistently acceptable effluents. Operation of the biodisc unit consists of oiling motors and greasing chains. It is not necessary to adjust the mixed liquor suspended solids or run sludge-settling tests to determine when to waste excess solids as required in the extended aeration process. Foaming does not occur with the biodisc, so there is no need for foam suppression by chemicals or sprays.

Operator time and the need for operator skill are minimized. One study of a biodisc serving a recreational camp (flow = 4500 gpd) showed that only 14 hr of operator attention were required for an 11-week season. Of the 14 hr, 8 hr were spent in preparing chlorine solutions for disinfection of the effluent, and 6 hr were used for repair and maintenance.[4]

The sludge produced by the biodisc is reported to be more dense than activated sludge and have better settling characteristics. However, it is reasonable to expect that the problem of denitrification accompanied by rising sludge could be a problem in biodisc clarifiers where nitrification has occurred. Since the package biodisc units normally are not equipped with skimmers for floating scum, this may require operator attention.

11-6. PRELIMINARY DESIGN

The design procedure for selection of a rotating biodisc package unit as shown in Figure 11-1 and manufactured by Autotrol Corporation is given below. The design concept for other manufacturers will be similar, but plant configuration may differ.

a. Inputs required.

Q = average daily flow, gpd

S_o = influent BOD_5, mg/ℓ

T_w = wastewater temperature during winter months

b. Design calculations.

(1) Determine primary treatment requirements. Autotrol recommends a minimum 12-hr retention time in a septic tank for primary treatment. This will reportedly provide at least a 1-yr storage capacity before sludge removal is required.[2] Volume of the septic tank is determined by

$$V_{st} = Qt_{st} \tag{11-1}$$

where

V_{st} = septic tank volume, gal

t_{st} = septic tank detention time, days

(2) Determine flow equalization requirements. Table 11-1 describes Autotrol general guidelines for determining the size of the equalization basin. Volume of the equalization tank is given by

$$V_{et} = Q(f)(1 \text{ day}) \tag{11-2}$$

where

V_{et} = volume of equalization tank, gal

f = fraction of daily flow from Table 11-1 for number of hours that flow is less than 0.25Q

1 day = one day of flow

Q = average daily flow, gal

Table 11-1. Flow Equalization for Biodisc System.[2]

DAILY PERIOD OF WASTEWATER FLOW Less than 25 Percent of Average Flow hr	FLOW EQUALIZATION TANK CAPACITY Daily Flow, %
0	0
4	10
6	15
8	25
12	33
14	50
16	60
18 or more	67

It should be noted that when biodiscs are to be used at rest areas the capacity of the flow equalization tank must be, as a minimum, 33% of the daily flow (WASTE 24). Another method of designing an equalization tank has been presented previously; this method may be used by the design engineer if he so desires.

(3) Determine biodisc surface area using either Figure 11-2 or 11-3. Figure 11-2 gives hydraulic loading as a function of retention time in the septic tank, whereas Figure 11-3 relates hydraulic loading to BOD_5 or primary (septic tank) effluent. By choosing a desired percentage of BOD_5 removal and knowing the effect of primary treatment on the raw wastewater, the designer can determine hydraulic loading in gpd/ft^2. Correct hydraulic loading for temperature using Figure 11-4. Biodisc surface area A, in sq ft, is then given by

$$A = \frac{Q}{\text{hydraulic loading}} \qquad (11\text{-}3)$$

(4) Design secondary clarifier using Ten States' Standards. For small systems, Ten States' Standards specifies an overflow rate of 300 gpd/ft^2 and a detention time of 4 hours. Clarifier surface area (A_c, in sq ft) is then given by

$$A_c = \frac{Q}{300\ \text{gpd/ft}^2} \qquad (11\text{-}4)$$

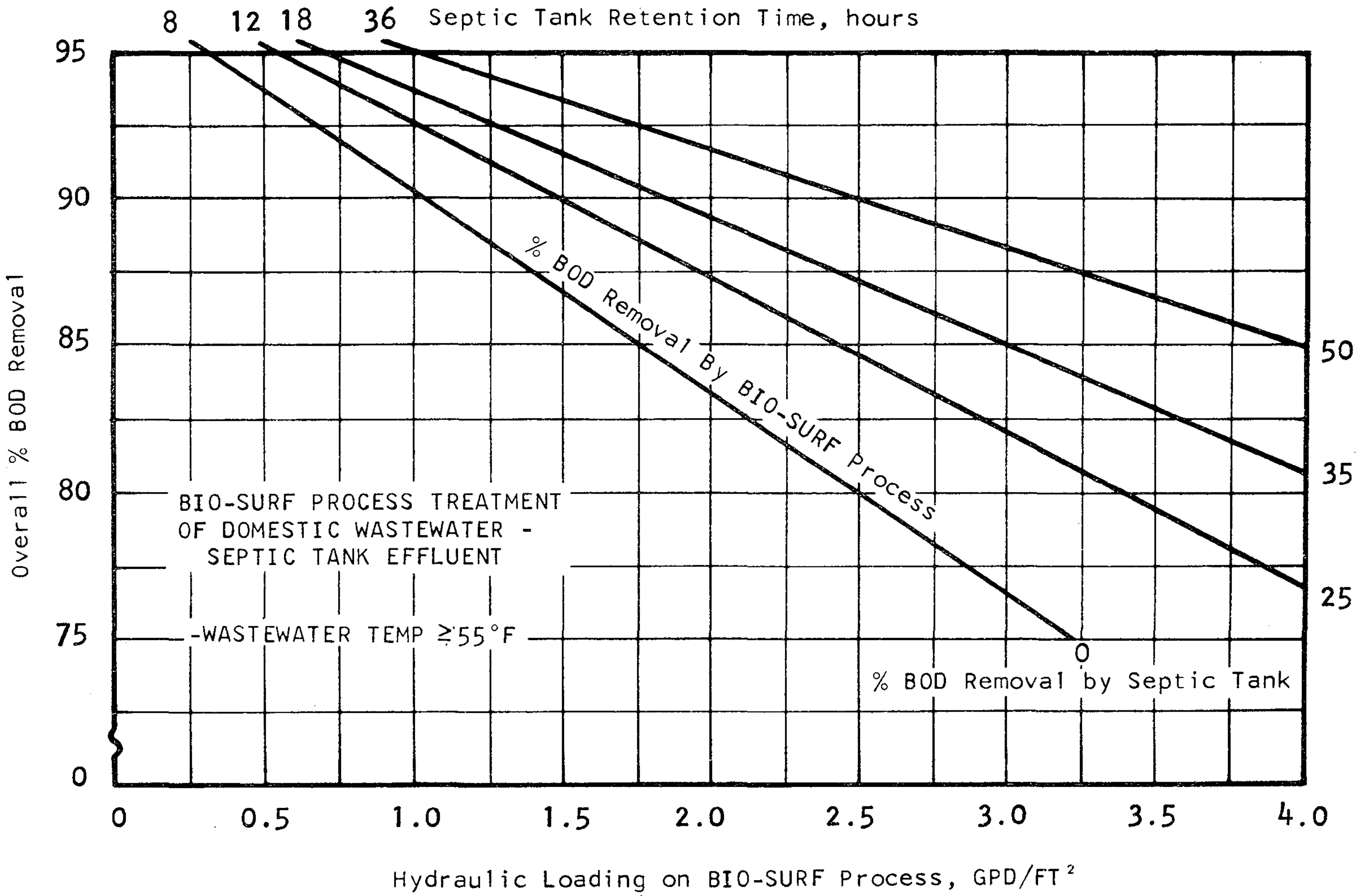

Figure 11-2. Biosurf process treatment of domestic wastewater--septic tank effluent.

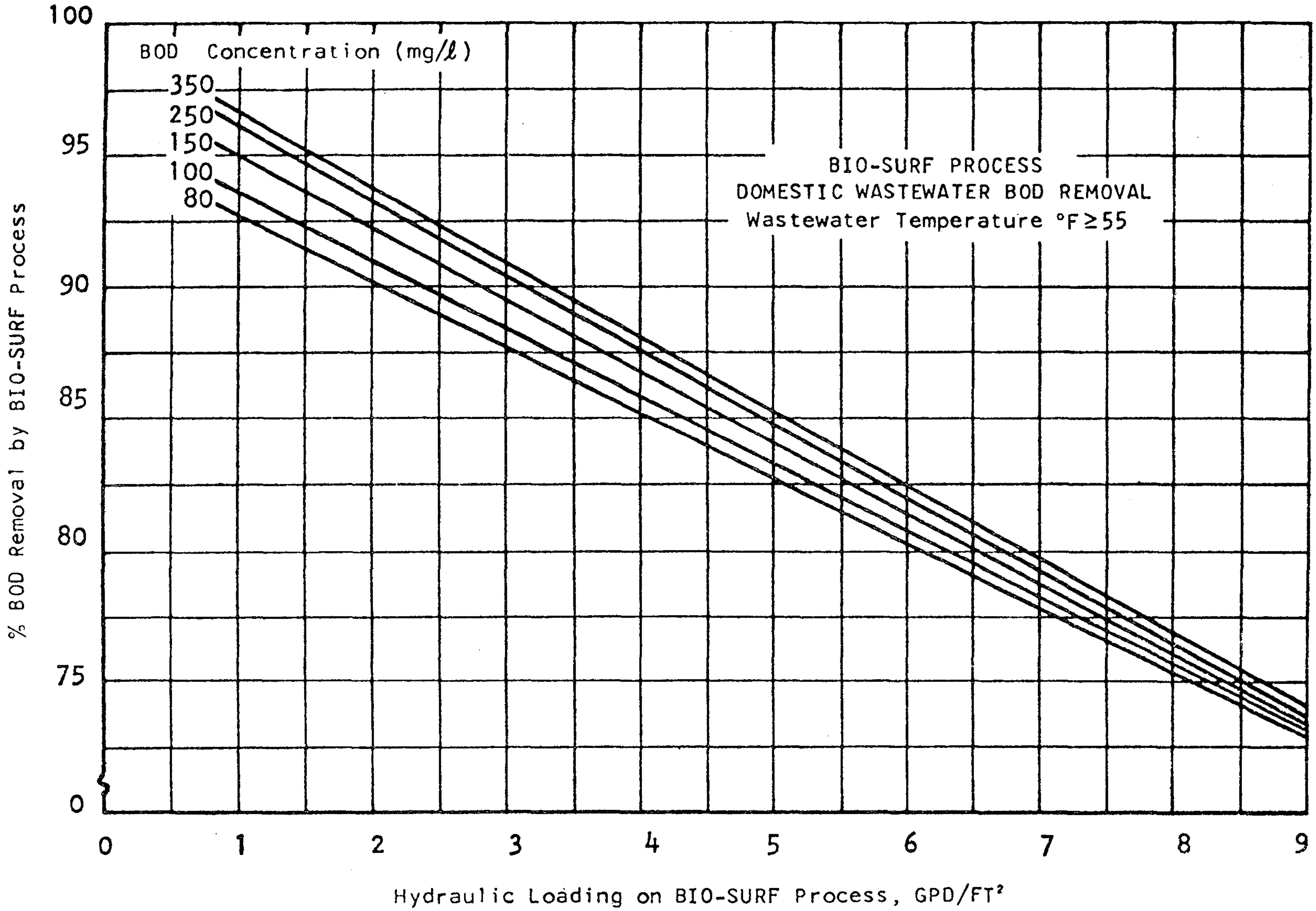

Figure 11-3. Biosurf process treatment of domestic wastewater--BOD removal for biomodule only.

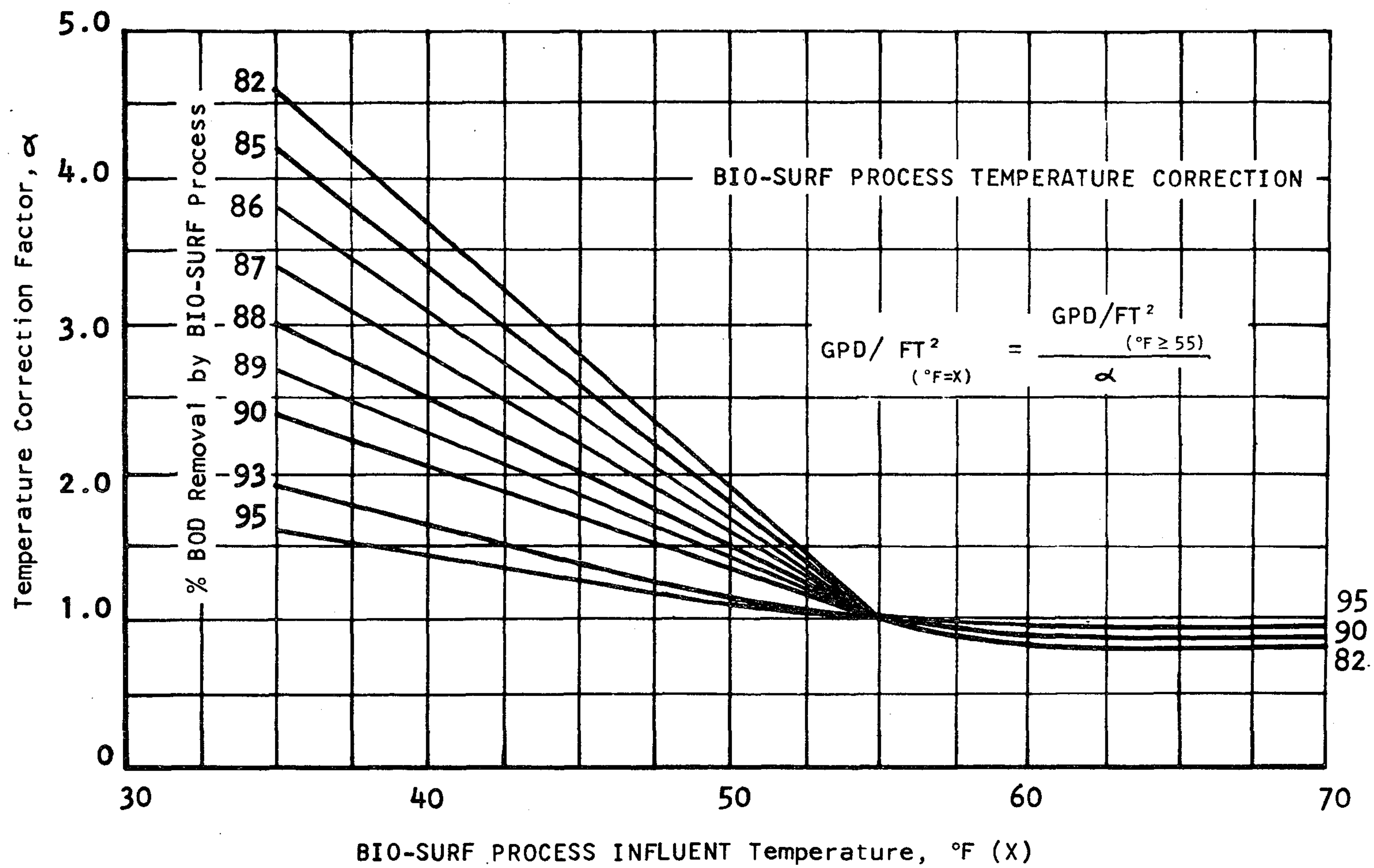

Figure 11-4. Biosurf process temperature correction.

Higher overflow rates may be used where filtration or microscreening are added to polish the effluent.

c. Design example.

$Q = 6000$ gpd

$S_o = 165$ mg/ℓ

$T_w = 42°F\ (6°C)$

(1) Septic tank volume

$V_{st} = Qt_{st}$

$t_{st} = 12$ hr

$V_{st} = (6000\ \text{gpd})(12/24\ \text{day})$

$V_{st} = 3000$ gal

(2) Flow equalization volume.

$V_{et} = Q(f)(1\ \text{day})$

Assume that during 12 hr of the day the flow is less than 0.25Q. From Table 11-1, f = 0.33 .

$V_{et} = (6000\ \text{gpd})(0.33)(1\ \text{day})$

$V_{et} = 2000$ gal

(3) Biodisc surface area.

Assume that 90 percent BOD_5 removal is required for conservative design. The allowable hydraulic loading is then determined from Figure 11-2. Figure 11-2 is used by entering the top of the figure with the appropriate septic tank retention time in hours (12 in this example) and by entering the left-hand side of the figure with the appropriate percent BOD removal (90% in this case). The septic tank retention time line is followed diagonally and the percent BOD line followed horizontally until the two lines intersect. A line is dropped vertically from this point to the hydraulic loading line at the bottom of the chart. The value of intersection on this line is the design hydraulic loading in gpd/ft^2 (in this example the value is 1.5 gpd/ft^2).

Hydraulic loading may also be determined using Figure 11-3 and 11-4. Again assume that a 90 percent reduction is desired. As seen in the work of Sylvester and Seabloom and shown in the section on septic tanks-adsorption fields of this report, the average BOD effluent from a septic tank will be 133 mg/ℓ.

Table 11-3 is used first. Whereas a total BOD reduction of 90 percent is desired the final effluent from the plant must be 16.5 mg/ℓ (165 mg/ℓ - 0.9 × 165 mg/ℓ = 16.5 mg/ℓ). Since the effluent from the septic tank entering the rotating biological filter is 133 mg/ℓ the percent of BOD removal required by the biodiscs is 87.5 percent $\left(\frac{135 \text{ mg/}\ell - 16.5 \text{ mg/}\ell \times 100}{133 \text{ mg/}\ell}\right)$
Figure 11-3 is used by entering the left-hand side of of the figure with the desired percent removal of BOD by the filter (in this case 87.5 percent) and entering the diagonal lines with influent BOD concentration (in this case 133 mg/ℓ). From the point where these two lines intersect a line is dropped vertically to the bottom of the figure. The point of intersection on this line is then read as the hydraulic loading (in this case 3.6 gpd/ft^2). Figure 11-4 is now used to correct this loading for temperature effects.

Figure 11-4 is used by entering the bottom of graph with the influent temperature (in this case 42°F). A vertical line is drawn from this point to where it intersects the diagonal line corresponding to the percent BOD removal obtained in the biodiscs (in this case 88 percent). A line is now drawn horizontally from this point of intersection to the left-hand side of the figure. The resultant temperature correction factor, α , is then read from the scale (in this case $\alpha = 2.3$).

The hydraulic loading rate obtained from Figure 11-3 is now adjusted by the temperature correction factor obtained from Figure 11-4).

$$\text{Design hydraulic loading} = \frac{\text{Hydraulic loading from Figure 11-3}}{\text{Temperature correction factor from Figure 11-4}}$$

$$\text{Design hydraulic loading} = \frac{3.6 \text{ gpd/ft}^2}{2.3}$$

$$\text{Design hydraulic loading} \simeq 1.6 \text{ gpd/ft}^2$$

Since both the hydraulic loading obtained from Figure 11-2 and that obtained from Figures 11-3 and 11-4 agree this value will be used. If the values had differed then the lower hydraulic loading rate would be used for design.

$$A = \frac{Q}{\text{hydraulic loading}}$$

$$A = \frac{6000 \text{ gpd}}{1.5 \text{ gpd/ft}^2}$$

$$A = 4000 \text{ ft}^2$$

(4) Clarifier surface area.

$$A_c = \frac{6000 \text{ gpd}}{300 \text{ gpd/ft}^2}$$

$$A_c = 20 \text{ ft}^2$$

11-7. REFERENCES

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12. LAND TREATMENT

12-1. PROCESS DESCRIPTION

The objective of land treatment of wastewater should be aimed at treatment of the water with adequate consideration given to the impacts on the environment. Land application of wastewater effluents has been categorically divided into three major processes, each having specific objectives and requirements for site characteristics. These three processes are slow infiltration, overland flow, and rapid infiltration. The basic application methods are shown schematically in Figure 12-1. Although the methods of applying the effluents are distinctly different for each process, the primary goal of land treatment is the treatment of wastewater in a cost-effective, socially and environmentally acceptable manner to produce high quality renovated waste. While all three land treatment processes are discussed design procedure is given only for spray irrigation.

12-1-1. Spray irrigation. This system is also termed "slow infiltration" and may have two basic objectives, depending upon the volumetric loading of effluent to the soil system. The intended purpose when maximizing application rates for wastewater renovation by spray irrigation systems is to exhaust the plant and soil matrix's ability to renovate the wastewater constituents, thereby providing maximum treatment of effluent.

A secondary benefit of this method is realized in the production of high-yield crops. In this process the selection of crops that are capable of maximizing nutrients and other constituent uptakes is essential. In addition, it is necessary to select crops that will respond to the dilute nutrients in wastewater, otherwise amendments such as commercial fertilizer may be required to produce the desired crop growth.

12-1-2. Overland flow. The overland flow method is the treatment of wastewater on soils of rather low infiltration capacity. It is intended to maximize waste treatment as the wastewater passes over the soil surface and through the vegetative grass cover. A cover crop is

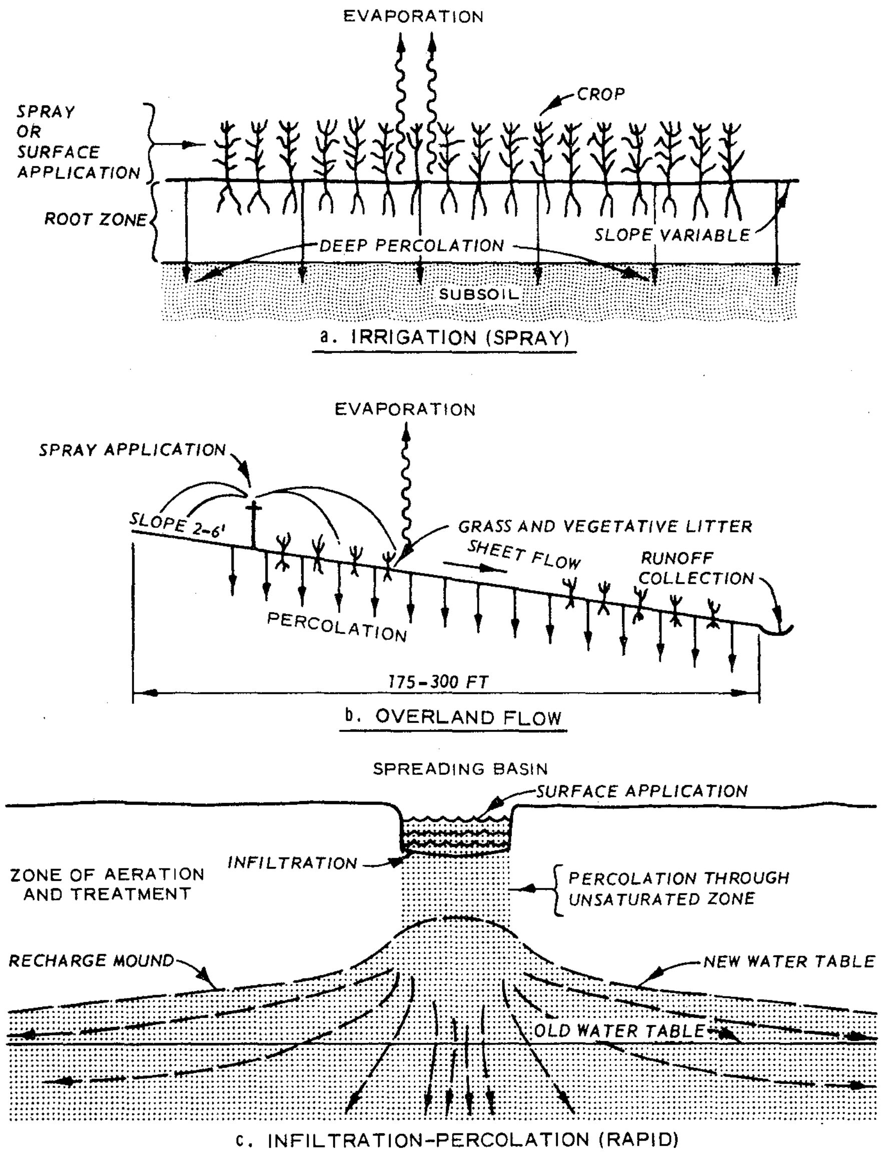


Figure 12-1. Land application methods.

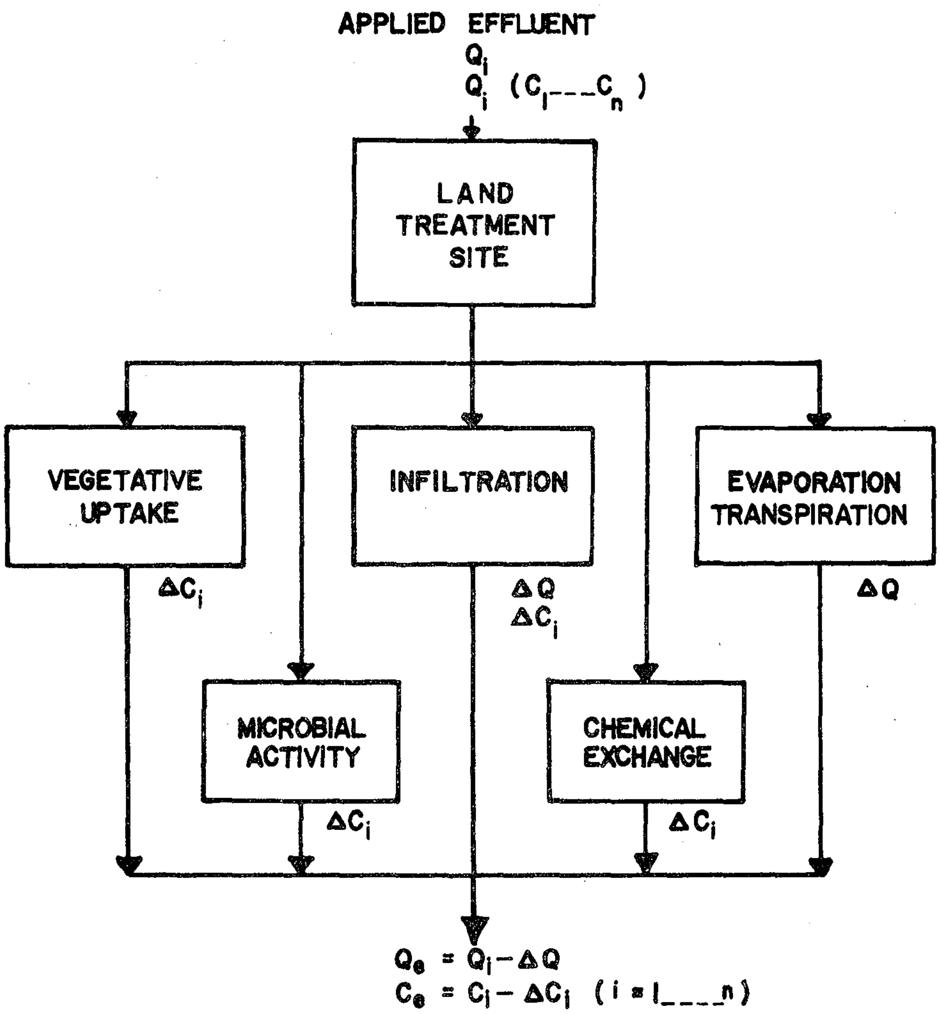
an integral part of an overland flow system, since the plants remove substantial amounts of nitrogen and phosphorus in the form of plant protein and also prevent erosion. One of the principal advantages of an overland flow site is that renovated water can easily be monitored and sampled as it leaves the site. This provides for a high degree of confidence when assessing the performance of the ongoing treatment process.

12-1-3. Rapid infiltration. The primary objective of this process is two-fold: treatment of wastewater and groundwater recharge. Large quantities of water pass through a highly permeable soil mantle with periodic resting to allow the soil profile to dry out and restore the soil's infiltration and treatment capacity. A cover crop may be grown to maintain adequate surface infiltration rates that could otherwise be reduced by solids.

12-1-4. Basic model of all land treatment methods. As presently conceived, all three land applicable processes previously discussed can be represented as having five principle mechanisms for treatment and utilization of wastewater and its constituents. These five mechanisms are listed below and shown graphically in Figure 12-2.

1. Vegetative uptake
2. Infiltration
3. Evaporation/transpiration
4. Microbial activity
5. Chemical exchange

The major differences between the three processes are exemplified in the methods of applying wastewater effluent and predominant mechanisms of treatment and utilization. For example, the process of crop irrigation is principally concerned with maximizing crop production. For this reason the predominant mechanisms involved are vegetative uptake and evaporation/transpiration. Similarly, spray irrigation maximizes vegetative uptake, and to a lesser degree utilizes evaporation/transpiration; however, it also depends heavily on the mechanisms of infiltration, microbial activity, and chemical exchange. Rapid infiltration implies that the soil profile is highly permeable, resulting in



LEGEND

Q = Flow
C_i = Constituent concentration
Δ = Incremental reduction
Q_e = Effluent flow
C_e = Effluent constituent concentration

Figure 12-2. Principal mechanisms of land-application methods.

a rapid downward percolation to the water table. This process is almost entirely dependent on the infiltration mechanism, with microbial activity and chemical exchange being major supportive functions. Unlike rapid infiltration, overland flow has little dependency on infiltration, since the process involves the phenomenon of surface treatment. It does utilize microbial activity and vegetative uptake in their fullest capacity with some emphasis on evaporation/transpiration and chemical exchange.

The benefits that can be derived from properly designed land treatment systems include recycling of nutrients, organic matter, and water back to the land; conservation of natural resources; drainage of agricultural fields; and harmony with physical, social, economic, and environmental makeup of region.

Many factors influence the selection of a particular process including wastewater quality, climate, topography, soil type, geology, land availability, and return flow quality.

The basic objectives and unique characteristics of each process are given in Table 12-1.

12-1-5. Summary. General guidelines and criteria for the land treatment of liquid waste are available from the Environmental Protection Agency[2-4] and from the Agricultural Research Service. Covered in these reports are methods for evaluating land treatment systems, guidelines for sizing land treatment systems, methods of estimating cost of land treatment systems, and general guides for selecting treatment type and location. Of particular note is the fact that none of these publications give specific methodology for sizing and constructing land treatment systems. This is due to the variance among land treatment sites in regard to soil characteristics, ground cover, depth to groundwater, prevailing atmospheric conditions, and variability in wastewaters to be treated. Rest areas are no exception to the rule; however, an attempt to generalize a methodology and guidelines for design of land treatment systems for rest areas will be presented below.

Table 12-1. Land Application Processes for Treatment of Municipal Wastewater.[1]

Treatment Method	Loading Acre ft/acre/yr	Net Irrigated Land Area Requirement for 1-mgd Flow	Objective	Suitable Soils	Dispersal of Applied Water	Impact on Quality of Applied Water
Overland	5 to 25	45 to 225 acres plus buffer areas, etc.	Maximize water treatment. Crop is incidental	Slow permeability and/or high water table	Most to surface runoff; some to evapotranspiration and groundwater	BOD and SS greatly reduced. Nutrients reduced by fixation and crop growth
Crop	<1 to >5	<225 to >1000 acres plus buffer areas, etc.	Maximize agricultural production	Almost all soils suitable for irrigated agriculture	Most to evapotranspiration; some to groundwater; little or no runoff	BOD and SS removed. Most nutrients consumed in crop or fixed. TDS greatly increased
Slow	1 to >10	<110 to 1100 acres plus buffer areas, etc.	Maximize water treatment by percolation through the soil-crop system with crop production as a side benefit	More permeable soils suitable for irrigated agriculture; may use soils marginal because of coarse texture	Evapotranspiration and groundwater; little or no runoff	BOD and SS mostly removed. Nutrients removed in crop or fixed. TDS increased by evapotranspiration and leaching
Infiltration-percolation	11 to 500	2 to 100 acres plus buffer areas, etc.	Recharge filter water or groundwater; crop may be grown with little benefit	Highly permeable sands and gravels	Most to groundwater; some evapotranspiration; runoff	BOD and SS reduced. Some nutrient removal by soil. TDS may increase because of leaching

12-2. PERFORMANCE

To predict removal efficiencies for various wastewater constituents is extremely difficult because treatment efficiencies are highly site specific. The degree to which certain constituents are removed depends on factors such as loading rates, soils, crops, climate, design, construction, operation, and maintenance.

Anticipated removal efficiencies of well designed and properly operated land application systems are listed in Table 12-2. The values

Table 12-2. Anticipated Removal Efficiencies for Well-Designed and Properly Operated Land Treatment Systems.[4]

	Removal Efficiency, %, 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+

shown were derived from existing systems and are presented for the purpose of illustrating that land-application systems, when properly designed and operated, are capable of producing effluents comparable to other advanced wastewater-treatment (AWT) methods.

The filtering mechanism of the soils for spray irrigation and rapid infiltration provided for excellent removals of biochemical oxygen demand (BOD) and suspended solids (SS) as shown in Table 12-2. Overland flow is expected to provide higher residuals of SS in the runoff water partly due to the horizontal filtering mechanism of the organic mat not being as effective in trapping the suspended solids as the soil matrix.

The removal of nitrogen and phosphorus in land treatment is attributed to microbial activity, plant uptake, and chemical reaction in the soil.

When properly managed, overland flow and spray irrigation systems can provide a high percentage of nitrogen removal. Rapid infiltration is less effective in removing nitrogen than overland flow; however, it can effect significant reduction in phosphorus depending on the soils exchange and fixation capacity.

Increases in nitrogen renovation may be accomplished through the addition of organic carbon. A high carbon to nitrogen ratio minimizes nitrogen loss through leaching by accumulating nitrogen in the soil.

Spray irrigation is the most effective system for heavy metal removal due to ion exchange and absorption within the soil column; rapid infiltration provides fair removals of metals (50%-95%), since reaction time with the soil is not as great as that with soil infiltration systems. Overland flow does not call for percolation into the soil and contact with the soil exchange mechanisms; therefore, the degree of heavy metal renovation is limited. However, there is evidence that the surface organic matter and vegetation may provide significant removal of heavy metals.[5,6]

Microorganisms are effectively removed by land treatment systems. Spray irrigation and rapid infiltration systems are very efficient for bacteria removal in the upper few feet of the soil column. There is a greater possibility for bacteria and viruses to be present in surface runoff water from the horizontal flow in the overland flow system than in the vertical percolation of wastewater effluent through the soil.

In summary, to accurately predict the performance of a particular land treatment system the designer can elect to perform a pilot study or review the literature (such as references 1-9) in search of a similar system, based on wastewater characteristics, site characteristics, and design features. Obviously, pilot work can be very expensive and time consuming, whereas a literature review requires the designer to exercise <u>sound engineering judgement</u> when final selections of treatment efficiences are made. Each removal mechanism (Figure 12-2) must be investigated and the expected removals estimated. The degree of detail expected in deriving these estimates will depend on the fate of the constituent in the environment and the water quality requirements for the treated water.

12-3. FLEXIBILITY

Once constructed, land treatment systems do not readily lend themselves to expansion. Future expansion of a land treatment site may be accomplished through purchase of additional acreage, process modification attained through operating experience, and technological advances made in preapplication treatment processes.

12-4. RELIABILITY

Because land treatment is a factor of mechanical application and the land is the treatment medium, reliability will be a function of operational ability and experience and mechanical reliability. Standby duplicate mechanical equipment will insure continued application, and continued operational experience will insure reliability of the land as a treatment medium.

12-5. OPERATION

Operation of a land treatment facility may be accomplished through the pumping of treated wastewater onto the site. Generally, the wastewater will be applied over a short period of time (1 to 8 hr) and the site allowed to dry out between applications. Application should take place during periods of little or no rainfail only and when the prevailing temperature is above 32°F (0°C) so that the soil will not become saturated and excessive runoff will not occur. In most instances, particularly in the northern latitudes, application cannot take place during the winter due to freezing conditions. During the winter all wastewater should be stored in a lagoon and the land treatment site should not be operated.

Operating of a land treatment system will also entail periodic removal of crops growing on the site. Crop removal may be necessary only during the growing season, and the application site can remain undisturbed for the remainder of the year. Many types of crops (commercial and noncommercial) can be utilized on a land treatment site; hence the frequency of crop removal will be site specific and will vary yearly with prevailing climatic conditions.

All mechanical equipment should be checked periodically to insure proper operation. Any equipment malfunctions should be corrected at the earliest possible time and any worn or damaged parts replaced.

If underdrains, collection ditches, or monitoring wells are installed at the land treatment site, samples of renovated wastewater should be periodically collected and analyzed to insure proper operation of the treatment system. Early detection of insufficiently treated wastewater will allow for process modification by changing the frequency and duration of wastewater application.

Proper operational procedures will thus consist of inspection of all mechanical equipment to insure proper functioning, replacement or repair of improperly operating mechanical equipment, possible collection and analysis of renovated wastewater samples, periodic cutting and removal of cover crops (mowing the grass), and activation and deactivation of application equipment.

12-6. PRELIMINARY DESIGN

Before detailed design work is accomplished on a land treatment system certain factors affecting the type and location of the land treatment system must be investigated. Among these factors are the quality of the wastewater to be applied, the prevailing climate (with particular emphasis on periods of freezing temperatures and excessive rainfall), the underlying geological formations and types of soil on the potential treatment sites, the depth to groundwater, proximity to other facilities (houses, roads, etc.) and present vegetative cover, and topography of the potential sites.

Spray irrigation land treatment systems may be designed using the following procedure:

a. Inputs required.

Q = wastewater flow, gpd

C_i = constituent concentrations

Various constituents must be investigated with particular emphasis on BOD_5, SS, nitrogen, phosphorus, and various heavy metals. Geology of the potential sites must be determined from geologic maps and on-site soil borings. Water table depth and movement must be determined through on-site borings. Soil characteristics must be determined through available soil maps and on-site borings.

Topography of the potential site must be determined through use of topographic maps and site visitation. Ground cover of the potential sites may be determined from vegetative maps and site investigations.

Prevailing climatic conditions with emphasis on periods of freezing conditions and excessive rainfall must be determined through investigation of records maintained by the local U. S. Weather Bureau.

b. Design criteria. Design criteria for land treatment systems are site specific, depending on local climate, soils, and geology. However, a general range of design criteria (Table 12-3) may be used in selecting the type of land treatment to be used and for determining the size of the system to be installed.

Criteria currently employed in the design of land treatment systems are hydraulic loading (in./wk and ft/yr) and nutrient loading (lb/acre/yr).

c. Design procedure.

(1) Collect information on potential land treatment sites. Information needs and sources are listed in Table 12-4. In particular, climatic data should be obtained from the U. S. Weather Bureau and a table constructed such as in Table 12-5 which was constructed for a northern state. Also soil characteristics at the rest area may be determined by contacting the local Soil Conservation Service agent. Wastewater quantity and constituent concentration should be determined at this time. The wastewater to be applied in land treatment is the effluent from some previous treatment process such as extended aeration activated sludge or lagoon.

(2) Calculate the quantity of wastewater to be applied per year.

$$Q_{year} = Q \times \text{days/year} \qquad (12\text{-}1)$$

Table 12-3. Comparative Characteristics of Land-Application Approaches.[2]

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 soils 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	Undetermined	About 15 ft
Wastewater losses	Predominantly evaporation or deep percolation	Predominantly surface discharge but some evaporation and percolation	Percolation to groundwater

Table 12-4. Information Needs and Sources for Land Application of Wastewater.

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 and/or Public Health	Historical Society	State Geological Survey	State Div. or Dept. of Water Resources	State University Extension Specialists	State Planning Office	Corps of Engineers	U. S. Forest Service	U. S. Geological Survey	USDA, Agricultural Research 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 <5 ft										●												
Soil properties (chemical and physical)	●					●				●					●							
Agricultural land use capability	●	●				●	●			●												
Depth to bedrock										●			●						●			
Unconsolidated materials													●						●			
Bedrock type and structural characteristics													●						●			
Jointing and permeability of rock	●												●	●					●			
Rock outcrops										●			●						●			
Surface slope, categories (ex. 0-3 pct)										●									●			
Floodplain, flood hazard	●	●					●							●			●		●			
Streamflows														●					●			●
Groundwater yield	●												●	●					●			●
Groundwater elevation and 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	●	●			●		●															●

Table 12-5. Climatic Summary (10-Yr Record).[8]

Parameter	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total (or Avg)
	Month												
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)						Operational period (170 days)			No application (61 days)			

or

$$Q_{year} = 365Q$$

where

Q_{year} = yearly wastewater flow, gal

Q = average daily flow, gpd

Also compute the quality of tested wastewater to be applied with emphasis on BOD_5, SS, nitrogen, phosphorus, and trace elements listed in Table 12-6.

(3) Calculate the weeks per year during which wastewater may be applied to the land. This is accomplished by constructing a table similar to Table 12-5. From this table it may be determined when rainfall is too excessive or the temperature too low to allow for application of the wastewater.

(4) Compute the amount of wastewater that must be stored during periods of nonapplication. This is accomplished by taking the weeks in a year (52) and subtracting the weeks of application of wastewater from it.

(5) Compute the acre feet of wastewater that is to be applied per year.

$$Q_{year}(\text{acre feet}) = Q_{year} \times 3.07 \times 10^{-6} \qquad (12\text{-}2)$$

where

Q_{year}(acre feet) = total acre feet of wastewater to be applied

Q_{year} = yearly wastewater flow, gal

3.07×10^{-6} = conversion factor

(6) Calculate the application rate per week of application.

$$\frac{\text{Application}}{\text{Week}} = \frac{Q_{year}(\text{acre feet})}{\text{Application period(weeks)}} \qquad (12\text{-}3)$$

where

Application/week = wastewater to be applied, acre feet/week

Q_{year} (acre feet) = total yearly wastewater flow

Table 12-6. Recommended Maximum Concentrations of Trace Elements in Irrigation Waters.[7]

Element	For Waters Used Continuously on All Soil, mg/ℓ	For Use Up to 20 yr on Fine-Textured Soils of pH 6.0-8.5, mg/ℓ
Aluminum	5.0	20.0
Arsenic	0.10	2.0
Beryllium	1.10	0.50
Boron	0.75	2.0-10.0
Cadmium	0.010	0.050
Chromium	0.10	1.0
Cobalt	0.050	5.0
Copper	0.20	5.0
Fluoride	1.0	15.0
Iron	5.0	20.0
Lead	5.0	10.0
Lithium	2.5[b]	2.5[b]
Manganese	0.20	10.0
Molybdenum	0.010	0.050[c]
Nickel	0.20	2.0
Selenium	0.020	0.020
Zinc	2.0	10.0

[a]Normally these levels will not adversely affect plants or soils. No data are available for mercury, silver, tin, titanium, or tungsten.

[b]Recommended maximum concentration for irrigating citrus is 0.075 mg/ℓ.

[c]For acid fine-textured soils only or acid soils with relatively high iron oxide contents.

Application period = weeks of application

(7) Select an application rate from Table 12-3 and convert to feet/week. This is the design application rate.

(8) Compute the land area required for application.

$$\text{Area required} = \frac{\text{Application/week}}{\text{Design application rate}} \qquad (12\text{-}4)$$

where

Area required = land area for treatment, acres

Application/week = wastewater to be applied/week

Design application rate = application rate from (7) above

(9) Compute the nutrient loading on the treatment area by converting the nutrient concentration in the wastewater to pounds per acre.

(10) Select a crop to be planted on the land treatment site that will remove the applied nutrients. Crops may be selected from Table 12-7.

Table 12-7. Reported Nutrient Removal by Forage Crops, Field Crops, and Forest Crops.[9]

Crops	Nitrogen Uptake (lb/acre/yr)
Forage	
Coastal bermuda grass	480-600
Reed canary grass	226-359
Fescue	275
Alfalfa	155-220[a]
Sweet clover	158[a]
Red clover	77-126[a]
Lespedeza hay	130
Forest	
Young deciduous (up to 5 yr)	100[b]
Young evergreen (up to 5 yr)	60[b]
Medium and mature deciduous	30-50[b]
Medium and mature evergreen	20-30[b]

[a] Legumes remove substantial nitrogen requirements from the air.
[b] Estimated.

d. Example calculations.

Example calculations will be given for spray irrigation only.

(1) The following parameters are known:

(a) The climatic summary is given in Table 12-5.

(b) Q = 6000 gpd

(c) BOD_5 = 29 mg/ℓ

(d) SS = 30 mg/ℓ

(e) Total nitrogen = 20 mg/ℓ

(f) Total phosphorus = 15 mg/ℓ

(2) Calculate the quantity of wastewater to be applied per year.

$$Q_{year} = 6000 \text{ gpd} \times 365 \text{ days/year}$$

$$Q_{year} = 2{,}190{,}000 \text{ gal/year}$$

(3) Calculate the weeks per year it is feasible to apply the wastewater.

From Table 12-5,

Operation period = 170 days

170 days = 24.3 weeks, say 24 weeks

(4) Compute storage needed for year.

52 weeks/year - 24 weeks application
= 28 weeks storage

(5) Since 1,000,000 gallons = 3.07 acre feet

2.19 million gallons × 3.07
= 6.72 acre feet/year.

$$Q_{year} = 6.72 \text{ acre ft/year}$$

(6) Since application period is 24 weeks

$$\text{Application/week} = \frac{6.72 \text{ acre feet/year}}{24 \text{ weeks/year}}$$

Application rate = 0.28 acre feet/week

(7) Soils and underlying rock formations allow an application rate of 2 in./week

$$2 \text{ in./week} \times \frac{1 \text{ ft}}{12 \text{ in.}} = 0.17 \text{ ft/week}$$

(8) Compute land area required

$$\text{Area required} = \frac{\text{Application rate}}{\text{Design application rate}}$$

$$\text{Area required} = \frac{0.28 \text{ acre feet/week}}{0.17 \text{ ft/week}}$$

Area required = 1.65 acres

(9) Compute the nutrient loading (N,P) on the treatment area.

Total N available = 20 mg/ℓ

Convert N to lb N/acre/year

$$\text{lb N/acre/year} = \frac{\text{Q year} \times \text{mg/ℓ N} \times 8.34}{\text{acres}}$$

$$= \frac{2.19 \text{ M6} \times 20 \text{ mg/ℓ} \times 8.34}{1.65}$$

$$= 222.1 \text{ lb/acre/year}$$

Similarly:

Total P available = 15 mg/ℓ

Convert P to lb P/acre/year

$$\text{lb P/acre/year} = \frac{2.19 \text{ M6} \times 15 \text{ mg/ℓ} \times 8.34}{1.65}$$

lb P/acre/year = 166 lb/acre/year

(10) Select some crop to plant that has a high nitrogen uptake. From Table 12-7 select Reed Canary Grass which removes 226-359 lb N/acre/year. Therefore, Reed Canary Grass will utilize the available nitrogen and the load to the ground water will be negligible.

12-7. REFERENCES

1. Agriculture Research Service, U. S. Department of Agriculture Publication, "Factors Involved in Land Application of Agricultural and Municipal Wastes," National Program Staff, Soil, Water, and Air Sciences, Beltsville, Md., Jul 1974.

2. United States Environmental Protection Agency, "Alternative Waste Management Techniques for Best Practicable Waste Treatment."

3. Kardos, L. T. et al., "Renovation of Secondary Effluent for Reuse as a Water Resource," EPA 660/2-74-016, Feb 1974, Environmental Protection Agency.

4. Technical Bulletin, "Evaluation of Land Application Systems," EPA-430-9-75-001, Mar 1975, U. S. Environmental Protection Agency.

5. Reed, S. et al., "Wastewater Management by Disposal on the Land," Special Report 171, May 1972, U. S. Army Cold Regions Research Engineering Laboratory, CE, Hanover, N. H.

6. Hoeppel, R. E. et al., "Wastewater Treatment on Soils of Low Permeability," Miscellaneous Paper Y-74-2, Jul 1974, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

7. National Academy of Science--National Academy of Engineering, Environmental Study Board, ad hoc Committee on Water Quality Criteria 1972, "Water Quality Criteria 1972," 1974, U. S. Government Printing Office.

8. Cold Regions Research and Engineering Laboratory, "Short Course on Application of Geology and Soil Science to Wastewater Management Via Land Treatment."

9. Environmental Protection Agency, "Land Treatment of Municipal Wastewater Effluents; Design Factors--I," Jan 1976.

13. PROPRIETARY SYSTEMS

13-1. GENERAL

Recirculation systems recently placed on the market that may not be familiar to the reader are the Chrysler Corporation (Space Division) closed-loop no-discharge system (Aqua-Sans) which uses mineral oil as the flush fluid to transport waste, the Thiokol Chemical Corporation closed-loop recirculating saltwater process (Zero Discharge), and the Monogram Industries Magic Flush toilet system.[1,2]

13-2. AQUA-SANS SYSTEM

The Aqua-Sans is a unique system that may be applicable to rest areas where available water is a problem. The system is a no-discharge, nonbiological, recirculating sewage treatment system that disposes of all sanitary (toilet) wastes. The system does not require water for operation of conventional toilets, but uses a permanent flush fluid, i.e., mineral oil, to transport the waste. The volume of the waste requiring disposal is reduced by about 98 percent by recovery of most of the flush fluid.

A special training report furnished by FHWA, Region 6, described the use of the Aqua-Sans system at a rest area in Texas.[3] After 5 months of operation of the system, the following conclusions were made.

- a. Aqua-Sans Disposal system can be designed to produce a highly treated effluent, or can incorporate a recirculating system that eliminates the need for effluent discharge.
- b. The system can be used in areas of low water supply or areas of low soil permeability.
- c. Incineration of the sewage sludge complies with EPA standards.
- d. There are high initial capital and operating costs.
- e. No major operational problems have been encountered; however, continued research and development is needed to make the system more efficient and economical to operate.

Although long-term operating experience is not presently available because of the newness of the process, certain situations requiring water conservation or no effluent discharge may mandate the use of this or a similar system in order to provide hygienically safe and adequate wastewater disposal for rest areas.

Another rest area employing an Aqua-Sans system is located on Highway 82 near Starkville, Mississippi. To date, no operational or maintenance problems have been encountered with this system. An Aqua-Sans system is also being employed at the New Kent County, Virginia, rest area on Interstate 64 on an experimental basis. As more information becomes available about these or other installations it will be furnished to the user.

13-3. ZERO DISCHARGE SYSTEM

The Thiokol Chemical Corporation Zero Discharge system uses a closed-loop recirculating saltwater treatment process. Flushed waste enters the system over a screen that separates large and intermediate size solids into a sludge tank. Underflow water from the screen collects in a surge storage tank, where it is pumped at a constant rate through a settling tank for further solids removal. Settled solids are periodically pumped to the sludge tank.

An oil-fired incinerator may be used for sludge disposal. The water from the settling tank overflows to a recirculating tank, where it is recirculated through Pepcon cells for removal of dissolved organic material and for disinfection. The portion of the recirculating water equal to the overflow from the settling tank flows into an effluent-holding tank. Flush water is provided on demand from the effluent-holding tank to an accumulator, which provides flush water to the toilets and urinals in the rest rooms.

The Thiokol Zero Discharge system was evaluated in the spring of 1973 by an on-site demonstration at a Utah State Department of Highways rest area.[4] After 2-1/2 months of study, the Utah State Department of Highways reached the following conclusions.

a. The Thiokol system was virtually maintenance free.

b. Operation and maintenance (O&M) time required approximately 30 min per day.

c. The major operating cost was electricity (approximately $5.00 per day).

d. It was evident that the system could not always compare with conventional biological treatment on an economical basis. In the case of this test site, water was hauled by truck from a municipal source; therefore, the system provided a great savings.

e. Acceptance of the system by the general public was good.

The system produces clear, odorless, sterile water for recycling as a flushing medium. It uses standard flush fixtures, is modular in construction, and can be obtained in various capacities.

13-4. MAGIC FLUSH SYSTEM

The Monogram Industries Magic Flush system is similar to the Aqua-Sans system in that a mineral-derived liquid is used as transport media instead of water. This system is also recirculating with storage area for the settled waste. The system only requires periodic waste pumpout and replacement of the purification elements located on the recirculation line.

The Magic Flush system is presently being used at rest areas in Minnesota. As information becomes available on the initial capital cost, operating and maintenance costs, and the reliability of the system; it will be furnished the users of this manual.

13-5. REFERENCES

1. Matthew, F. L. and Nesheim, E. E., "Demonstration of a Non-Aqueous Sewage Disposal System," EPA 670/2-73-088, Dec 1973, U. S. Environmental Protection Agency, Washington, D. C.

2. Hayes, S. F. and Clark, D. P., "Performance Testing of the Thiokol Zero Discharge Waste Treatment System with Special Emphasis on Virus Chlorination," May 1974, Thiokol (Wasatch Division), Brigham, Utah.

3. Rodman, R. S., "Rest Area Sewage Disposal Systems," 1974, Texas Highway Department, Austin, Texas.

4. Anderson, C. V., "Zero Discharge Sanitation System," <u>53rd Annual WASHO Conference</u>, Jun 1974.

14. INTERMITTENT SAND FILTER SYSTEMS

14-1. PROCESS DESCRIPTION

Intermittent sand filters will provide secondary treatment of septic tank effluent in areas where soil absorption systems are not acceptable because of unsuitable soils, possible contamination of groundwater, or regulatory agency prohibitions, or they can provide tertiary or additional treatment to the effluent from such secondary treatment processes as extended aeration activated sludge, rotating biological filter and lagoons. The intermittent sand filter removes suspended solids, oxidizes carbonaceous- and nitrogenous-oxygen demanding materials, and removes large numbers of bacteria. Intermittent dosing of the filter is an absolute necessity for optimum performance. Therefore, a dosing tank, equipped with a dosing siphon or pump, is an indispensable element of the septic tank-sand filter system. In contrast to soil absorption systems where most of the wastewater percolates through the soil to groundwater, sand filtration systems produce an effluent that is usually discharged to surface waters.

Intermittent sand filtration of domestic wastewater has been used in this country since 1889.[1] Much of the early development work was performed by the Massachusetts Board of Health. Sand filtration of municipal wastewater soon became a prevalent treatment alternative. However, as cities grew in size, the land requirements for large systems caused cities to look to other forms of treatment, although the sand filter had proved to be a viable wastewater-treatment alternative. The WPCF Manual of Practice No. 8[1] expressed considerable confidence in the process in stating in 1967 "Intermittent sand filters, when used for treating domestic sewage, will produce effluents of the highest degree of treatment now known." For small systems, such as those required at rest areas, intermittent sand filtration must be included in the processes to be considered for meeting the requirements of PL 92-500.

Removal of contaminants by an intermittent sand filter is accomplished by physical-chemical processes and biological processes. Suspended solids that escape through the septic tank may be strained or physically adsorbed on the soil surfaces. Dissolved organics and other substances may also be adsorbed on the soil particles. In this manner the settled effluent from secondary treatment processes may receive further treatment with the intermittent sand filter. A continued buildup of adsorbed substances on the soil particles would clog the bed and require replacement of the first few inches of the filter media. However, in a properly constructed and operated sand filter, this is not necessary. Microorganisms, primarily bacteria, in an aerobic environment metabolize the adsorbed organic carbonaceous and nitrogenous substances to water, carbon dioxide, bicarbonates, sulfates, phosphates, and nitrates.[2] Growth and reproduction of the bacteria produce a microbial mass which also would clog the pores of the filter if it were not for the consumption of the bacteria by protozoa and metazoa. These predators consume the bacterial sludges and slimes and keep the bed open and active.[3] The key to successful operation of the sand filter is maintaining an aerobic environment in the bed for the aerobic bacteria. Anaerobic bacteria will not sufficiently stabilize the organics in the wastewater, and will also produce slimes that tend to clog the filter.[2] Aerobic conditions are maintained by operating the filter intermittently since continuous flooding of the filter would not allow atmospheric oxygen to diffuse into the pores of the filter. Dosing the filter several times per day, with rest periods between doses, allows the liquid to percolate through the media, and then allows the media to dry out between doses so that the oxygen consumed by biological degradation or organic materials may be replaced. Removal of coliform bacteria in a sand filter has been attributed to formations of a mat of organic matter on the sand surface that acts as a retentive filter and aids in removing these bacteria.[4]

Intermittent sand filters for municipal systems are generally uncovered and may be above or below grade. Most of the literature deals with this type of system. However, for small waste flows earthen covered sand filters constructed below grade have proved to be effective and offer the following advantages over open sand filters: reduced odors and improved aesthetics, fewer problems with flies and other insects, less maintenance and operation, and superior protection from temperature extremes. On the other hand, open sand filters have the following advantages: higher application rates, accessibility for cleaning or replacement of sand, control of distribution, and higher quality effluent. Where accessible land is the controlling factor, open filters are usually used. State and local health regulations should be reviewed to determine if open sand filters are permitted. At rest areas, the aesthetic factor and low operational requirements will generally dictate a subsurface sand filter unless prohibited by a high groundwater table. In which case, a covered or uncovered abovegrade installation would be required.

Design guidance and construction features for septic tank-sand filter systems are given in Manual of Septic Tank Practice.[5] Subsurface systems may be either sand filter trenches or sand filter beds, the latter being a more economical design for larger systems. Open sand filters are also discussed in detail in the WPCF Manual of Practice No. 8.[1] Design features of the subsurface sand filter will be reviewed in this paragraph since this is the most likely choice for rest areas.

Figure 14-1 is a diagram of a typical subsurface sand filter. Effluent enters a dosing tank that is sized for a capacity equal to 60-75 percent of the volume of the distributors dosed at one time; two or more separate sand filter sections should be alternately dosed where more than 1000 ft of distributors are required.[5] Dosing siphons or level-controlled pumps may be used to discharge effluent out of the bed. Distributors are commonly spaced 4 to 6 ft apart and are usually perforated pipe. Subsurface sand filters may be dosed several times per day if the dosing tank is designed according to Manual of Septic Tank Practice as given above. Open sand filters are normally dosed twice daily, covering the bed with 2 to 4 in. of water each dose.[1,6] Subsurface beds cannot accommodate as large a volume in one application as can open filters, but intermittent loading, even with shortened rest periods, will increase oxygenation of the bed and hence increase efficiency and life of the bed.

The media for a subsurface sand filter should be clean, washed

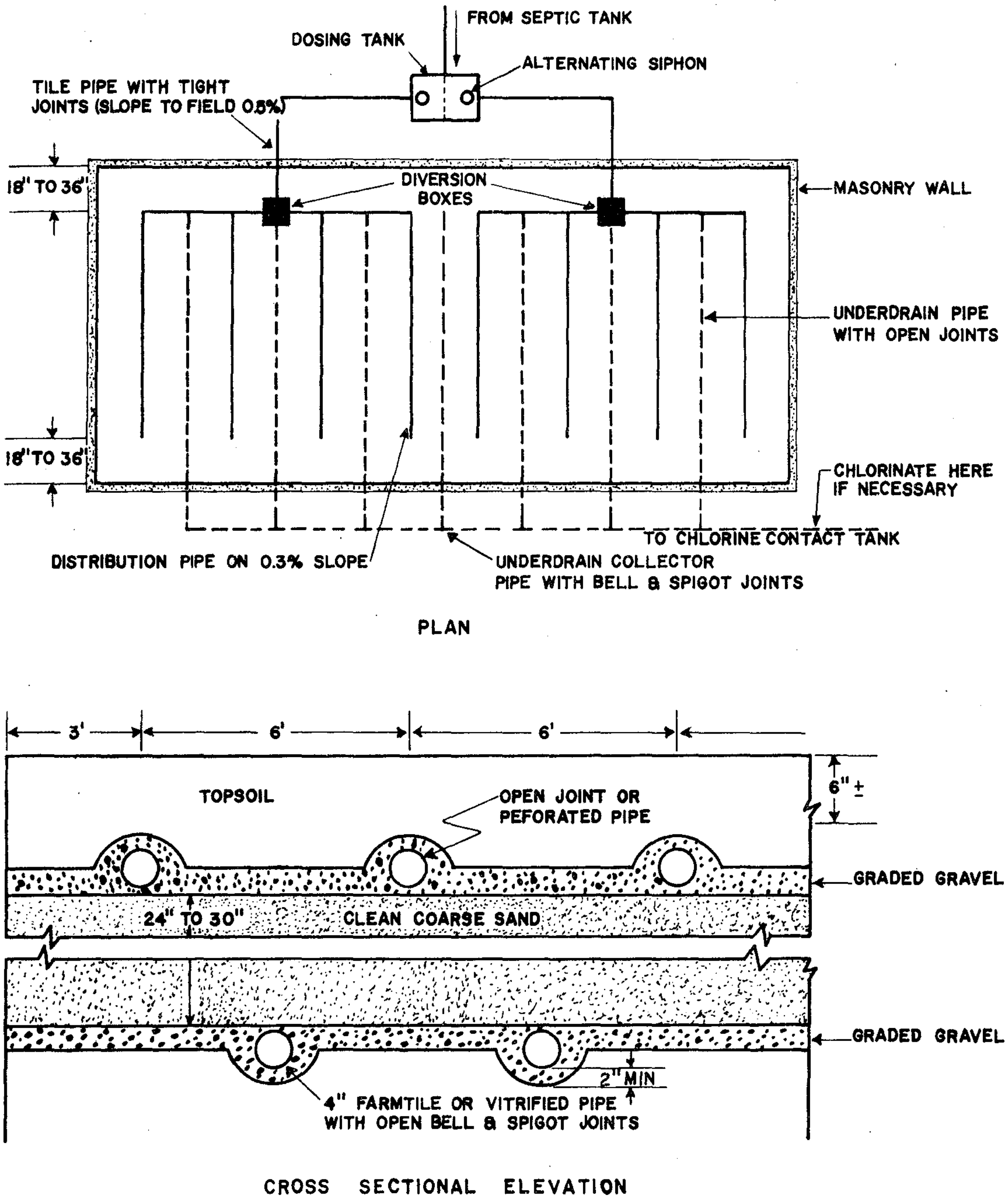


Figure 14-1. Typical subsurface sand filter.

sand with an effective size between 0.4 and 0.6 mm and a uniformity coefficient less than 4.[5] The sand depth is normally 2 to 2-1/2 ft. Grantham et al.,[7] studied application of settled domestic wastewater on Florida sands and demonstrated that smaller sands and increased bed depths afforded greater removal of BOD and TKN. However, higher application rates are possible with coarser sands because the chance for clogging due to anaerobic growth is less. Application rates for open sand filters with different sized sands are given in References 1, 5, and 7.

14-2. PERFORMANCE

Intermittent sand filter systems will provide the necessary degree of treatment to meet the requirements of PL 92-500. Salvato[8] analyzed 51 septic tank effluent samples and 56 subsurface sand filter effluent samples from 19 establishments, including 3 children's summer camps, 1 school, 5 motels, 2 private residences, 3 restaurants, 2 housing developments, 2 trailer camps and 1 factory in the state of New York. Selections from Salvato's work are given in Table 14-1. These results

Table 14-1. Septic Tank and Subsurface Sand Filter Effluent Characteristics.[8]

Parameter	Septic Tank Effluent	Subsurface Sand Filter Effluent
Coliform bacteria, MPN	>110,000,000	150,000
pH	7.4	7.4
BOD_5	140	4
DO, mg/ℓ	0	5.2
Nitrogen, total, mg/ℓ	36	21
Free ammonia, mg/ℓ	12	0.7
Organic nitrogen, mg/ℓ	12	3.4
Nitrite + nitrate nitrogen, mg/ℓ	0.12	17
Suspended solids, mg/ℓ	101	12

indicate that efficient removal of BOD_5, TKN, and suspended solids can be provided by intermittent subsurface sand filters. Such an effluent would meet most stream standards insofar as oxygen-demanding criteria are concerned. Phosphorus removal has not been reported in most literature studies, but phosphorus removal is not expected to be significant.

Open sand filters also produce a high quality effluent with BOD and TSS concentrations less than 10 mg/ℓ.[1]

Intermittent sand filter systems can withstand shock hydraulic or organic loadings encountered at rest areas as well as any treatment alternative. One possible problem might be washout of solids from the septic tank or the final clarifier in the extended aeration plant to the sand filter, thereby clogging the filter. This problem is minimized with timely removal of solids and scum buildup from the septic tank and control of the solids level in extended aeration plants. The septic tanks, lagoons, and extended aerated plants provide equalization for the loading to the sand filter, thereby eliminating overload of the filter. Periods of little or no flow may benefit the intermittent sand filter system by allowing the bed to dry out and aerate in contrast to significant harm to other biological systems. The filter can, therefore, be designed to accommodate future maximum flows while working with low daily flows.

Temperature slightly decreases efficiency of the sand filter system,[7] but freezing of the surface of the bed will terminate operation. Placing the filter underground provides protection from freezing, however the problem of freezing must be considered for northern climates.

14-3. FLEXIBILITY

Expansion of a sand filter system at a rest area will require construction of a new filter. Subsurface filters must be completely reconstructed if the sand or distributors should become completely clogged. Where nutrient removal is required, additional processes may be added to an intermittent sand filter system. Chemical precipitation of phosphorus could take place before or after sand filtration. Removal prior to filtration is preferable since any suspended solids produced are removed by the intermittent sand filter. Nitrogen removal would require a denitrification system after the sand filter.

14-4. RELIABILITY

Intermittent tank sand filters provide reliable treatment at rest areas. This is evidenced by the number of systems in use at highway rest areas. Salvato[8] surveyed 1500 individual home systems in New York and found only nine systems that had failed and causes for these failures "were obvious on inspection." A primary concern might be odors from the system, but Salvato spot-checked 70 systems and found only one case where a slight odor was detected.

14-5. OPERATION

Operation of a subsurface sand filter is one of the simplest among wastewater-treatment alternatives at rest areas. Frequent inspection of the dosing tank for possible clogging of siphons or distribution lines should prevent serious problems for a properly designed system. An odoriferous black effluent from the filter will indicate that the filter is not being allowed adequate time to dry out between doses. This can be confirmed by measuring the dissolved oxygen (DO) concentration in the effluent. If no DO is detected, then a longer rest period between doses may improve operation. Otherwise, the bed may have to be replaced or expanded by building an alternate bed.

14-6. PRELIMINARY DESIGN

a. Inputs required.

Daily flow rate Q , gpd

b. Design calculations.

(1) Determine area A_f required for filter, sq ft

$$A_f = \frac{Q}{AR} \tag{14-1}$$

where

Q = Flow, gpd

AR = Application rate, gpd/sq ft (2-10 gpd/sq ft)

(2) Determine length L_d of distributors, ft

$$L_d = \frac{A_f}{S} \tag{14-2}$$

where S = spacing between distributor lines, ft.

(3) Determine volume of dosing tank, V_{dt}

$$V_{dt} = 0.75(L_d)(\pi)\left(r_d^2\right) \quad (14\text{-}3)$$

where

V_{dt} = volume of tank, cu ft

r_d = radius of distributor line, ft

c. Design example.

(1) $A_f = \frac{Q}{AR}$

$Q = 6000$ gpd

Assume

$AR = 2.5\ \text{gpd/ft}^2$

$$A_f = \frac{6000\ \text{gpd}}{2.5\ \text{gpd/ft}^2}$$

$$A_f = 2400\ \text{ft}^2$$

(2) $AR = 2.5\ \text{gpd/ft}^2$

$1\ \text{gpd/ft}^2 \times 1.6 = \text{in./day/ft}^2$

$2.5\ \text{gpd/ft}^2 = 4\ \text{in./day/ft}^2$

Apply 2 in. twice daily. Use sand with uniformity coefficient ≈3 and size 0.30 to 0.55 mm.

(3) $L_d = \frac{A_f}{S}$

assume $S = 6$ ft

$$L_d = \frac{2400\ \text{ft}^2}{6\ \text{ft}}$$

$$L_d = 400\ \text{ft}$$

(4) $V_{dt} = 0.75(L_d)(\pi)r_d^2$

Assume $r_d = 2$ in. $= 0.167$ ft

$$V_{dt} = 0.75(400)(\pi)(0.167^2)$$

$V_{dt} = 27$ cu ft (numbers rounded up for conservation design)

14-7. REFERENCES

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15. DISINFECTION

15-1. PROCESS DESCRIPTION

Disinfection of treated wastewater is commonly practiced throughout the United States for the control or elimination of waterborne disease causing organisms (pathogens). Pathogens consist mainly of viruses and bacteria but may also include protozoa, fungi, and other organisms.

In rest area wastewater treatment, disinfection is normally accomplished just prior to final discharge. In this way any waterborne pathogen is controlled before leaving the rest area and introduction into a potential water supply (stream, river, etc.) is accomplished.

At present there are many types of disinfection available for use in treating wastewater. Among these are the use of ozone (O_3), chlorine, and other halogens, radiation and treatment with light. Because it is the most economical method of disinfection and because it has received the most widespread use, disinfection by chlorine will be the only method discussed in this report.

In disinfection with chlorine (chlorination), chlorine in some form comes in contact with the wastewater for some period of time during which the chlorine is completely mixed with the wastewater and destruction of the pathogens takes place. In most rest areas chlorination will take place in a chlorine contact chamber that is designed for a minimum flow-through contact time of 15 to 30 minutes (a minimum detention time of 15 minutes is specified to insure that the chlorine has completely mixed with the effluent and that disinfection has taken place).

The rate at which disinfection takes place is affected by both the concentration of the chlorine and the contact time and is described by Chick's law,

$$-\frac{dN^{\circ}}{dt} = kN^{\circ} \qquad (15\text{-}1)$$

where dN°/dt is the die-off rate of the organisms, k is the die-off rate constant and N° is the number of organisms present at any time, t . The amount of chlorine required for disinfection and the contact time may also be influenced by the pH of the wastewater (Figure 15-1), the temperature of the wastewater and the interference of wastewater constituent concentrations (Table 15-1).

Table 15-1. Typical Chlorine Dosages for Disinfection.

Effluent from	Dosage Range mg/ℓ
Untreated wastewater (prechlorination)	6 to 25
Primary sedimentation	5 to 20
Trickling filter plant	3 to 15
Activated sludge plant	2 to 8
Multimedia filter following activated sludge plant	1 to 5

The amount of chlorine that is necessary for the destruction of the pathogens is referred to as chlorine demand. To determine if enough chlorine has been added to the wastewater to destroy the pathogens the amount of chlorine that is present after the contact period is measured. This value is called the chlorine residual and will be detected only when the chlorine demand of the wastewater has been met (Figure 15-2). In the treatment of wastewater it is desirable that the chlorine residual should be, as a minimum, 0.5 mg/ℓ after a minimum contact period of 15 minutes. In this manner, disinfection of the wastewater may be assured. Local requirements may require a higher residual or no residual chlorine at discharge, therefore, it is recommended that the local health authorities be contacted prior to the design of the chlorination unit.

In the treatment of wastewater, various types of chlorine-feed equipment (called chlorinators) may be used. These types are gas cylinders using direct feed or solution feed and hypochlorinators which use a solid form of chlorine.

A typical chlorinator setup is shown in Figure 15-3. In this method of chlorination, water and chlorine gas are mixed together to form a solution that is then injected into the wastewater in the

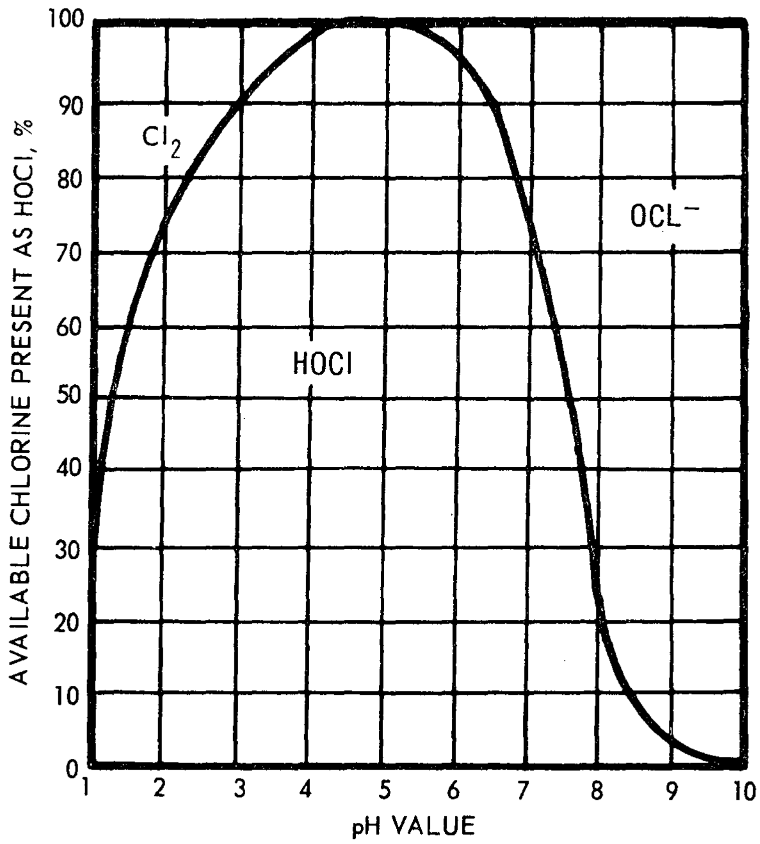

Figure 15-1. Effect of pH value on form of free available chlorine in water.

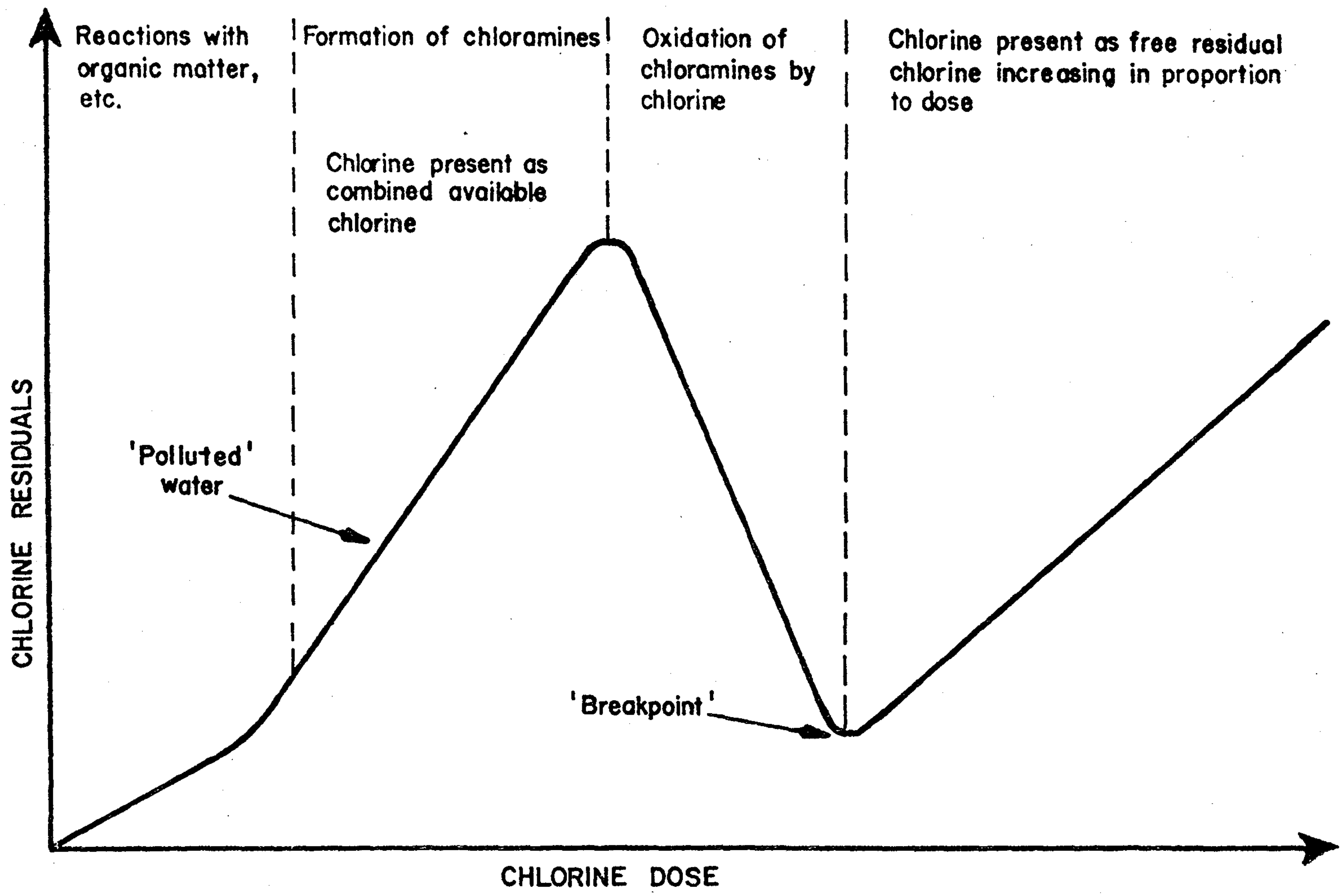

Figure 15-2. Reactions occurring during breakpoint chlorination.

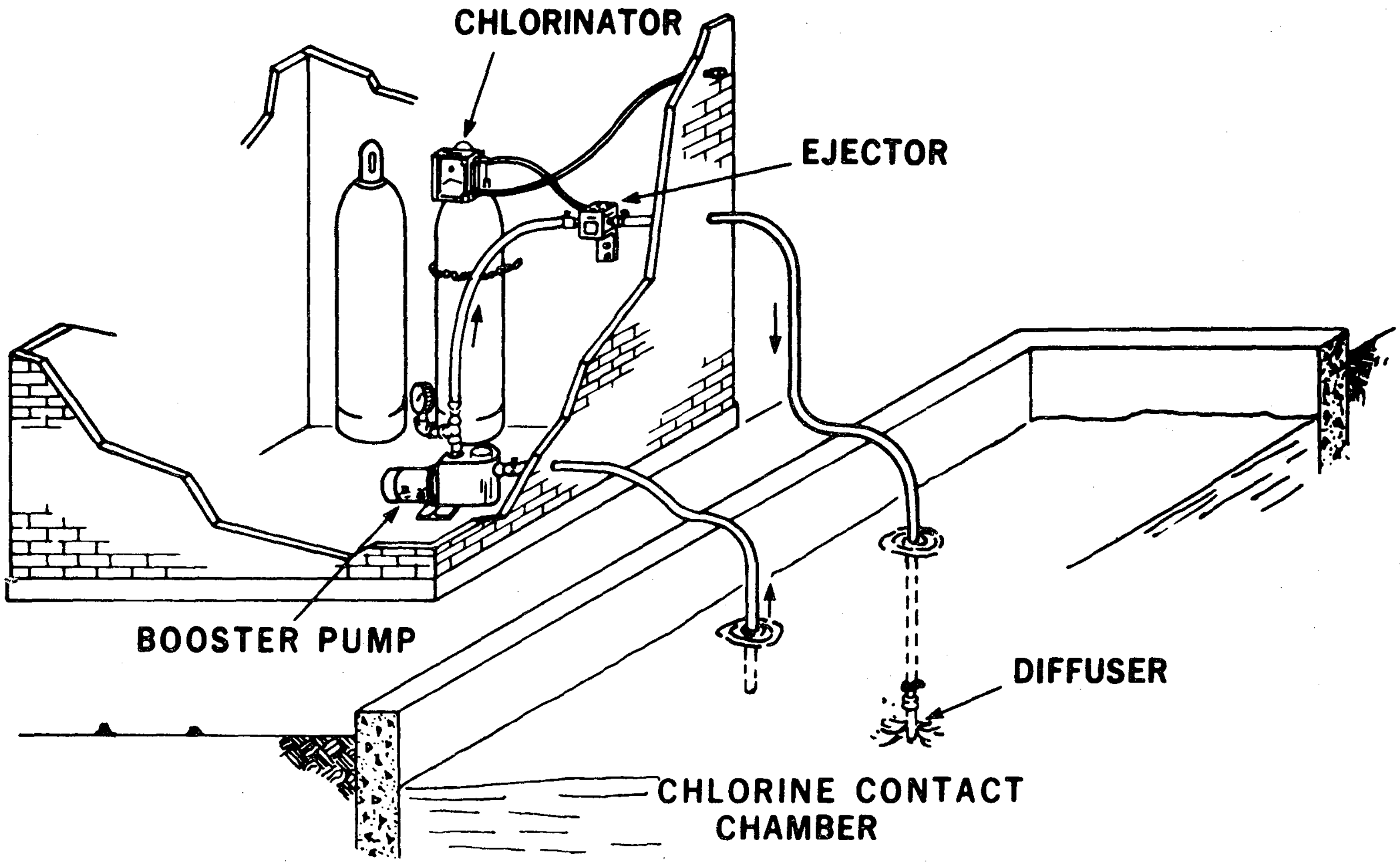

Figure 15-3. Chlorination of sewage.

chlorine contact chamber. By adjusting the rate of feed water and gaseous chlorine the desired dosage of chlorine can be applied to the wastewater. In a similar manner wastewater passes through a hypochlorinator and the wastewater comes into contact with the solid chlorine.

15-2. PERFORMANCE

The chlorination of wastewater prior to final discharge has been shown to destroy pathogens in the wastewater. Chlorination of wastewater also removes odors from the water and may reduce the final BOD of the effluent. Chlorination has been successfully practiced for over 50 years for the control of waterborne disease and should continue to be one of the major forms of wastewater disinfection.

15-3. FLEXIBILITY

The use of gas chlorinators with solution water readily lends itself to the increase or decrease of the chlorine dose to the wastewater. However, the chlorine contact chamber, because it is constructed for one detention time, cannot be expanded without major physical alterations. As such it may be necessary to construct a second chlorine contact tank to be used for increased future flows or it may be wise to overdesign the original chlorine contact tank by providing for a minimum detention time of 30 minutes instead of 15.

15-4. RELIABILITY

The ability of chlorine to disinfect wastewater has been proven by time. However, because chlorinators are mechanical and because wastewater flows from rest areas are variable, some interruption of optimum dosing levels may occur.

15-5. OPERATION

Operation of a chlorinator at a rest area is accomplished by adjusting the flow rate from the gas chlorine cylinder and from the makeup water pump. In this manner the desired chlorine dose may be maintained. It is good practice to maintain two gas chlorine cylinders at the treatment facility to insure that when one cylinder becomes empty another may be quickly switched on to avoid any interruption in chlorination. Booster pump feed rate, gas chlorine feed rate, and chlorine residual should be checked periodically (weekly) to insure proper disinfection. All mechanical equipment should be checked and maintained weekly to insure proper functioning and to reduce abnormal wear. Proper safety procedures must always be followed when operating and maintaining chlorination equipment.

15-6. PRELIMINARY DESIGN

The design of gas cylinder solution feed chlorinators is given below. Additional design for other types of chlorination equiment may be made by consulting the literature and/or the manufacturer of chlorination equipment.

a. Input required.

Q = average daily flow, gpd

Q_p = peak daily flow = $4Q$, gpd

t = contact time at Q_p, minutes (30 minutes)

n = number of contact tanks (2)

d = depth (8 ft)

b. Design procedure.

(1) Select minimum time (t) at peak flow (Q_p) to insure disinfection.

$$V = \frac{Q\left(\frac{Q_p}{Q}\right)t}{1440} \quad (15\text{-}2)$$

where

V = volume of chlorine contact tank, gal

Q_p = peak flow = $4Q$

Q = average daily flow

t = contact time = 30 minutes

1440 = conversion factor

(2) Select a depth and calculate surface area.

$$SA = \frac{V}{d7.48} \quad (15\text{-}3)$$

where

SA = surface area, sq ft

V = volume, gal

d = depth, feet (8)

7.48 = conversion factor

(3) Select chlorine dosage from Table 15-1 and calculate chlorine requirements.

$$CR = Q(CD)8.34 \times 10^{-6} \tag{15-4}$$

where

CR = chlorine requirement, lb/day

Q = average daily flow, gpd

CD = chlorine dosage, mg/ℓ

8.34×10^{-6} = conversion factor

c. Output obtained.

V = volume of chlorine contact tanks, gal

SA = surface area of chlorine contact tank, sq ft

CR = chlorine requirement, lb/day

d. Example calculations.

(1) Determine required inputs.

Q = average daily flow = 6000 gpd

$Q_p = 4Q$ = peak flow = 2400 gpd

t = minimum contact time = 30 minutes

n = number of tanks = 2

d = depth = 8 ft

(2) Calculate volume (V) of chlorine contact tank.

$$V = \frac{Q \; \frac{Q_p}{Q} \; t}{1440}$$

$$V = \frac{6{,}000 \text{ gpd} \left(\frac{24{,}000 \text{ gpd}}{6{,}000 \text{ gpd}}\right) 30 \text{ minutes}}{1{,}440 \text{ minutes/day}}$$

$V = 500$ gallons

(3) Calculate surface area.

$$SA = \frac{V}{d(7.48)}$$

$$SA = \frac{500 \text{ gal}}{8 \text{ ft}(7.48 \text{ gal/cu ft})}$$

$$\underline{SA = 8.4 \text{ sq ft}}$$

Provide two tanks, each 8.4 feet long by 1 foot wide to insure mixing.

(4) Calculate chlorine requirements. Select chlorine dosage of 8 mg/ℓ for activated sludge plant effluent.

$$CR = Q(CD)8.34 \times 10^{-6}$$

$$CR = 6000 \text{ gpd}(8 \text{ mg}/\ell)8.34 \times 10^{-6}$$

$$\underline{CR = 0.4 \text{ lb/day}}$$

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16. ECONOMIC EVALUATION OF TREATMENT ALTERNATIVES

16-1. INTRODUCTION

The various sizes and types of wastewater-treatment systems available for use at rest areas have associated capital costs and O&M costs. To arrive at a basis for comparing the costs of the various treatment systems, a method for determining the present worth of all O&M costs and the equivalent annual costs of the systems is presented.

Present worth for a uniform series O&M costs is equal to the annual O&M costs times the uniform series present worth factor at some interest rate for the design life of the treatment system. The uniform series present worth factor is tabulated in engineering economics textbooks for various interest rates (percent) and design periods (years).

Equivalent annual cost of a system is calculated by multiplying the sum of the capital cost and the present worth of the annual O&M costs by the capital recovery factor. The capital recovery factor is found in engineering economics textbooks and is a function of the interest rate (percent) and the design period (years).

16.2. EXAMPLES

Examples for determining the equivalent annual cost of a wastewater treatment system follow (Table 16-1).

Table 16-1. Interest Factors.

i	Compound Interest $(1 + i)^n$	Present Worth $(1 + i)^{-n}$	Amount of Annuity $\frac{(1 + i)^n - 1}{i}$	Present Worth of Annuity $\frac{1 - (1 + i)^{-n}}{i}$
5	2.6533	0.37689	33.0660	12.4622
5-1/2	2.918	0.34272	34.8683	11.9504
6	3.207	0.31180	36.7856	11.4699
6-1/2	3.524	0.2838	38.8253	11.0185
7	3.870	0.2584	40.995	10.594
7-1/2	4.248	0.2354	43.305	10.194
8	4.661	0.2145	45.762	9.818
9	5.604	0.1784	51.160	9.129
10	6.727	0.1486	57.275	8.514

Note: n = period = 20 yr; i = interest.

Example 1

Treatment system size = 15,000 gpd

Design life = 20 yr

Initial cost = $40,000

Annual O&M cost = $1,500

Interest rate = 7%

a. Present worth (PW) of O&M equals annual O&M times uniform series present worth factor at 7% for 20 yr.

From Table 16-1 the PW of an annuity at 7% interest is 10.594 times that annuity ($1,500).

$$PW = \frac{1 - (1 + i)^{-n}}{i} (O\&M) \tag{16-1}$$

where

PW = present worth

$\frac{1 - (1 + i)^{-n}}{i}$ = PW of annuity factor (Table 16-1)

$O\&M$ = annual operation and maintenance cost

In this example PW is expressed as:

PW = 10.594 ($1,500)

PW = $15,891

b. Total present worth (TPW) equals present worth of O&M plus capital cost.

$$TPW = PW + \text{initial cost} \tag{16-2}$$

where

TPW = total present worth

PW = present worth of annual costs

Initial cost = initial cost

In this example total present worth is expressed as:

TPW = $15,891 and $40,000

TPW = $55,891

c. Equivalent annual cost (EAC) equals total present worth times capital recovery factor at 7 percent for 20 yr.

$$EAC = TPW \times CRF \tag{16-3}$$

where

EAC = equivalent annual cost

TPW = total present worth

CRF = capital recovery factor = $\dfrac{1}{\text{present worth of annuity factor}}$

(from Table 16-1)

or

$$EAC = TPW \times \frac{i}{1 - (1 + i)^{-n}}$$

$$EAC = \$55{,}891 \times 0.09439 = \$5{,}275.55$$

In this example equivalent annual cost is expressed as:

$$EAC = \$55{,}891 \times \frac{1}{10.594}$$

$$EAC = \$5{,}275.72$$

Example 2

Treatment system size = 15,000 gpd

Design life = 20 yr

Initial cost = $20,000

Annual O&M cost = $4,300

Interest rate = 7%

(1) $$PW = \frac{1 - (1 + i)^{-n}}{i} \text{ (O\&M)}$$

where

PW = 10.594 ($4,300)

<u>PW = $45,554.20</u>

(2) TPW = PW + Initial cost

where

TPW = $45,554.20 + $20,000.00

<u>TPW = $65,554.20</u>

(3) EAC = TPW × CRF

where

$$EAC = \$65{,}554.20 \left(\frac{1}{10.594}\right)$$

$$EAC = \$6,187.86$$

From the examples above it is seen that both the capital cost and the O&M costs determine the equivalent annual cost. Therefore, even though the capital cost for the system in Example 2 is only one-half that of the system in Example 1, the equivalent annual cost of system 2 is 17 percent greater than system 1 because of the larger O&M cost of system 2.

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GLOSSARY

absorption -- The taking up of one substance into the body of another.

acid -- (1) A substance that tends to lose a proton. (2) A substance that dissolves in water with the formation of hydrogen ions. (3) A substance containing hydrogen which may be replaced by metals to form salts.

acidity -- The quantitative capacity of aqueous solutions to react with hydroxyl ions. It is measured by titration with a standard solution of a base to a specified end point. Usually expressed as milligrams per litre of calcium carbonate.

activated carbon -- Carbon particles usually obtained by carbonization of cellulosic material in the absence of air and possessing a high absorptive capacity.

activated sludge -- Sludge floc produced in raw or settled wastewater by the growth of zoogleal bacteria and other organisms in the presence of dissolved oxygen and accumulated in sufficient concentration by returning floc previously formed.

activated sludge process -- A biological wastewater treatment process in which a mixture of wastewater and activated sludge is agitated and aerated. The activated sludge is subsequently separated from the treated wastewater (mixed liquor) by sedimentation and wasted or returned to the process as needed.

adsorption -- (1) The adherence of a gas, liquid, or dissolved material on the surface of a solid. (2) A change in concentration of gas or solute at the interface of a two-phase system. Should not be confused with absorption.

advanced wastewater treatment (AWT) -- Those processes that achieve pollutant reductions by methods other than those used in conventional treatment (sedimentation, activated sludge, trickling filter, etc.). It employs a number of different unit operations, including lagoons, postaeration, microstraining, filtration, carbon adsorption, membrane solids separation, phosphorus removal, and nitrogen removal.

aerated pond -- A natural or artificial wastewater treatment pond in which mechanical or diffused-air aeration is used to supplement the oxygen supply. See oxidation pond.

aeration -- (1) The bringing about of intimate contact between air and a liquid by one or more of the following methods: (a) spraying the liquid in the air, (b) bubbling air through the liquid, (c) agitating the liquid to promote surface absorption of air. See following terms modifying aeration: diffused-air, mechanical, modified, spiral-flow, step.

aeration period -- (1) The theoretical time, usually expressed in hours, during which mixed liquor is subjected to aeration in an aeration tank while undergoing activated sludge treatment. It is equal to the volume of the tank divided by the volumetric rate of flow of the wastewater and return sludge. (2) The theoretical time during which water is subjected to aeration.

aeration tank -- A tank in which sludge, wastewater, or other liquid is aerated.

aerator -- A device that promotes aeration.

aerobic -- Requiring, or not destroyed by, the presence of free elemental oxygen.

aerobic bacteria -- Bacteria that require free elemental oxygen for their growth.

agglomeration -- The coalescence of dispersed suspended matter into larger flocs or particles which settle rapidly.

air -- The mixture of gases that surrounds the earth and forms its atmosphere, composed primarily of oxygen and nitrogen. It also contains carbon dioxide, some water vapor, argon, and traces of other gases.

algae -- Primitive plants, one- or many-celled, usually aquatic, and capable of elaborating their foodstuffs by photosynthesis.

alkalinity -- The capacity of water to neutralize acids, a property imparted by the water's content of carbonates, bicarbonates, hydrozides, and occasionally borates, silicates, and phosphates. It is expressed in milligrams per liter of equivalent calcium carbonate.

alum -- A common name, in the water and wastewater treatment field, for commercial-grade aluminum sulfate.

aluminum sulfate -- A chemical, formerly sometimes called "waterworks alum" in water or wastewater treatment, prepared by combining a mineral known as bauxite with sulfuric acid.

ammonia -- A chemical combination of hydrogen (H) and nitrogen (N) occurring extensively in nature. The combination used in water and wastewater engineering is expressed as NH_3.

anaerobic -- Requiring, or not destroyed by, the absence of air or free elemental oxygen.

anaerobic bacteria -- Bacteria that grow only in the absence of free elemental bacteria.

anaerobic denitrification -- A means to remove nitrates from wastewaters, especially irrigation return waters that may be high in nitrates and low in organics. In this method, an organic chemical such as methanol, ethanol, acetone, or acetic acid is added as a carbon source and the waste is placed in an anaerobic environment. Under these conditions, nitrate will be reduced by denitrifying bacteria to nitrogen gas and some nitrous oxide, which escapes to the atmosphere. With methanol, the chemistry can be represented as:

$$6H^+ + 6NO_3^- + 5CH_3OH \longrightarrow 5CO_2 + 3N_2 + 13H_2O$$

anaerobic digestion -- The degradation of organic matter brought about through the action of microorganisms in the absence of elemental oxygen.

anion -- A negatively charged ion in an electrolyte solution, attracted to the anode under the influence of electric potential.

average daily flow -- The total quantity of liquid tributary to a point divided by the number of days of flow measurement.

backwashing -- The operation of cleaning a filter by reversing the flow of liquid through it and washing out matter previously captured in it. Filters would include true filters such as sand and diatomaceous-earth types but not other treatment units such as trickling filters.

bacteria -- A group of universally distributed, rigid, essentially unicellular microscopic organisms lacking chlorophyll. Bacteria usually appear as spheroid, rodlike, or curved entities, but occasionally appear as sheets, chains, or branched filaments. Bacteria are usually regarded as plants.

baffles -- Deflector vanes, guides, grids, gratings, or similar devices constructed or placed in flowing water, wastewater, or slurry systems to check or effect a more uniform distribution of velocities; absorb energy; divert, guide, or agitate the liquids; and check eddies.

bar screen -- In a waste-treatment plant, a screen that removes large suspended solids.

biodegradation (biodegradability) -- The destruction or mineralization of either natural or synthetic organic materials by the microorganisms populating soils, natural bodies of water, or wastewater treatment systems.

biologically active floc -- Floc formed by the action of biological agencies; for example, activated sludge.

biological oxidation -- The process whereby living organisms in the presence of oxygen convert the organic matter contained in wastewater into a more stable or a mineral form.

biological process -- (1) The process by which the life activities of bacteria and other microorganisms, in the search for food, break down complex organic materials into simple, more stable substances. Self-purification of polluted streams, sludge digestion, and all the so-called secondary wastewater treatments result from this process. (2) Process involving living organisms and their life activities. Also called biochemical process.

biological slime -- The gelatinous film of zoogleal growths covering the medium or spanning the interstices of a biological bed. Also called microbial film.

biological treatment systems -- "Living" systems which rely on mixed biological cultures to break down waste organics and remove organic matter from solution.

biological wastewater treatment -- Forms of wastewater treatment in which bacterial or biochemical action is intensified to stabilize, oxidize, and nitrify the unstable organic matter present. Intermittent sand filters, contact beds, trickling filters, and activated sludge processes are examples.

BOD -- (1) Abbreviation for biochemical oxygen demand. The quantity of oxygen used in the biochemical oxidation of organic matter in a specified time, at a specified temperature, and under specified conditions. (2) A standard test used in assessing wastewater strength.

BOD load -- The BOD content of wastewater passing into a waste treatment system or to a body of water, usually expressed in pounds per unit of time.

breakpoint chlorination -- Addition of chlorine to water or wastewater until the chlorine demand has been satisfied and further additions result in a residual that is directly proportional to the amount added beyond the breakpoint.

buffer -- Any of certain combinations of chemicals used to stabilize the pH values or alkalinities of solutions.

calcium hypochlorite -- A dry powder consisting of lime and chlorine combined in such a way that, when dissolved in water, it releases active chlorine.

carbonate hardness -- Hardness caused by the presence of carbonates and bicarbonates of calcium and magnesium in water. Such hardness may be removed to the limit of solubility by boiling the water. When the hardness is numerically greater than the sum of the carbonate alkalinity and the bicarbonate alkalinity, that amount of hardness which is equivalent to the total alkalinity is called carbonate hardness. See <u>hardness.</u>

cation -- The ion in an electrolyte which carries the positive charge and which migrates toward the cathode under the influence of a potential difference.

chemical coagulation -- The destabilization and initial aggregation of colloidal and finely divided suspended matter by the addition of a floc-forming chemical. See flocculation.

chemical dose -- The application of a specific quantity of chemical to a specific quantity of fluid for a specific purpose. See dose.

chemical feeder -- A device for dispensing a chemical at a predetermined rate for the treatment of water or wastewater. Change in rate of feed may be affected manually or automatically by flow-rate changes. Feeders are designed for solids, liquids, or gases.

chemical oxygen demand (COD) -- A measure of the oxygen-consuming capacity of inorganic and organic matter present in water or wastewater. It is expressed as the amount of oxygen consumed from a chemical oxidant in a specific test. It does not differentiate between stable and unstable organic matter and thus does not necessarily correlate with biochemical oxygen demand. Also known as OC and DOC, oxygen consumed and dichromate oxygen consumed, respectively.

chemical precipitation -- (1) Precipitation induced by addition of chemicals. (2) The process of softening water by the addition of lime or lime and soda ash as the precipitants.

chemical toilet -- (1) A commode chair in which a pail containing a chemical solution for deodorizing and liquefying fecal matter is placed immediately beneath the seat. (2) A nonwater-carriage toilet arranged to discharge fecal matter directly into a deodorizing and liquefying chemical solution contained in a watertight tank.

chemical treatment -- Any process involving the addition of chemicals to obtain a desired result.

chlorination -- The application of chlorine to water or wastewater, generally for the purpose of disinfection, but frequently for accomplishing other biological or chemical results.

chlorination chamber -- A detention basin provided primarily to secure the diffusion of chlorine through the liquid. Also called chlorine contact chamber.

chlorine -- An element ordinarily existing as a greenish-yellow gas about 2.5 times as heavy as air. At atmospheric pressure and a temperature of -30.1°F, the gas becomes an amber liquid about 1.5 times as heavy as water. The chemical symbol of chlorine is Cl, its atomic weight is 35.457, and its molecular weight is 70.914.

chlorine contact chamber -- A detention basin provided primarily to secure the diffusion of chlorine throughout the liquid. Also called chlorination chamber.

chlorine demand -- The difference between the amount of chlorine added to the wastewater and the amount of residual chlorine remaining at the end of a specific contact time. The chlorine demand for given water varies with the amount of chlorine applied, time of contact, temperature pH, nature, and amount of impurities in the water.

chlorine residual -- The total amount of chlorine (combined and free available chlorine) remaining in water, sewage, or industrial wastes at the end of specified contact period following chlorination.

clarification -- Any process or combination of processes which reduce the concentration of suspended matter in a liquid.

clarified wastewater -- Wastewater from which most of the settleable solids have been removed by sedimentation. Also called settled wastewater.

clarifier -- A unit which secures clarification. Usually applied to sedimentation tanks or basins. See <u>sedimentation tank.</u>

coagulant -- A chemical added to wastewater or sludge to promote agglomeration and flocculation of suspended solids to induce faster settling or more efficient filtration. Typical coagulants are polyelectrolytes, alum, and ferric chloride.

coagulation basin -- A basin used for the coagulation of suspended or colloidal matter, with or without the addition of a coagulant, in which the liquid is mixed gently to induce agglomeration with a consequent increase in settling velocity of particulates.

colloidal matter -- Finely divided solids which will not settle but may be removed by coagulation or biochemical action or membrane filtration. See <u>colloids.</u>

colloids -- (1) Finely divided solids which will not settle but may be removed by coagulation or biochemical action or membrane filtration; they are intermediate between true solutions and suspensions. (2) In soil physics, discrete mineral particles less than 2 microns (μ) in diameter. (3) Finely divided dispersions of one material, called the dispersed phase; with another, called the dispersion medium. (4) In general, particles of colloidal dimensions are approximately 10 Å to 1 μ in size. Colloidal particles are distinguished from ordinary molecules by their inability to diffuse through membranes that allow ordinary molecules and ions to pass freely.

color -- Colors in water are usually due to decomposition of organic matter of vegetable or soil origin. Color caused by suspended matter is referred to as "apparent color"; color due to colloid vegetable or organic extracts is called "true color."

combined residual chlorination -- The application of chlorine to water or wastewater to produce, with the natural or added ammonia or with certain organic nitrogen compounds, a combined chlorine residual.

comminuted solids -- Solids which have been divided into fine particles.

comminuting screen -- A mechanically operated device for screening wastewater and cutting the screenings into particles sufficiently fine to pass through the screen openings.

comminution -- The process of cutting and screening solids contained in wastewater flow before it enters the flow pumps or other units in the treatment plant.

comminutor -- A device for catching and shredding heavy solid matter in the primary stage of waste treatment.

concentration -- (1) The amount of a given substance dissolved in a unit volume of solution. (2) The process of increasing the dissolved solids per unit volume of solution, usually by evaporation of the liquid.

dechlorination -- The partial or complete reduction of residual chlorine in a liquid by any chemical or physical process.

decomposition -- The breakdown of complex material into simpler substances by chemical or biological means.

decomposition of wastewater -- (1) The breakdown of organic matter in wastewater by bacterial action, either aerobic or anaerobic. (2) Transformation of organic or inorganic materials contained in wastewater through the action of chemical or biological processes.

defoamant -- A material having low compatibility with foam and a low surface tension. Defoamants are used to control, prevent, or destroy various types of foam, the most widely used being silicone defoamers. A droplet of silicone defoamant which contacts a bubble of foam will cause the bubble to undergo a local and drastic reduction in film strength, thereby breaking the film. Unchanged, the defoamant continues to contact other bubbles, thus breaking up the foam. A valuable property of most defoamants is their effectiveness in extremely low concentration. In addition to silicones, defoamants for special purposes are based on polyamides, vegetable oils, and stearic acid.

denitrification -- (1) Chemically bound oxygen in the form of either nitrates or nitrites is stripped away for use by microorganisms. This produces nitrogen gas which can bring up floc in the final sedimentation process. It is an effective method of removing nitrogen from wastewater. (2) A biological process in which gaseous nitrogen is produced from nitrite and nitrate.

depth of side water -- The depth of a liquid measured along the inside of the vertical exterior wall of a tank.

detention time -- The theoretical time required to displace the contents of a tank or unit at a given rate of discharge (volume divided by rate of discharge).

dewatering -- Any process of water removal or concentration of a sludge slurry, as by filtration, centrifugation, or drying. (A dewatering method is any process which will concentrate the sludge solids to at least 15 percent solids by weight.)

diffused air -- A technique by which air under pressure is forced into sewage in an aeration tank. The air is pumped down into the sewage through a pipe and escapes out through holes in the side of the pipe.

diffused-air aeration -- Aeration produced in a liquid by air passed through a diffuser.

diffusion aerator -- An aerator that blows air under low pressure through submerged porous plates, perforated pipes, or other devices so that small air bubbles rise through the water or wastewater continuously.

digested sludge -- Sludge digested under either aerobic or anaerobic conditions until the volatile content has been reduced to the point at which the solids are relatively nonputrescible and inoffensive.

dilution -- Disposal of wastewater or treated effluent by discharging it into a stream or body of water.

discharge -- (1) As applied to a stream or conduit, the rate of flow or volume of water flowing in the stream or conduit at a given place and within a given period of time. (2) The passing of water or other liquid through an opening or along a conduit or channel. (3) The rate of flow of water, silt, or other mobile substance which emerges from an opening, pump, or turbine, or passes along a conduit or channel, usually expressed as cubic feet per second, gallons per minute, or million gallons per day.

disinfectant -- A substance used for disinfection.

disinfected wastewater -- Wastewater to which chlorine or other disinfecting agents has been added, during or after treatment, to destroy pathogenic organisms.

disinfection -- The killing of the larger portion of microorganisms in or on a substance with the probability that all pathogenic bacteria are killed by the agent used.

dissolved oxygen (DO) -- The oxygen dissolved in water, wastewater, or other liquid, usually expressed in milligrams per litre, parts per million, or percent of saturation.

dissolved solids -- Theoretically, the anhydrous residues of the dissolved constituents in water. Actually, the term is defined by the method used in determination. In water and wastewater treatment the Standard Methods tests are used.

ditch -- A small artificial open channel or waterway constructed through earth or rock to convey water.

domestic wastewater -- Wastewater derived principally from dwellings, business buildings, institutions, and the like. It may or may not contain groundwater, surface water, or storm water.

dose -- (1) The quantity of substance applied to a unit quantity of liquid for treatment purposes. It can be expressed in terms of either volume or weight, e.g., pounds per million gallons, parts per million, grains per gallon, milligrams per litre, or grams per cubic metre. (2) Generally, a quantity of material applied to obtain a specific effect.

effective size -- The diameter of the particles, spherical in shape, equal in size, and arranged in a given manner, of a hypothetical sample of granular material that would have the same transmission constant as the actual material under consideration.

efficiency -- (1) The relative results obtained in any operation in relation to the energy or effort required to achieve such results. (2) The ratio of the total output to the total input, expressed as a percentage.

effluent -- (1) A liquid which flows out of a containing space. (2) Wastewater or other liquid, partially or completely treated, or in its natural state, flowing out of a reservoir, basin, treatment plant or industrial treatment plant, or part thereof.

effluent stream -- A stream or stretch of stream which receives water from groundwater in the zone of saturation. The water surface of such a stream stands at a lower level than the water table or piezometric surface of the groundwater body from which it receives water.

endogenous respiration -- An auto-oxidation of cellular material, which takes place in the absence of assimilable organic material, to furnish energy required for the replacement of protoplasm.

environment -- The physical environment of the world consisting of the atmosphere, the hydrosphere, and the lithosphere.

environmental pollution -- The presence of any foreign substance or interference (organic, inorganic, radiological, acoustic, or biological) in the environment (water, air, or land) which tends to degrade its quality so as to constitute a hazard or impair the usefulness of environmental resources.

equalization -- A process by which variations in flow and composition of a waste stream are averaged in an equalizing unit.

equalizing basin -- A holding basin in which variations in flow and composition of a liquid are averaged. Also called balancing reservoir.

eutrophication -- (1) The normally slow aging process by which a lake evolves into marsh and ultimately becomes completely filled with detritus and disappears. (2) The intentional or unintentional enrichment of water.

evaporation -- (1) The process by which water becomes a vapor at a temperature below the boiling point. (2) The quantity of water that is evaporated; the rate is expressed in depth of water, measured as liquid water, removed from a specified surface per unit of time, generally in inches or centimetres per day, month, or year.

evaporation rate -- The quantity of water, expressed in terms of depth of liquid water, evaporated from a given water surface per unit of time. It is usually expressed in inches depth per day, month, or year.

evapotranspiration -- Water withdrawn from soil by evaporation and/or plant transpiration. Considered synonymous with consumptive use.

evapotranspiration potential -- Water loss that would occur if there never was a deficiency of water in the soil for use by vegetation.

extended aeration -- A modification of the activated sludge process which provides for aerobic sludge digestion within the aeration system. The concept envisages the stabilization of organic matter under aerobic conditions, disposal of the end products into the air as gases, with the plant effluent in the form of finely divided suspended and soluble matter.

facultative anaerobic bacteria -- Bacteria which can adapt to growth in the presence, as well as in the absence, of oxygen. May be referred to as facultative bacteria.

filter -- A device or structure for removing solid or colloidal material, usually of a type that cannot be removed by sedimentation, from water, wastewater, or other liquid. The liquid is passed through a filtering medium, usually a granular material but sometimes finely woven cloth, unglazed procelain, or specially prepared paper. There are many types of filters used in water or wastewater treatment. See trickling filter.

filter bed -- (1) A type of bank revetment consisting of layers of filtering medium of which the particles gradually increase in size from the bottom upward. Such a filter allows the groundwater to flow freely, but it prevents even the smallest soil particles from being washed out. (2) A tank for water filtration having a false bottom covered with sand, as a rapid sand filter. (3) A pond with sand bedding, as a sand filter of slow sand filter.

filtering medium -- (1) Any material through which water, wastewater, or other liquid is passed for the purpose of purification, treatment, or conditioning. (2) A cloth or metal material of some appropriate design used to intercept sludge solids in sludge filtration.

filtrate -- The liquid which has passed through a filter.

filtration -- The process of passing a liquid through a filtering medium (which may consist of granular material, such as sand, magnetite, or diatomaceous earth, finely woven cloth, unglazed porcelain, or specially prepared paper) for the removal of suspended or colloidial matter.

final effluent -- The effluent from the final treatment unit of a wastewater treatment plant.

final sedimentation -- The separation of solids from wastewater in a final settling tank.

final sedimentation tank -- A tank through which the effluent from a trickling filter or an aeration or contact-aeration tank is passed to remove the settleable solids. Also called final settling basin. See sedimentation tank.

final settling tank -- A tank through which the effluent from a trickling filter or an aeration or contact-aeration tank is passed to remove the settleable solids. Also called final settling basin. See sedimentation tank.

floc -- Small gelatinous masses formed in a liquid by the reaction of a coagulant added thereto, through biochemical processes, or by agglomeration.

flocculating tank -- A tank used for the formation of floc by the gentle agitation of liquid suspensions, with or without the aid of chemicals.

flocculation -- In water and wastewater treatment, the agglomeration of colloidal and finely divided suspended matter after coagulation by gentle stirring by either mechanical or hydraulic means. In biological wastewater treatment where coagulation is not used, agglomeration may be accomplished biologically.

flocculation agent -- A coagulating substance which, when added to water, forms a flocculent precipitate which will entrain suspended matter and expedite sedimentation; examples are alum, ferrous sulfate, and lime.

flocculator -- (1) A mechanical device to enhance the formation of floc in a liquid. (2) An apparatus for the formation of floc in water and wastewater.

flowing-through time -- (1) The time required for a volume of liquid to pass through a basin, identified in terms of the characteristic being measured, such as mean time, modal time, minimum time. (2) The average time required for a small volume of liquid to pass through a basin from inlet to outlet.

flow rate -- The rate at which a substance is passed through a system.

flume -- (1) An open conduit of wood, masonry, or metal constructed on a grade and sometimes elevated. Sometimes called aqueduct. (2) A ravine or gorge with a stream running through it. (3) To transport in a flume, as logs.

foam -- (1) A collection of minute bubbles formed on the surface of a liquid by agitation, fermentation, etc. (2) The frothy substance composed of an aggregation of bubbles on the surface of liquids by violent agitation or by the admission of air bubbles to liquid containing surface-active materials, solid particles, or both.

food-to-microorganism ratio -- An aeration tank loading parameter.

gravity filter -- A rapid sand filter of the open type, the operating level of which is placed near the hydraulic grade line of the influent and through which the water flows by gravity.

grit -- The heavy suspended mineral matter present in water or wastewater, such as sand, gravel, cinders.

grit chamber -- A detention chamber or an enlargement of a sewer designed to reduce the velocity of flow of the liquid to permit the separation of mineral from organic solids by differential sedimentation.

groundwater -- Subsurface water occupying the saturation zone, from which wells and springs are fed. In a strict sense the term applies only to water below the water table. Also called phreatic water, plerotic water.

halogen -- Any one of the chemically related elements--fluorine, chlorine, bromine, iodine, and astatine.

hardness -- A characteristic of water, imparted by salts of calcium, magnesium, and iron such as bicarbonates, carbonates, sulfates, chlorides, and nitrates, that causes curdling of soap and increased consumption of soap, deposition of scale in boilers, damage in some industrial processes, and sometimes objectionable taste. See carbonate hardness.

heavy metals -- Metals that can be precipitated by hydrogen sulfide in acid solution, for example, lead, silver, gold, mercury, bismuth, copper.

hydraulic loading -- The flow (volume per unit time) applied to the surface area of the clarification or biological reacter units (where applicable).

hydraulic surface loading influent -- (1) The flow (volume per unit time) applied to a unit of surface area (square feet), applicable to trickling filter and filtration processes. (2) Wastewater or other liquid--raw or partially treated--flowing into a reservoir, basin, treatment process, or treatment plant.

impervious -- Not allowing, or allowing only with great difficulty, the movement of water; impermeable.

incineration -- Burning sludge to remove the water and reduce the remaining residues to a safe, nonburnable ash. The ash can then be disposed of safety on land, in some waters, or into caves or other underground locations.

industrial wastes -- The liquid wastes from industrial processes, as distinct from domestic or sanitary wastes.

infiltrate -- (1) To filter into. (2) The penetration by a liquid or gas of the pores or interstices.

infiltration -- (1) The flow or movement of water through the interstices of pores of a soil or other porous medium. (2) The quantity of groundwater that leaks into a pipe through joints, porous walls, or breaks. (3) The entrance of water from the ground into a gallery. (4) The absorption of liquid by the soil, either as it falls as precipitation or from a stream flowing over the surface. See percolation.

influent -- Water, wastewater, or other liquid flowing into a reservoir, basin, or treatment plant, or any unit thereof.

inorganic matter -- Chemical substances of mineral origin, or more correctly, not of basically carbon structure.

intake -- (1) The works or structures at the head of a conduit into which water is diverted. (2) The process or operation by which water is absorbed into the ground and added to the saturation zone.

interface -- (1) A stratum of water of varying thickness lying between the fresh water above and ocean water below in certain estuaries. (2) A boundary layer between two fluids such as liquid-liquid or liquid-gas.

ion -- (1) A charged atom, molecule, or radical, the migration of which affects the transport of electricity through an electrolyte or, to a certain extent through a gas. (2) An atom or molecule that has lost or gained one or more electrons. By such ionization it becomes electrically charged. An example is the alpha particle.

ion exchange -- (1) A chemical process involving reversible interchange of ions between a liquid and a solid but no radical change in structure of the solid. (2) A chemical process in which ions from two different molecules are exchanged.

irrigation -- The artificial application of water to lands to meet the water needs of growing plants not met by rainfall.

kinematic viscosity -- Ratio of absolute viscosity, expressed in poises (grams per centimetre second), to the density, in grams per cubic centimetre, at room temperature.

lagoon -- A pond containing raw or partially treated wastewater in which aerobic or anaerobic stabilization occurs.

land disposal -- Disposal of wastewater onto land.

lime -- Any of a family of chemicals consisting essentially of calcium hydroxide made from limestone (calcite) which is composed almost wholly of calcium carbonate or a mixture of calcium and magnesium carbonate.

liquid -- A substance that flows freely. Characterized by free movement of the constituent molecules among themselves, but without the tendency to separate from one another characteristic of gases. Liquid and fluid are often used synonymously, but fluid has the broader significance, including both liquids and gases.

liquor -- Water, wastewater, or any combination; commonly used to designate liquid phase when other phases are present.

loading -- The time rate at which material is applied to a treatment device involving length, area, or volume, or other design factor.

mechanical aeration -- (1) The mixing, by mechanical means, of wastewater and activated sludge in the aeration tank of the activated sludge process to bring fresh surfaces of liquid into contact with the atmosphere. (2) The introduction of atmospheric oxygen into a liquid by the mechanical action of paddle, paddle wheel, spray, or turbine mechanisms.

mechanical aerator -- A mechanical device for the introduction of atmospheric oxygen into a liquid. See mechanical aeration.

microbial activity -- Chemical changes resulting from the metabolism of living organisms. Biochemical action.

microbial film -- A gelatinous film of microbial growth attached to or spanning the interstices of a support medium. Also called biological slime.

microbiology -- Study of very small units of living matter and their processes.

micron -- Unit of length: 10^{-6} meters; 39×10^{-6} in.

microorganism -- Minute organism, either plant or animal, invisible or barely visible to the naked eye.

milligrams per litre -- A unit of the concentration of water or wastewater constituent. It is 0.001 g of the constituent in 1,000 ml of water. It has replaced the unit formerly used commonly, parts per million, to which it is approximately equivalent, in reporting the results of water and wastewater analysis.

minimum flow -- The flow occurring in a stream during the driest period of the year. Also called low flow.

mixed liquor -- A mixture of activated sludge and organic matter undergoing activated sludge treatment in the aeration tank.

mixed-liquor volatile suspended solids (MLVSS) -- The concentration of volatile suspended solids in an aeration basin. It is commonly assumed to equal the biological solids concentration in the basin.

moisture -- Condensed or diffused liquid, especially water.

natural water -- Water as it occurs in its natural state, usually containing other solid, liquid, or gaseous materials in solution or suspension.

neutralization -- Reaction of acid or alkali with the opposite reagent until the concentrations of hydrogen and hydroxyl ions in the solution are approximately equal.

nitrification -- (1) The conversion of nitrogenous matter into nitrates by bacteria. (2) The treatment of a material with nitric acid.

Nitrosomonas -- A genus of bacteria that oxidize ammonia to nitrite.

nonbiodegradable -- Incapable of being broken down into innocuous products by the actions of living beings (especially microorganisms).

nonpotable water -- Water which is unsatisfactory for consumption.

nonsettleable matter -- That suspended matter which does not settle nor float to the surface of water in a period of 1 hr.

nonsettleable solids -- Wastewater matter that will stay in suspension for an extended period of time. Such period may be arbitrarily taken for testing purposes as 1 hr. See suspended solids.

nutrient -- (1) Any substance assimilated by organisms which promotes growth and replacement of cellular constituents. (2) A chemical substance (an element or an inorganic compound, e.g., nitrogen or phosphate) absorbed by a green plant and used in organic synthesis.

odor control -- (1) In water treatment, the elimination or reduction of odors in a water supply by aeration, algae elimination, superchlorination, activated carbon treatment, and other methods. (2) In wastewater treatment, the prevention or reduction of objectionable odors by chlorination, aeration, or other processes or by masking with chemical aerosols.

organic loading -- Pounds of BOD applied per day to a biological reactor.

organic matter -- Chemical substances of animal or vegetable origin, or more correctly, of basically carbon structure, comprising compounds consisting of hydrocarbons and their derivatives.

organic-matter degradation -- The conversion of organic matter to inorganic forms by biological action.

orthophosphate -- An acid or salt containing phosphorus as PO_4.

overflow -- (1) The excess water that overflows the ordinary limits such as the streambanks, the spillway crest, or the ordinary level of a container. (2) To cover or inundate with water or other fluid.

overflow rate -- One of the criteria for the design of settling tanks in treatment plants; expressed in gallons per day per square foot of surface area in the settling tank.

overland runoff -- Water flowing over the land surface before it reaches a definite stream channel or body of water.

oxidation -- The addition of oxygen to a compound. More generally, any reaction which involves the loss of electrons from an atom.

oxidation pond -- A basin used for retention of wastewater before final disposal, in which biological oxidation of organic material is effected by natural or artificially accelerated transfer of oxygen to the water from air.

oxygen demand -- (1) The quantity of oxygen utilized in the biochemical oxidation of organic matter in a specified time, at a specified temperature, and under specified conditions. See BOD.

oxygen saturation -- The maximum quantity of dissolved oxygen that liquid of given chemical characteristics, in equilibrium with the atmosphere, can contain at a given temperature and pressure.

ozone -- Oxygen in molecular form with three atoms of oxygen forming each molecule (O_3).

particle -- Any dispersed matter, solid or liquid, in which the individual aggregates are larger than single small molecules (about 0.0002 mm in diameter), but smaller than about 500 μm in diameter.

particle size -- (1) An expression for the size of liquid or solid particles expressed as the average or equivalent diameter. (2) The sizes of the two screens, either in the U. S. Sieve Series or the Tyler Series between which the bulk of a carbon sample falls, e.g., 8 × 30 means most of the carbon passes a No. 8 screen but is retained on a No. 30 screen.

parts per million -- The number of weight or volume units of a minor constituent present with each one million units of the major constituent of a solution or mixture. Formerly used to express the results of most water and wastewater analyses, but more recently replaced by the ratio milligrams per liter.

pathogens -- Pathogenic or disease-producing organisms.

peak demand -- The maximum momentary load placed on a water or wastewater plant or pumping station or on an electric generating plant or system. This is usually the maximum average load in 1 hr or less, but may be specified as instantaneous or with some other short time period.

peak load -- (1) The maximum average load carried by an electric generating plant or system for a short time period such as 1 hr or less. See peak. (2) The maximum demand for water placed on a pumping station, treatment plant, or distribution system, expressed as a rate. (3) The maximum rate of flow of wastewater to a pumping station or treatment plant. Also called peak demand.

percolation -- (1) The flow or trickling of a liquid downward through a contact or filtering medium. The liquid may or may not fill the pores of the medium. Also called filtration. (2) The movement of flow of water through the interstices or the pores of a soil or other porous medium.

pH -- The reciprocal of the logarithm of the hydrogen-ion concentration. The concentration is the weight of hydrogen ions, in grams, per litre of solution. Neutral water, for example, has a pH value of 7 and a hydrogen-ion concentration of 10-7.

phosphate -- A salt or ester of phosphoric acid.

pit privy -- A privy placed directly over an excavation in the ground.

pollution -- A condition created by the presence of harmful or objectionable material in water.

pollutional load -- (1) The quantity of material in a waste stream that requires treatment or exerts an adverse effect on the receiving system. (2) The quantity of material carried in a body of water that exerts a detrimental effect on some subsequent use of that water.

porous -- Having small passages; permeable by fluids.

potable water -- Water that does not contain objectional pollution, contamination, minerals, or infective agents and is considered satisfactory for domestic consumption.

preaeration -- A preparatory treatment of wastewater consisting of aeration to remove gases, add oxygen, promote flotation of grease, and aid coagulation.

precipitation -- (1) The total measurable supply of water received directly from clouds as rain, snow, hail, or sleet; usually expressed as depth in a day, month, or year, and designated as daily, monthly, or annual precipitation. (2) The process by which atmospheric moisture is discharged onto a land or water surface. (3) The phenomenon that occurs when a substance held in solution in a liquid passes out of solution into solid form.

preliminary treatment -- (1) The conditioning of a waste at its source before discharge, to remove or to neutralize substances injurious to sewers and treatment processes or to effect a partial reduction in load on the treatment process. (2) In the treatment process, unit operations, such as screening and comminution, that prepare the liquor for subsequent major operations.

primary settling tank -- The first settling tank for the removal of settleable soils through which wastewater is passed in a treatment works.

primary sludge -- Sludge obtained from a primary settling tank.

primary treatment -- (1) The first major (sometimes the only) treatment in a wastewater treatment works, usually sedimentation. (2) The removal of a substantial amount of suspended matter but little or no colloidal and dissolved matter.

privy -- A building, either portable or fixed directly to a pit or vault, equipped with seating and used for excretion of bodily wastes.

privy vault -- A concrete or masonry vault that is provided with a cleanout opening and over which is placed a privy building containing seats.

public water supply -- A water supply from which water is available to the people at large or to any considerable number of members of the public indiscriminately.

pumping station -- A station housing relatively large pumps and their accessories. Pump house is the usual term for shelters for small water pumps.

rapid filter -- A rapid sand filter or pressure filter.

rapid sand filter -- A filter for the purification of water, in which water that has been previously treated, usually by coagulation and sedimentation, is passed downward through a filtering medium. The medium consists of a layer of sand, prepared anthracite coal, or other suitable material, usually 24-30 in. thick, resting on a supporting bed of gravel or a porous medium such as carborundum. It is characterized by a rapid rate of filtration, commonly from two to three gallons per minute per square foot of filter area.

raw wastewater -- Wastewater before it receives any treatment.

receiving body of water -- A natural watercourse, lake, or ocean into which treated or untreated wastewater is discharged.

recycling -- An operation in which a substance is passed through the same series of processes, pipes, or vessels more than once.

retention -- That part of the precipitation falling on a drainage area which does not escape as surface streamflow, during a given period. It is the difference between total precipitation and total runoff during the period, and represents evaporation, transpiration, subsurface leakage, infiltration, and, when short periods are considered, temporary surface or underground storage on the area.

returned sludge -- Settled activated sludge returned to mix with incoming raw or primary settled wastewater.

runoff -- (1) That portion of the earth's available water supply that is transmitted through natural surface channels. (2) Total quantity of runoff water during a specified time. (3) In the general sense, that portion of the precipitation which is not absorbed by the deep strata, but finds its way into the streams after meeting the persistent demands of evapotranspiration, including interception and other losses. (4) The discharge of water in surface streams, usually expressed in inches depth on the drainage area, or as volume in such terms as cubic feet or acre-feet. (5) That part of the precipitation which runs off the surface of a drainage area and reaches a stream or other body of water or a drain or sewer.

scum -- (1) The layer or film of extraneous or foreign matter that rises to the surface of a liquid and is formed there. (2) A residue deposited on a container or channel at the water surface. (3) A mass of solid matter that floats on the surface.

secondary settling tank -- A tank through which effluent from some prior treatment process flows for the purpose of removing settleable solids. See sedimentation tank.

secondary wastewater treatment -- The treatment of wastewater by biological methods after primary treatment by sedimentation.

sedimentation -- The process of subsidence and deposition of suspended matter carried by water, wastewater, or other liquids, by gravity. It is usually accomplished by reducing the velocity of the liquid below the point at which it can transport the suspended material. Also called settling. See chemical precipitation.

septicity -- A condition produced by growth of anaerobic organisms.

septicization -- In anaerobic decomposition, the process whereby intensive growths of bacteria with the enzymes secreted by them liquify and gasify solid organic matter.

septic sludge -- Sludge from a septic tank or partially digested sludge from an Imhoff tank or sludge-digestion tank.

septic tank -- A settling tank in which settled sludge is in immediate contact with the wastewater flowing through the tank and the organic solids are decomposed by anaerobic bacterial action.

septic wastewater -- Wastewater undergoing putrefaction under anaerobic conditions.

settleable solids -- (1) That matter in wastewater which will not stay in suspension during a preselected settling period, such as 1 hr, but either settles to the bottom or floats to the top. (2) In the Imhoff cone test, the volume of matter that settles to the bottom of the cone in 1 hr.

settled wastewater -- Wastewater from which most of the settleable solids have been removed by sedimentation. Also called clarified wastewater.

settling -- The process of subsidence and deposition of suspended matter carried by water, wastewater, or other liquids, by gravity. It is usually accomplished by reducing the velocity of the liquid below the point at which it can transport the suspended material. Also called sedimentation. See chemical precipitation.

settling basin -- A basin or tank in which water or wastewater containing settleable solids is retained to remove by gravity a part of the suspended matter. Also called sedimentation basin, sedimentation tank, settling tank.

settling solids -- Solids that are settling in sedimentation tanks or sedimentation chambers and other such tanks that are constructed for the purpose of removing this fraction of suspended solids. See settleable solids.

settling tank -- A basin or bank in which water or wastewater containing settleable solids is retained to remove by gravity a part of the suspended matter. Also called sedimentation basin, sedimentation tank, settling basin.

settling velocity -- The velocity at which subsidence and deposition of the settleable suspended solids in water and wastewater will occur.

sewage -- The spent water of a community. Term now being replaced in technical usage by preferable term wastewater. See wastewater.

sewer -- A pipe or conduit that carries wastewater or drainage water.

sewer gas -- Gas evolved in sewers that results from the decomposition of the organic matter in the wastewater.

short-circuiting -- A hydraulic condition occurring in parts of a tank where the time of travel is less than the flowing-through time.

side water depth -- The depth of water measured along a vertical exterior wall.

skimming -- The process of removing floating grease or scum from the surface of wastewater in a tank.

slimes -- Substances of viscous organic nature, usually formed from microbiological growth.

slow sand filter -- A filter for the purification of water in which water without previous treatment is passed downward through a filtering medium consisting of a layer of sand or other suitable material, usually finer than for a rapid sand filter and from 24 to 40 in. thick.

It is characterized by a slow rate of filtration, commonly 3-6 mgd/acre of filter area.

sludge -- (1) The accumulated solids separated from liquids, such as water or wastewater, during processing, or deposits on bottoms of streams or other bodies of water. (2) The precipitate resulting from chemical treatment, coagulation, or sedimentation of water or wastewater.

sludge volume index (SVI) -- The ratio of the volume in millilitres of sludge settled from a 1000-ml sample in 30 min to the concentration of mixed liquor in milligrams per litre multiplied by 1000.

sodium carbonate -- A salt used in water treatment to increase the alkalinity or pH value of water or to neutralize acidity. Chemical symbol is Na_2-CO_3. Also called soda ash.

solids-retention time -- The average residence time of suspended soils in a biological waste treatment system, equal to the total weight of suspended solids in the system divided by the total weight of suspended solids leaving the system per unit to time (usually per day).

specific gravity -- The ratio of the mass of a body to the mass of an equal volume of water.

specific resistance -- A sludge filterability index, generally expressed as sec^2/g.

spray irrigation -- A method for disposing of some organic wastewaters by spraying them on land, usually from pipes equipped with spray nozzles. This has proved to be an effective way to dispose of wastes from the canning, meat-packing, and sulfite-pulp industries where suitable land is available.

stabilization -- (1) Maintenance at a relatively nonfluctuating level, quantity, flow, or condition. (2) In lime-soda water softening, any process that will minimize or eliminate scale-forming tendencies. (3) In waste treatment, a process used to equalize wastewater flow composition prior to regulated discharge.

stabilization lagoon -- A shallow pond for storage of wastewater before discharge. Such lagoons may serve only to detain and equalize wastewater composition before regulated discharge to a stream, but often they are used for biological oxidation. See <u>stabilization pond.</u>

stabilization pond -- A type of oxidation pond in which biological oxidation of organic matter is effected by natural or artificially accelerated transfer of oxygen to the water from air.

sterilization -- The destruction of all living microorganisms, as pathogenic or saprophytic bacteria, vegetative forms, and spores.

subsoil -- That portion of a normal soil profile underlying the surface. In humid climates it is lower in content of organic matter, lighter in color, usually of finer particles, of denser structure, and of lower fertility than the surface soil. Its depth and physical properties control to a considerable degree the movement of soil moisture. In arid climates there is less difference between surface and subsoil.

sump -- (1) A tank or pit that receives drainage and stores it temporarily, and from which the drainage is pumped or ejected. (2) A tank or pit that receives liquids.

supernatant -- The liquid standing above a sediment or precipitate.

surface evaporation -- Evaporation from the surface of a body of water, moist soil, snow, or ice. Also see evapotranspiration.

suspended solids -- Solids that either float on the surface of, or are in suspension in, water, wastewater, or other liquids, and which are largely removable by laboratory filtering.

tank -- Any artificial receptacle through which liquids pass or in which they are held in reserve or detained for any purpose.

temperature -- (1) The thermal state of a substance with respect to its ability to communicate heat to its environment. (2) The measure of the thermal state on some arbitrarily chosen numerical scale.

tertiary treatment -- A method used to refine the effluents from secondary treatment systems or otherwise increase the removal of pollutants.

total Kjedahl nitrogen -- The sum of free ammonia and of organic compounds which are converted to $(NH_4)_2SO_4$ under the conditions of digestion.

total organic carbon (TOC) -- A measure of the amount of organic material in a water sample expressed in milligrams of carbon per litre of solution.

treated sewage -- Wastewater that has received partial or complete treatment.

trickling filter -- A treatment unit consisting of a material such as broken stone, clinkers, slate, slats, or brush, over which sewage is distributed and applied in drops, films, or spray, from troughs, drippers, moving distributors, or fixed nozzles, and through which it trickles to the underdrains, giving opportunity for the formation of zoological slimes which clarify and oxidize the sewage.

trickling-filter process -- In wastewater treatment, a process in which the liquid from a primary clarifier is distributed on a bed of stones. As the wastewater trickles through to drains underneath, it comes in contact with slime on the stones, by which organic material in the water is oxidized and impurities are reduced.

turbidity -- (1) A condition in water or wastewater caused by the presence of suspended matter, resulting in the scattering and absorption of light rays. (2) A measure of fine suspended matter in liquids. (3) An analytical quantity usually reported in arbitrary turbidity units determined by measurements of light diffraction.

underdrain -- A drain that carries away groundwater or the drainage from prepared beds to which water or wastewater has been applied.

underflow -- (1) The movement of water through a given cross section of permeable rock or earth, possibly under the bed of a stream or a structure. (2) The flow of water under a structure.

viscosity -- The cohesive force existing between particles of a fluid which causes the fluid to offer resistance to a relative sliding motion between particles.

volatile -- Capable of being evaporated at relatively low temperatures.

volatile solids -- The quantity of solids in water, wastewater, or other liquids, lost on ignition of the dry solids at 600°C.

wasted sludge -- The portion of settled solids from the final clarifier that was removed from the wastewater treatment processes and transferred to the solids handling facilities for ultimate disposal.

waste treatment -- Any process to which wastewater or industrial waste is subjected to make it suitable for subsequent use.

waste water -- In a legal sense, water that is not needed or that has been used and is permitted to escape, or that unavoidably escapes from ditches, canals, or other conduits, or reservoirs of the lawful owners of such structures. See <u>wastewater.</u>

wastewater -- The spent water of a community. From the standpoint of source, it may be a combination of the liquid and water-carried wastes from residences, commercial buildings, industrial plants, and institutions, together with any groundwater, surface water, and storm water that may be present. Also referred to as sewage.

wastewater decomposition -- Transformations of organic or inorganic materials contained in wastewater through the action of chemical or biological processes. Also see <u>decomposition of wastewater.</u>

wastewater disposal -- The act of disposing of wastewater by any method (not synonymous with wastewater treatment). Common methods of disposal are: dispersion, dilution, broad irrigation, privy, cesspool.

wastewater facilities -- The structures, equipment, and processes required to collect, carry away, and treat domestic and industrial wastes, and dispose of the effluent.

wastewater lagoon -- An impoundment into which wastewater is discharged at a rate low enough to permit oxidation to occur without substantial nuisance.

wastewater treatment -- Any process to which wastewater is subjected in order to remove or alter its objectional constituents and thus render it less offensive or dangerous. See intermediate treatment, primary treatment.

wastewater treatment works -- (1) An arrangement of devices and structures for treating wastewater, industrial wastes, and sludge. Sometimes used as synonymous with waste treatment plant or wastewater treatment plant. (2) A water pollution control plant.

water -- A transparent, odorless, tasteless liquid, a compound of hydrogen and oxygen, H_2O, freezing at 32°F or 0°C and boiling at 212°F or 100°C, which, in more or less impure state, constitutes rain, oceans, lakes, rivers, and other such bodies; it contains 11.188 percent hydrogen and 88.812 percent oxygen, by weight. It may exist as a solid, liquid, or gas and, as normally found in the lithosphere, hydrosphere, and atmosphere, may have other solid, gaseous, or liquid materials in solution or suspension.

waterborne disease -- A disease caused by organisms or toxic substances carried by water, the most common of which diseases are typhoid fever, Asiatic cholera, dysentery, and other intestinal disturbances.

water closet -- A plumbing fixture, usually a toilet bowl, seat, and water tank, or valved pressure water connection, for carrying off excreta and liquid wastes to a drain pipe connected below, by the agency of flushing water.

water conditioning -- Treatments, exclusive of disinfection, intended to produce a water free of taste, odor, and other undesirable qualities.

water treatment -- The filtration or conditioning of water to render it acceptable for a specific use.

water treatment plant -- That portion of water treatment works intended specifically for water treatment; may include, among other operations, sedimentation, chemical coagulation, filtration, and chlorination. See water treatment works.

weir -- (1) A diversion dam. (2) A device that has a crest and some side containment of known geometric shape, such as a V, trapezoid, or rectangle, and is used to measure flow of liquid. The liquid surface is exposed to the atmosphere. Flow is related to upstream height of water above the crest, to position of crest with respect to downstream water surface, and to geometry of the weir opening.

zooglea -- A jelly-like matrix developed by bacteria, associated with growths in oxidizing beds.

zoogleal matrix -- The floc formed primarily by slime-producing bacteria in the activated sludge process or in biological beds.

APPENDIX A. NATIONAL POLLUTANT DISCHARGE ELIMINATION SYSTEM (NPDES) REGULATIONS AND REQUIREMENTS

NOTE: THIS MATERIAL IS PROVIDED AS A REFERENCE TO THE USER. HOWEVER, AS THIS MATERIAL MAY BECOME DATED, IT IS RECOMMENDED THAT THE USER OBTAIN THE MOST CURRENT REQUIREMENTS FROM THE APPROPRIATE REGULATORY AGENCY.

A-1. PL 92-500

Section 301(a) of PL 92-500 prohibits the discharge of any pollutant into a stream unless the operation is in compliance with provisions of the act relating to effluent limitations, water-quality-related effluent limitations, national standards of performance, toxic- and pretreatment-effluent standards, aquaculture, the NPDES, and permits for dredged or fill material.[1]

The NPDES, established by Section 402 of PL 92-500, is of particular significance to almost any point-source discharge of pollutants. Section 402 established the NPDES permit program whereby the EPA regional administrator may, after opportunity for public hearing, issue a permit for discharge of effluent containing any pollutant or combinations of pollutants. Operators of rest areas are required to obtain a permit for any discharge of wastewater. This permit may be issued by EPA or by the state pollution control agency which has been granted authority to issue such permits under the NPDES program. The EPA Regional Administrator can furnish guidance in regard to NPDES permit procedures and application requirements for the states in his region.

A-2. EFFLUENT LIMITATIONS

The section of an NPDES permit that is of the most interest to the permittee is the section that specifies the effluent limitations. For sewage treatment facilities, the limited parameters include BOD, SS, fecal coliform bacteria, and pH. Secondary treatment requirements for publicly owned treatment facilities have been presented earlier. In addition to the concentration limitations given in Table B1, permits specify weekly and monthly quantitative limitations by weight for BOD and SS. Some states or the EPA may also limit flow to the hydraulic design capacity of the treatment plant, and many states specify the level of residual chlorine in the effluent. If necessary to maintain water-quality standards, other parameters, such as ammonia nitrogen, total Kjehldahl nitrogen, phosphorus, and maintenance DO, may be limited by the permit.

Table A-1. Secondary Treatment Requirements of PL 92-500.

Parameter	30-Day Mean	7-Day Mean
Biochemical oxygen demand (5-day) (arithmetic mean)		
Influent $\geq$ 200 mg/ℓ	30 mg/ℓ	45 mg/ℓ
Influent $\leq$ 200 mg/ℓ	15% of influent	45 mg/ℓ
Suspended solids (arithmetic mean)		
Influent $\geq$ 200 mg/ℓ	30 mg/ℓ	45 mg/ℓ
Influent $\leq$ 200 mg/ℓ	15% of influent	45 mg/ℓ
pH of effluent	$\geq$6.0 , $\leq$9.0	$\geq$6.0 , $\leq$9.0

NOTES: (1) These requirements represent the minimum effluent standards that must be achieved by 1977 by publicly owned facilities.

(2) The pH limitation is applicable only when chemical addition is used for wastewater treatment and/or where industrial sources affect the pH of the discharge.

If a permittee cannot achieve the final effluent limitations for an existing discharge when the permit is issued, a schedule of compliance may be included in the permit to allow a reasonable time for the permittee to make such modifications necessary to achieve the final effluent limitations.

A-3. MONITORING REQUIREMENTS

Monitoring requirements in NPDES permits appear to be without pattern or regulation and dependent on the policy of the EPA regional administrator or the state with NPDES authority. The NPDES regulation requires that major discharges (those with flows greater than 50,000 gpd) be monitored for the parameters limited by the permit, for flow rate, and for parameters having a significant impact on water quality. Monitoring of minor discharges is left to the discretion of the permit-issuing authority. Therefore, it is difficult to predict the frequency of sample collection and analysis for a discharge from a rest area.

The general policy for several states surveyed is to require monitoring of minor discharges that is directed at proper operation and maintenance of the treatment works rather than at proving compliance or noncompliance with effluent limitations. Determination of the major parameters (BOD, SS, and fecal coliform) that must be performed in an analytical laboratory may be required as infrequently as once every 3 months or not at all; while parameters such as chlorine residual, settleable solids, DO in the aeration basin and effluent, flow, and pH may require monitoring on a daily basis. Such daily monitoring will insure frequent and routine inspection of the treatment facility to eliminate or reduce the impact of equipment failure or other malfunctions.

To say that the permittee is not required to monitor an effluent discharge does not relieve him of the responsibility of complying with the effluent limitations at all times; nor does it prevent the permit-issuing authority from monitoring effluent for possible enforcement action. A permittee may want to perform additional monitoring, particularly after startup of a new plant or plant modification, to insure that the effluent from the plant will comply with average and maximum effluent limitations.

Monitoring procedures used for rest-area operations must conform to analytical methods described in: Standard Methods for Examination of Water and Wastewater,[2] ASTM Standards,[3] and Methods for Chemical Analysis of Water and Wastes.[4] A description of test procedures established pursuant to Section 304(g) of PL 92-500 was published in the Federal Register on 16 October 1973. All NPDES permits require that monitoring records be retained for a minimum of 3 yr.

A-4. REFERENCES

1. "National Pollutant Discharge Elimination System," Federal Register, Vol 38, No. 98, May 1973, pp 13,528-13,540.

2. American Public Health Association, Standard Methods for the Examination of Water Wastewater, 13th ed., American Water Works Association and Water Pollution Control Federation, Washington, D. C. 1971.

3. American Society for Testing and Materials, ASTM Standards, Part 23, Water, Atmospheric Analysis, Philadelphia, Pa., 1973.

4. U. S. Environmental Protection Agency, "Methods for Chemical Analysis of Water and Wastes," National Environmental Research Center, Analytical Quality Control Laboratory, Cincinnati, Ohio, 1971.

APPENDIX B. ALTERNATIVE WASTE MANAGEMENT TECHNIQUES FOR BEST PRACTICABLE WASTE TREATMENT

NOTE: THIS MATERIAL IS PROVIDED AS A REFERENCE TO THE USER. HOWEVER, AS THIS MATERIAL MAY BECOME DATED, IT IS RECOMMENDED THAT THE USER OBTAIN THE MOST CURRENT INFORMATION FROM THE U.S. ENVIRONMENTAL PROTECTION AGENCY OR THE APPROPRIATE REGULATORY AGENCY.

Maximum microbiological contaminant levels. The maximum contaminant levels for coliform bacteria, applicable to community water systems and non-community water systems, are as follows:

(a) When the membrane filter technique pursuant to § 141.21(a) is used, the number of coliform bacteria shall not exceed any of the following:

(1) One per 100 milliliters as the arithmetic mean of all samples examined per month pursuant to § 141.21 (b) or (c);

(2) Four per 100 milliliters in more than one sample when less than 20 are examined per month; or

(3) Four per 100 milliliters in more than five percent of the samples when 20 or more are examined per month.

(b) (1) When the fermentation tube method and 10 milliliter standard portions pursuant to § 141.21(a) are used, coliform bacteria shall not be present in any of the following:

(i) More than 10 percent of the portions in any month pursuant to § 141.21 (b) or (c);

(ii) Three or more portions in more than one sample when less than 20 samples are examined per month; or

(iii) Three or more portions in more than five percent of the samples when 20 or more samples are examined per month.

(2) When the fermentation tube method and 100 milliliter standard portions pursuant to § 141.21(a) are used, coliform bacteria shall not be present in any of the following:

(i) More than 60 percent of the portions in any month pursuant to § 141.21 (b) or (c);

(ii) Five portions in more than one sample when less than five samples are examined per month; or

(iii) Five portions in more than 20 percent of the samples when five or more samples are examined per month.

(c) For community or non-community systems that are required to sample at a rate of less than 4 per month, compliance with Paragraphs (a), (b) (1), or (2) shall be based upon sampling during a 3 month period, except that, at the discretion of the State, compliance may be based upon sampling during a one-month period.

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APPENDIX C. METHODS FOR ASSESSING THE PROBABILITY OF COMPLIANCE OF POINT DISCHARGE REST-AREA WASTEWATER-TREATMENT PLANTS

NOTE: THIS MATERIAL IS FURNISHED AS A REFERENCE TO THE USER. IT IS NOT RECOMMENDED FOR USE WITHOUT A SOUND WORKING. KNOWLEDGE IN STATISTICS.

C-1. INTRODUCTION

Rest-area wastewater-treatment plants currently in operation or to be constructed are required to meet the effluent discharge limitations and water-quality standards of PL 92-500 or state effluent limitations whichever is more stringent. Generally, the regulations restrict organic, solid, nutrient, and coliform bacteria contents. Selection of specific criteria is based on the more restrictive of either effluent or water-quality standards. Assessment of compliance by rest-area wastewater treatment plants is dependent on whether or not the treatment system has a point discharge and on the performance of the treatment system. Performance of a treatment system is dependent on configuration of unit processes, adaptability to time-varying hydraulic and organic loads, and loading imposed on the system. The first factor is related to the treatment system employed, and the latter two factors are related to the specific rest-area site.

If the effluent of a rest area is required to be monitored on any basis (i.e., weekly, monthly, etc.) then monitoring shall be performed in accordance with the NPDES requirements discussed in Appendix A. If a rest area is also required to show compliance with the effluent limitations of PL 92-500, then there is a need to determine the probability through the collection and testing of a given number of samples that either compliance or noncompliance with the law can be shown. Therefore, methods for assessing the probability of compliance (or noncompliance) of point discharge rest-area wastewater treatment plants are given herein.

C-2. METHODS FOR ASSESSING PROBABILITY OF COMPLIANCE

Performance of a treatment plant as reflected by constitutive properties of the effluent determines compliance with regulatory criteria. Normally, one of two types of criteria is specified: criteria expressed as the limiting values of a parameter that may not be exceeded by the average of a selected number of consecutive samples taken over a specified time interval or criteria given as a single parametric value that may not be exceeded at any time.

Examples of methods for determining the probability that the performance of a treatment plant will be in compliance with either type of regulatory criteria are given below. These examples are based on the premise that sufficient effluent data exist so that the mean value $\overline{X}$ and the standard deviation S are the best estimators of the parametric mean μ and the parametric standard deviation σ. In both cases, a similar data base is required for evaluation. The effects of time on the data are assumed to be represented in the variation present in the effluent data. The effects of geographic variation and its related factors on compliance preclude examination of multiple sampling points.

C-2-1. Method I. For the first type of criteria, time intervals and limiting values are normally specified (i.e., the requirements of PL 92-500). These specifications and the statistical distribution of effluent sample data can be used to determine probability of compliance for a selected treatment system. Evaluation for compliance under the first type of criteria (Method I) may be made in the following manner.

Assume that the regulatory criteria for a pollutant P is given as follows:

a. P_1, the mean of N_1 grab samples taken over N_1 days, may not exceed C_1.

b. P_2, the mean of N_2 grab samples taken over N_2 days, may not exceed C_2.

If the design period for the treatment system to be evaluated is T^{days}, then the following conditions must be attained for compliance to be obtained:

a. $\text{PROB}\ (P_1 > C_1) < N_1/T$

b. $\text{PROB}\ (P_2 > C_2) < N_2/T$

If the effluent data are obtained for a treatment system in a consecutive time period, then $\overline{X}$ and S may be calculated for the data. The assumption is made that the effluent values are normally distributed and $\overline{X}$ and S represent the best estimators for μ and σ. Since the values P_1 and P_2 are means of effluent values, it is relatively assured that P_1 and P_2 are normally distributed. The standard deviations of P_1 and P_2 may be estimated by $S/N_1^{1/2}$ and $S/N_2^{1/2}$, respectively. The probability that $P_2 > C_1$ or $P_2 > C_2$ can be obtained directly from a cumulative normal distribution table (Table C-1).[1] The probabilities above also represent the probabilities for compliance under the stated conditions. The following example demonstrates the application of this procedure to effluent data.

Example (Method I):

Criteria: (a) $C_1 = 45$, for $N_1 = 7$

(b) $C_2 = 30$, for $N_2 = 30$

Variables: $\overline{X} = 19.5$

$S = 16.8$

$T = 7305$

For compliance

$$\text{PROB } (P_1 > C_1) < \frac{N_1}{T} = \frac{7}{7305} = 0.00096$$

and

$$\text{PROB } (P_1 > C_2) < \frac{N_2}{T} = \frac{30}{7305} = 0.0041$$

The probability for $P_1 > C_1$ is given by

$$\frac{C_1 - \overline{X}}{S(N_1)^{-1/2}} = \frac{45 - 19.5}{16.8(7)^{-1/2}} = 4.02 \, , \quad P < 0.000032 \text{ (from Table C-1)}$$

and similarly for $P_2 > C_2$, the probability is given by

$$\frac{C_2 - \overline{X}}{S(N_2)^{-1/2}} = \frac{30 - 19.5}{16.8(30)^{-1/2}} = 3.42 \, , \quad P < 0.0003 \text{ (from Table C-1}$$

Standard deviation = 3.42 then area under curve = 0.4997
and P = 0.5 - 0.4997 = 0.0003)

Conclusion: This particular site will comply with regulatory criteria.

C-2-2. Method II. Evaluation for compliance under the second

Table C-1. Areas of the Normal Curve.[1]

Standard Deviation Units	0	1	2	3	4	5	6	7	8	9	Standard Deviation Units
0.0	0.0000	0.0040	0.0080	0.0120	0.0160	0.0199	0.0239	0.0279	0.0319	0.0359	0.0
0.1	0.0398	0.0438	0.0478	0.0517	0.0557	0.0596	0.0636	0.0675	0.0714	0.0753	0.1
0.2	0.0793	0.0832	0.0871	0.0910	0.0948	0.0987	0.1026	0.1064	0.1103	0.1141	0.2
0.3	0.1179	0.1217	0.1255	0.1293	0.1331	0.1368	0.1406	0.1443	0.1480	0.1517	0.3
0.4	0.1554	0.1591	0.1628	0.1664	0.1700	0.1736	0.1772	0.1808	0.1844	0.1879	0.4
0.5	0.1915	0.1950	0.1985	0.2019	0.2054	0.2088	0.2123	0.2157	0.2190	0.2224	0.5
0.6	0.2257	0.2291	0.2324	0.2357	0.2389	0.2422	0.2454	0.2486	0.2517	0.2549	0.6
0.7	0.2580	0.2611	0.2642	0.2673	0.2704	0.2734	0.2764	0.2794	0.2823	0.2852	0.7
0.8	0.2881	0.2910	0.2939	0.2967	0.2995	0.3023	0.3051	0.3078	0.3106	0.3133	0.8
0.9	0.3159	0.3186	0.3212	0.3238	0.3264	0.3289	0.3315	0.3340	0.3365	0.3389	0.9
1.0	0.3413	0.3438	0.3461	0.3485	0.3508	0.3531	0.3554	0.3577	0.3599	0.3621	1.0
1.1	0.3643	0.3665	0.3686	0.3708	0.3729	0.3749	0.3770	0.3790	0.3810	0.3830	1.1
1.2	0.3849	0.3869	0.3888	0.3907	0.3925	0.3944	0.3962	0.3980	0.3997	0.4015	1.2
1.3	0.4032	0.4049	0.4066	0.4082	0.4099	0.4115	0.4131	0.4147	0.4162	0.4177	1.3
1.4	0.4192	0.4207	0.4222	0.4236	0.4251	0.4265	0.4279	0.4292	0.4306	0.4319	1.4
1.5	0.4332	0.4345	0.4357	0.4370	0.4382	0.4394	0.4406	0.4418	0.4429	0.4441	1.5
1.6	0.4452	0.4463	0.4474	0.4484	0.4495	0.4505	0.4515	0.4525	0.4535	0.4545	1.6
1.7	0.4554	0.4564	0.4573	0.4582	0.4591	0.4599	0.4608	0.4616	0.4625	0.4633	1.7
1.8	0.4641	0.4649	0.4656	0.4664	0.4671	0.4678	0.4686	0.4693	0.4699	0.4706	1.8
1.9	0.4713	0.4719	0.4726	0.4732	0.4738	0.4744	0.4750	0.4756	0.4761	0.4767	1.9
2.0	0.4772	0.4778	0.4783	0.4788	0.4793	0.4798	0.4803	0.4808	0.4812	0.4817	2.0
2.1	0.4821	0.4826	0.4830	0.4834	0.4838	0.4842	0.4846	0.4850	0.4854	0.4857	2.1
2.2	0.4861	0.4864	0.4868	0.4871	0.4875	0.4878	0.4881	0.4884	0.4887	0.4890	2.2
2.3	0.4893	0.4896	0.4898	0.4901	0.4904	0.4906	0.4909	0.4911	0.4913	0.4916	2.3
2.4	0.4918	0.4920	0.4922	0.4925	0.4927	0.4929	0.4931	0.4932	0.4934	0.4936	2.4

(Continued)

Table C-1. (Concluded).

Standard Deviation Units	0	1	2	3	4	5	6	7	8	9	Standard Deviation Units
2.5	0.4938	0.4940	0.4941	0.4943	0.4945	0.4946	0.4948	0.4949	0.4951	0.4952	2.5
2.6	0.4953	0.4955	0.4956	0.4957	0.4959	0.4960	0.4961	0.4962	0.4963	0.4964	2.6
2.7	0.4965	0.4966	0.4967	0.4968	0.4969	0.4970	0.4971	0.4972	0.4973	0.4974	2.7
2.8	0.4974	0.4975	0.4976	0.4977	0.4977	0.4978	0.4979	0.4979	0.4980	0.4981	2.8
2.9	0.4981	0.4982	0.4982	0.4983	0.4984	0.4984	0.4985	0.4985	0.4986	0.4986	2.9
3.0	0.4987	0.4987	0.4987	0.4988	0.4988	0.4989	0.4989	0.4989	0.4990	0.4990	3.0
3.1	0.4990	0.4991	0.4991	0.4991	0.4992	0.4992	0.4992	0.4992	0.4993	0.4993	3.1
3.2	0.4993	0.4993	0.4994	0.4994	0.4994	0.4994	0.4994	0.4995	0.4995	0.4995	3.2
3.3	0.4995	0.4995	0.4995	0.4996	0.4996	0.4996	0.4996	0.4996	0.4996	0.4997	3.3
3.4	0.4997	0.4997	0.4997	0.4997	0.4997	0.4997	0.4997	0.4997	0.4997	0.4998	3.4
3.5	0.499767										
3.6	0.499869										
3.7	0.499892										
3.8	0.499928										
3.9	0.499952										
4.0	0.499968										
4.1	0.499979										
4.2	0.499987										
4.3	0.499991										
4.4	0.499995										
4.5	0.499997										
4.6	0.499998										
4.7	0.499999										
4.8	0.499999										
4.9	0.500000										

type of criteria (Method II) may be made in the following manner. Assume that the regulatory value for a pollutant P is given as P_1, and that the effluent value for pollutant P may not exceed C_1. In an approach similar to that used in the first example, this means that the condition $P(P_1 > C_1) < N_1/T$ must be met. This approach assumes a daily sampling interval for monitoring requirements and, in general, requires the same data base for evaluation as Method I. The following example demonstrates the application of this method.

Example (Method II):

Criterion: $C_1 = 15$

Variables: $\overline{X} = 8.0$

$S = 3.0$

$T = 5000$

$N_1 = 1$

For compliance, $P(P > C_1) < N_1/T = 1/5000 = 0.0002$. The probability, assuming a normal distribution, for $P(P < C_1)$ is given by

$$\frac{C_1 - \overline{X}}{S} = \frac{15 - 8.0}{3.0} = 2.33 \ , \quad P < 0.0099$$

Conclusion: Since $0.0099 > 0.0002$ $P(P < C_1) > N_1/T$, compliance will not be attained.

C-2-3. References.

1. Rohlf, F. J. and Solcal, R. R., Statistical Tables, W. H. Freeman, San Francisco, 1969.

APPENDIX D. CONVERSION FACTORS FOR UNITS OF MEASUREMENT; DISSOLVED-OXYGEN SOLUBILITY DATA; PHYSICAL PROPERTIES OF WATER; CHEMICAL ELEMENTS AND SUBSTANCES; SPECIFIC WEIGHT

NOTE: METRIC EQUIVALENTS WERE NOT PROVIDED WITHIN THE TEXT OF THIS REPORT AS THIS RESEARCH WAS INITIATED BEFORE THIS REQUIREMENT BECAME OPERATIONAL.

D-1. CONVERSION FACTORS

U. S. customary, metric (SI), and British units of measurement.

Length

1 in. = 2.54 cm

1 ft = 0.3048 m

1 yd = 0.915 m

1 mile (U. S. statute) = 1.6093 km

1 μ = 39.37 μin.

1 mm = 0.0394 in.

1 cm = 0.394 in.

1 m = 1.092 yd

1 km = 0.6214 mile

Area

1 in.^{2} = 6.4516 cm^2

1 ft^2 = 0.0929 m^2

1 yd^2 = 0.8361 m^2

1 mile^2 (U. S. statute) = 2.59 km^2

1 acre = 4046.8 m^2

1 cm^2 = 0.155 in.^{2}

1 m^2 = 1.196 yd^2

1 ha = 2.471 acres

1 km^2 = 0.386 mile^2

Volume

1 in.^{3} = 16.39 cm^3

1 ft^3 = 0.0283 m^3

1 yd^3 = 0.7654 m^3

1 acre-ft = 1233.5 m^3

1 U. S. gal = 3.79 ℓ

1 U. K. gal = 4.55 ℓ

1 cm^2 = 0.061 $in.^3$

1 m^3 = 1.308 yd^3

1 ℓ = 0.26 U. S. gal = 0.22 U. K. gal

Mass

1 oz (avoirdupois) = 28.35 g

1 lb (avoirdupois) = 0.4536 kg

ton (short, 2000 lb) = 907.185 kg

ton (long, 2240 lb) = 1016.05 kg

1 g = 0.0352 oz

1 kg = 2.2 lb

Pressure

1 psi = 6894.757 Pa

1 psf = 47.88026 Pa

1 atm = 1 kg/cm^2

1 kg/cm^2 = 14.20 $lb/in.^2$

Velocity

1 in./sec = 2.54 cm/sec

1 ft/sec = 0.3048 m/sec

1 cm/sec = 0.394 in./sec

1 m/sec = 1.094 yd/sec

Power

1 hp = 0.7457 kw

1 Btu (Int. Table) = 1055.056 J

1 kw = 1.341 hp

Miscellaneous

$1\ ft^3/sec = 28.3\ \ell/sec$

$1\ mgd\ (U.\ S.) = 3780\ m^3/day$

$1\ mgd\ (U.\ K.) = 4550\ m^3/day$

$1\ lb/ft^3 = 16.0185\ kg/m^3$

$1\ mg/\ell = 0.000133\ oz/gal\ (U.\ S.)$

$1\ cp = 0.001\ Pa\text{-}sec$

$1\ lb\ BOD/acre/day = 1.12\ kg\ BOD/ha/day$

$1°F = 5/9\ °C$*

* To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following equation: $C = (5/9)(F - 32)$.

D-2. PHYSICAL PROPERTIES OF WATER

Temperture °F	Specific Weight, γ lb/ft^3	Density, ρ slug/ft^3	Viscosity, $\mu \times 10^5$ lb-sec/ft^2	Kinematic Viscosity, $\nu \times 10^5$ ft^2/sec	Surface Tension, σ lb/ft	Vapor Pressure, $\rho\upsilon$ psia
32	62.42	1.940	3.746	1.931	0.00518	0.09
40	62.43	1.940	3.229	1.664	0.00514	0.12
50	62.41	1.940	2.735	1.410	0.00509	0.18
60	62.37	1.938	2.359	1.217	0.00504	0.26
70	62.30	1.936	2.050	1.059	0.00498	0.36
80	62.22	1.934	1.799	0.930	0.00492	0.51
90	62.11	1.931	1.595	0.826	0.00486	0.70
100	62.00	1.927	1.424	0.739	0.00480	0.95

D-3. DISSOLVED-OXYGEN SOLUBILITY DATA

Temperature °C	Dissolved Oxygen, mg/ℓ — Chloride Concentration, mg/ℓ: 0	5,000	10,000	15,000	20,000
0	14.62	13.79	12.97	12.14	11.32
1	14.23	13.41	12.61	11.82	11.03
2	13.84	13.05	12.28	11.52	10.76

(Continued)

Dissolved Oxygen, mg/ℓ					
Temperature °C	Chloride Concentration, mg/ℓ				
	0	5,000	10,000	15,000	20,000
3	13.48	12.72	11.98	11.24	10.50
4	13.13	12.41	11.69	10.97	10.25
5	12.80	12.09	11.39	10.70	10.01
6	12.48	11.79	11.12	10.45	9.78
7	12.17	11.51	10.85	10.21	9.57
8	11.87	11.24	10.61	9.98	9.36
9	11.59	10.97	10.36	9.76	9.17
10	11.33	10.73	10.13	9.55	8.98
11	11.08	10.49	9.92	9.35	8.80
12	10.83	10.28	9.72	9.17	8.62
13	10.60	10.05	9.52	8.98	8.46
14	10.37	9.85	9.32	8.80	8.30
15	10.15	9.65	9.14	8.63	8.14
16	9.95	9.46	8.96	8.47	7.99
17	9.74	9.26	8.78	8.30	7.84
18	9.54	9.07	8.62	8.15	7.70
19	9.35	8.89	8.45	8.00	7.56
20	9.17	8.73	8.30	7.86	7.42
21	8.99	8.57	8.14	7.71	7.28
22	8.83	8.42	7.99	7.57	7.14
23	8.68	8.27	7.85	7.43	7.00
24	8.53	8.12	7.71	7.30	6.87
25	8.38	7.96	7.56	7.15	6.74
26	8.22	7.81	7.42	7.02	6.61
27	8.07	7.67	7.28	6.88	6.49
28	7.92	7.53	7.14	6.75	6.37
29	7.77	7.39	7.00	6.62	6.25
30	7.63	7.25	6.86	6.49	6.13

D-4. CHEMICAL ELEMENTS AND SUBSTANCES

Element	Symbol	Atomic Number	Atomic Weight	Valence
Aluminum	Al	13	26.98	3
Bromine	Br	35	79.92	1,3,5,7
Calcium	Ca	20	40.08	2
Carbon	C	6	12.01	2,4
Chlorine	Cl	17	35.46	1,3,5,7
Chromium	Cr	24	52.01	2,3,6
Cobalt	Co	27	58.94	2,3
Copper	Cu	29	63.54	1,2

(Continued)

Element	Symbol	Atomic Number	Atomic Weight	Valence
Fluorine	F	9	19.00	1
Hydrogen	H	1	1.008	1
Iodine	I	53	126.92	1,3,5,7
Iron (Ferrum)	Fe	26	55.85	2,3
Lead (Plumbum)	Pb	82	207.21	2,4
Magnesium	Mg	12	24.32	2
Manganese	Mn	25	54.93	2,3,4,6,7
Mercury (Hydragyrum)	Hg	80	200.61	1,2
Nickel	Ni	28	58.69	2,3
Nitrogen	N	7	14.01	3,5
Oxygen	O	8	16.00	2
Phosphorus	P	15	30.98	3,5
Platinum	Pt	78	195.23	2,4
Potassium (Kalium)	K	19	39.10	1
Silicon	Si	14	28.09	4
Silver (Argentum)	Ag	47	107.88	1
Sodium (Natrium)	Na	11	23.00	1
Strontium	Sr	38	87.63	2
Sulfur	S	16	32.07	2,4,6
Tin (Stannum)	Sn	50	118.70	2,4
Zinc	Zn	30	65.38	2

D-5. SPECIFIC WEIGHT

Water	1.00	62.5
Salt water	1.02	64
Ice	0.91	57.5
Snow	0.40	25
Gasoline	0.70	44
Diesel oil	0.90	56
Wood	0.68	42
Feces	1.02	64
Dust (loose)	1.20	75
Earth (loose)	1.60	100
Sand	2.80	175

1 cu ft of water = 62.43 lb
1 gal of water (U. S.) = 8.34 lb
1 m^3 of air = 1.2 kg
1 kg/ℓ = 62.5 lb/cu ft
1 lb/cu ft = 16.02 kg/m^3
= 0.012 tons long/cu yd

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FEDERALLY COORDINATED PROGRAM OF HIGHWAY RESEARCH AND DEVELOPMENT (FCP)

The Offices of Research and Development of the Federal Highway Administration are responsible for a broad program of research with resources including its own staff, contract programs, and a Federal-Aid program which is conducted by or through the State highway departments and which also finances the National Cooperative Highway Research Program managed by the Transportation Research Board. The Federally Coordinated Program of Highway Research and Development (FCP) is a carefully selected group of projects aimed at urgent, national problems, which concentrates these resources on these problems to obtain timely solutions. Virtually all of the available funds and staff resources are a part of the FCP, together with as much of the Federal-aid research funds of the States and the NCHRP resources as the States agree to devote to these projects.*

FCP Category Descriptions

1. Improved Highway Design and Operation for Safety

Safety R&D addresses problems connected with the responsibilities of the Federal Highway Administration under the Highway Safety Act and includes investigation of appropriate design standards, roadside hardware, signing, and physical and scientific data for the formulation of improved safety regulations.

2. Reduction of Traffic Congestion and Improved Operational Efficiency

Traffic R&D is concerned with increasing the operational efficiency of existing highways by advancing technology, by improving designs for existing as well as new facilities, and by keeping the demand-capacity relationship in better balance through traffic management techniques such as bus and carpool preferential treatment, motorist information, and rerouting of traffic.

3. Environmental Considerations in Highway Design, Location, Construction, and Operation

Environmental R&D is directed toward identifying and evaluating highway elements which affect the quality of the human environment. The ultimate goals are reduction of adverse highway and traffic impacts, and protection and enhancement of the environment.

4. Improved Materials Utilization and Durability

Materials R&D is concerned with expanding the knowledge of materials properties and technology to fully utilize available naturally occurring materials, to develop extender or substitute materials for materials in short supply, and to devise procedures for converting industrial and other wastes into useful highway products. These activities are all directed toward the common goals of lowering the cost of highway construction and extending the period of maintenance-free operation.

5. Improved Design to Reduce Costs, Extend Life Expectancy, and Insure Structural Safety

Structural R&D is concerned with furthering the latest technological advances in structural designs, fabrication processes, and construction techniques, to provide safe, efficient highways at reasonable cost.

6. Prototype Development and Implementation of Research

This category is concerned with developing and transferring research and technology into practice, or, as it has been commonly identified, "technology transfer."

7. Improved Technology for Highway Maintenance

Maintenance R&D objectives include the development and application of new technology to improve management, to augment the utilization of resources, and to increase operational efficiency and safety in the maintenance of highway facilities.

* The complete 7-volume official statement of the FCP is available from the National Technical Information Service (NTIS), Springfield, Virginia 22161 (Order No. PB 242057, price $45 postpaid). Single copies of the introductory volume are obtainable without charge from Program Analysis (HRD-2), Offices of Research and Development, Federal Highway Administration, Washington, D.C. 20590.

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