Environment Canada
Environmental Protection Service

Environnement Canada
Service de la protection de l'environnement

Design and Selection of Small Wastewater Treatment Systems

SPÉCIMEN

CONSULTATION SEULEMENT | ONLY

Economic and Technical Review
Report EPS 3-WP-80-3

Canada

ter Pollution Control Directorate
ch 1980

ENVIRONMENTAL PROTECTION SERVICE REPORT SERIES

Economic and Technical Review Reports relate to state-of-the-art reviews, library surveys, industrial inventories, and their associated recommendations where no experimental work is involved. These reports are undertaken either by an outside agency or by the staff of the Environmental Protection Service.

Other categories in the EPS series include such groups as Regulations, Codes, and Protocols; Policy and Planning; Technology Development; Surveillance; Training Manuals; Briefs and Submissions to Public Inquiries; and, Environmental Impact and Assessment.

Inquiries pertaining to Environmental Protection Service Reports should be directed to the Environmental Protection Service, Environment Canada, Ottawa, Ontario, Canada, K1A 1C8.

SÉRIE DE RAPPORTS DU SERVICE DE LA PROTECTION DE L'ENVIRONNEMENT

Les rapports d'analyse économique et technique portent sur les revues de l'état des connaissances, les relèves de bibliothèques, les inventaires industriels ainsi que leurs recommandations connexes dans les cas où celles-ci n'impliquent aucune recherche expérimentale. Ces analyses sont entreprises, soit par un organisme extérieur, soit par le personnel du Service de la protection de l'environnement.

Les autres catégories de la série de rapports du S.P.E. comprennent les groupes suivants: règlements, codes et accords; politique et planification; développement technologique; surveillance; guides de formation; rapports et exposés à des enquêtes publiques et impacts environnementaux.

Les demandes relatives aux rapports du Service de la protection de l'environnement doivent être adressées au Service de la protection de l'environnement, Environnement Canada, Ottawa, Ontario, K1A 1C8, Canada.

7000308H

DESIGN AND SELECTION OF SMALL WASTEWATER TREATMENT SYSTEMS

by

S.A. Ross
Abatement and Compliance Branch

and

P.H.M. Guo and B.E. Jank
Technology Development Branch

Water Pollution Control Directorate
Environmental Protection Service
Environment Canada

Report No. EPS 3-WP-80-3
March 1980

Cat No En 43-3/80-3E
ISBN 0-662-10863-9

Second printing February 1983

ABSTRACT

This report provides general information on the design and selection of wastewater systems with capacities for populations up to 2500. This information is intended for use by individuals with limited experience in wastewater treatment and disposal as a source of available alternatives in small systems, as well as an outline of the steps and procedures to undertake when selecting a particular wastewater management scheme for a small community.

Material covered includes: measurement and estimation of wastewater flows; physical, chemical and biological characteristics of domestic wastewater; on-site wastewater treatment and disposal processes; central wastewater collection and treatment systems; operating problems associated with small treatment systems; disposal of liquid effluents and waste sludges; and procedures employed in the selection and approval of wastewater systems. A case history is provided to illustrate the selection procedures discussed within the text.

RÉSUMÉ

Le présent rapport fournit des renseignements généraux sur la conception et le choix de systèmes d'épuration des eaux usées pouvant desservir jusqu'à 2 500 personnes, à l'intention de ceux qui sont peu familiers avec l'évacuation et l'épuration des eaux résiduaires. On y décrit les différents types de petites installations disponibles et on donne un aperçu des étapes et des règles à suivre dans le choix d'un système d'épuration pour une petite collectivité.

Le rapport traite des points suivants: la mesure et l'évaluation du débit des eaux usées; les caractéristiques physiques, chimiques et biologiques des eaux domestiques; les moyens d'évacuer et de traiter les eaux usées sur place; les installations centrales de collecte et d'épuration; les difficultés de fonctionnement propres aux petites installations; l'élimination des effluents liquides et des boues résiduaires; et la marche à suivre pour choisir et adopter un mode d'épuration. Un exemple illustre la marche à suivre pour effectuer un tel choix.

TABLE OF CONTENTS

		Page
ABSTRACT		i
1	INTRODUCTION	1
1.1	Scope	1
1.2	Objectives	2
1.3	Organization	2
2	WASTEWATER CHARACTERIZATION	4
2.1	Wastewater Monitoring Program	4
2.1.1	Development of Collection Systems Flow Sheet	4
2.1.2	Selection of Monitoring Stations	5
2.1.3	Coordination of Laboratory Support	5
2.1.4	Selection of Monitoring Equipment	5
2.2	Flow Measurement	6
2.2.1	Fill and Draw Devices	6
2.2.2	Depth Flow Measurement	8
2.2.3	Measuring Weirs	13
2.2.4	Measuring Flumes	16
2.2.5	Summary	20
2.3	Wastewater Sampling	20
2.3.1	Types of Samples	21
2.3.2	Sample Size	22
2.3.3	Sample Preservation	22
2.3.4	Sample Collection and Handling	24
2.3.5	Summary	27
2.4	Estimating Wastewater Loadings	27
3	ON-SITE TREATMENT AND DISPOSAL	32
3.1	Septic Tanks	32
3.1.1	Function of Septic Tanks	32
3.1.2	Construction	33
3.1.3	Locating Septic Tanks	33
3.1.4	Sizing Septic Tanks	35
3.1.5	Operation and Maintenance	35
3.2	Aerobic Tanks	36
3.2.1	Function of Aerobic Tanks	36
3.2.2	Construction	37
3.2.3	Locating Aerobic Tanks	40
3.2.4	Sizing Aerobic Tanks	40
3.2.5	Operation and Maintenance	40
3.3	Comparison of Septic Tanks and Aerobic Tanks	41
3.4	Subsurface Disposal Systems	42
3.4.1	General	42
3.4.2	Tile Fields	46

		Page
3.4.3	Leaching Cesspools	47
3.5	Non-Water Carriage Systems	47
3.5.1	Earth Pit Privy	50
3.5.2	Vault Privy	50
3.5.3	Compost Toilets	51
3.5.4	Chemical Toilets	55
3.5.5	Incinerator Toilets	58
3.6	Holding Tank and Sewage Haulage Systems	60
3.6.1	Function	60
3.6.2	Holding Tank Construction	60
3.6.3	Holding Tank Transfer Systems	61
3.6.4	Piping Couplings and Valves	62
3.6.5	Holding Tank Controls	62
3.6.7	Size of Holding Tanks	62
3.6.8	Operation and Maintenance	63
3.7	Treatment and Disposal of Septage	63
3.7.1	Septage Characteristics	63
3.7.2	Septage Disposal/Utilization on Land	66
3.7.3	Septage Treatment at Separate Facilities	70
3.7.4	Septage Treatment at Sewage Treatment Facilities	71
3.7.5	Costs	77
4	CENTRAL WASTEWATER TREATMENT SYSTEMS FOR SMALL COMMUNITIES	78
4.1	General Design Considerations	79
4.1.1	Flow Measurement	79
4.1.2	Sampling	79
4.1.3	Access to Mechanical Equipment	79
4.1.4	Buildings	80
4.1.5	Plant Site and Landscape	80
4.2	Headworks Components	81
4.2.1	Screening Devices	81
4.2.2	Grit Removal	84
4.2.3	Oil, Grease and Floating Solids Removal	88
4.2.4	Flow Equalization	89
4.3	Primary Treatment	96
4.3.1	Sedimentation	96
4.3.2	Fine Screens	100
4.4	Biological Treatment	101
4.4.1	Biological Reactions in a Treatment System	101
4.4.2	Factors Affecting Biological Activity	103
4.4.3	Nitrification and Denitrification	107
4.4.4	Suspended Growth Biological Treatment Processes	108
4.4.5	Aeration Equipment	119
4.4.6	Attached Growth Biological Treatment Processes	124
4.4.7	Secondary Clarification	135
4.4.8	Lagoons	137

		Page
4.4.9	Summary	146
4.5	Sludge Treatment and Disposal	153
4.5.1	Aerobic Sludge Digestion	153
4.5.2	Sludge Lagoons	158
4.5.3	Sludge Drying Beds	158
4.5.4	Application of Sludges to Agricultural Land	161
4.5.5	Use of Sludge for Landfill	167
4.6	Physical-Chemical Treatment	168
4.7	Disinfection	171
4.7.1	Chlorine Reactions	171
4.7.2	Chlorine Demand Measurement	172
4.7.3	Chlorine Compounds	174
4.7.4	Chlorination Apparatus	174
4.7.5	Control Systems	180
4.7.6	Mixing	181
4.7.7	Chlorine Contact Chambers	181
4.7.8	Monitoring Requirements	184
4.7.9	Summary	184
4.8	Dechlorination	186
4.8.1	Holding Ponds	186
4.8.2	Sulphur Compounds	187
5	EFFLUENT DISPOSAL ALTERNATIVES	189
5.1	Effluent Discharge to Surface Water Bodies	189
5.2	Surface Land Disposal of Effluents	190
6	SMALL WASTEWATER COLLECTION SYSTEMS	205
6.1	Conventional Gravity Systems	206
6.1.1	Sizing Gravity Sewers	206
6.1.2	Pipe Materials	206
6.1.3	Maintenance	207
6.1.4	Cost	207
6.2	Lift Stations	207
6.2.1	General	207
6.2.2	Types of Stations	207
6.2.3	Protection of Pumps	208
6.2.4	Types of Pumps	208
6.2.5	Pumps Controls	209
6.2.6	Wet Well Design	210
6.2.7	Flow Measurement	210
6.2.8	Power Requirements	210
6.2.9	Force Mains	210
6.2.10	Safety	211
6.3	Pressure Sewer Systems	211
6.3.1	General	211

		Page
6.3.2	Effect on Sewage Characteristics	212
6.3.3	Grinder Pumps	213
6.3.4	Sewer System Description	216
6.3.5	Operational Considerations	216
6.3.6	Costs	218
6.3.7	Design Example	218
6.4	Vacuum Systems	220
6.4.1	General	220
6.4.2	Effect on Sewage Characteristics	220
6.4.3	System Description	220
6.4.4	Vacuum Collection Lines	221
6.4.5	Vacuum Collection Station	225
6.4.6	Operational Considerations	225
6.4.7	Costs	227
7	REGULATORY AGENCY REQUIREMENTS AND THE APPROVAL PROCESS	228
7.1	Introduction	228
7.2	Purpose of Regulatory Agency Requirements	228
7.3	The Approval Process	229
7.3.1	Facilities Planning	229
7.3.2	Preapplication Conference	231
7.3.3	Temporary Approval	232
7.3.4	Final Approval	232
7.3.5	Construction Inspection and Start-up of Works	233
7.4	Provincial, Territorial and Federal Pollution Control Agencies	233
7.4.1	Provincial Regulatory Agencies	233
7.4.2	Territorial Regulatory Agencies	234
7.4.3	Federal Regulatory Agencies	235
7.5	Format and Contents of a Preliminary Design Report	236
8	SELECTION OF SMALL WASTEWATER TREATMENT SYSTEMS	244
8.1	Introduction	244
8.2	Selection Requirements	245
8.3	Case History - Provincial Park Treatment Facility	250
8.3.1	Introduction	250
8.3.2	Treatment Alternatives	250
8.3.4	Site Characteristics	252
8.3.5	Effluent Requirements	253
8.3.6	Interim Conclusions	255
8.3.7	Raw Waste Characteristics	255
8.3.8	Interim Conclusions	256
8.3.9	Operational Characteristics	257
8.3.10	Interim Conclusions	258
8.3.11	Cost Effectiveness Analysis	259

		Page
8.3.12	Summary	259
8.3.13	Governmental Approval	261
REFERENCES		262
APPENDIX A	- GLOSSARY	271
APPENDIX B	- PROVINCIAL, TERRITORIAL AND FEDERAL ENVIRONMENTAL AGENCY OFFICES	291

LIST OF FIGURES

Figure		Page
1	DIPPING PROBE AND RECORDER	10
2	CYLINDRICAL FLOAT INSTALLATION	11
3	SCOW FLOAT INSTALLATION	11
4	ELECTRONIC MANHOLE METER	12
5	AIR BUBBLER INSTALLATION	14
6	RECTANGULAR AND V-NOTCH WEIRS	15
7	MANUAL RECORDING OF WEIR DISCHARGE LEVELS	16
8	TYPICAL PARSHALL FLUME	18
9	LEOPOLD LAGCO FLUME (INSERT TYPE)	18
10	TRAPEZOIDAL FLUME LAYOUT	19
11	SAMPLING DEVICES	29
12	TYPICAL SEPTIC TANK CONSTRUCTION DETAILS	34
13	AN AEROBIC TANK SYSTEM	38
14	TYPICAL SEPTIC TANK TILE FIELD LAYOUT	48
15	TYPICAL SEEPAGE PIT CONFIGURATION	49
16	SMALL COMPOSTING TOILET	52
17	COMPOSTING TOILET	53
18	PORTABLE CHEMICAL TOILET	56
19	RECIRCULATING CHEMICAL TOILET	56
20	INCINERATOR TOILET	59
21	SEPTAGE ADDITION BY SLUG DUMPING TO ACTIVATED SLUDGE WASTEWATER TREATMENT PLANTS	74
22	METERED SEPTAGE ADDITION TO WASTEWATER TREATMENT PLANTS	74

Figure		Page
23	ADDITIONAL OXYGEN REQUIRED FOR SEPTAGE ADDITIONS IN ACTIVATED SLUDGE PLANTS	75
24	BAR SCREEN: CROSS-SECTION AND PLAN VIEW	83
25	COMMINUTOR	84
26	GRIT CHAMBER	85
27	ELEMENTS OF SUTRO WEIRS AND PROPORTIONAL WEIRS	87
28	GREASE TRAP	89
29	SIDELINE AND INLINE EQUALIZATION TECHNIQUES	91
30	TYPICAL DRY WEATHER FLOW AND BOD VARIATION OF MUNICIPAL WASTEWATER BEFORE EQUALIZATION	93
31	HYDROGRAPH OF A TYPICAL DIURNAL FLOW	94
32	TYPICAL RECTANGULAR PRIMARY SEDIMENTATION TANK	97
33	TYPICAL CIRCULAR PRIMARY SEDIMENTATION TANK	97
34	VERTICAL CROSS SECTION THROUGH A RECTANGULAR SETTLING TANK	98
35	HYDRASIEVE SCHEMATIC	101
36	SUBSTRATE REMOVAL AND MICROBIAL GROWTH RELATIONSHIPS	102
37	EFFECT OF TEMPERATURE ON MICROBIAL GROWTH RATE OF MESOPHILES	104
38	SLUDGE SETTLEABILITY AS A FUNCTION OF ORGANIC LOADING	107
39	FLOW DIAGRAM OF A CONVENTIONAL ACTIVATED SLUDGE TREATMENT PLANT	109
40	FILAMENTOUS MICROORGANISMS IN ACTIVATED SLUDGE	111
41	EXTENDED AERATION PROCESS FLOW DIAGRAM	113
42	OXIDATION DITCH FLOW DIAGRAM	116

Figure		Page
43	CONTACT STABILIZATION PROCESS FLOW DESIGN	118
44	TYPICAL DIFFUSED AIR DEVICES	121
45	TYPICAL SUBMERGED TURBINE AERATOR	123
46	SURFACE AERATORS	125
47	TRICKLING FILTER CROSS SECTION	129
48	FLOW DIAGRAMS OF SINGLE AND TWO STAGE TRICKLING FILTER PLANTS	129
49	FOUR-STAGE ROTATING BIOLOGICAL CONTACTOR	134
50	FLOW DIAGRAM OF AN RBC PLANT	134
51	FACULTATIVE POND SYSTEM CONFIGURATIONS	142
52	PLAN AND SECTION OF A TYPICAL SLUDGE DRYING BED	160
53	PELLET FEED CHLORINATOR	175
54	DIRECT FEED CHLORINATION SYSTEM	177
55	SOLUTION FEED CHLORINATION SYSTEM	178
56	MIXING DEVICES	182
57	CHLORINE CONTACT CHAMBER WITH LONGITUDINAL BAFFLING	183
58	IRRIGATION TECHNIQUES	191
59	ALTERNATE METHODS OF SPRAY IRRIGATION	192
60	SOIL TYPE vs. LIQUID LOADING RATES FOR DIFFERENT LAND APPLICATION APPROACHES	198
61	DRY PIT PUMPING STATION	208
62	WET PIT PUMPING STATION	209
63	IDEALIZATION OF PRESSURE AND GRAVITY COLLECTION SYSTEMS	212
64	TYPICAL INSTALLATION OF GRINDER PUMP UNIT WITHIN A HOUSE	214

Figure		Page
65	PLAN AND CROSS SECTION OF GRINDER PUMP UNIT	215
66	PRESSURE SYSTEM FLOW DIAGRAM	217
67	COMPARISON OF VACUUM AND GRAVITY SYSTEMS	220
68	ON-LINE VACUUM SYSTEM (DRY CHAMBER INTERFACE UNIT)	223
69	PROFILE FOR UPHILL TRANSPORT IN VACUUM MAINS	226
70	SCHEMATIC FLOW DIAGRAM OF PROJECT APPROVAL PROCESS	230
71	SITE PLAN FOR CASE HISTORY	251

LIST OF TABLES

Table		Page
1	SUMMARY OF FLOW MEASUREMENT TECHNIQUES	7
2	VALUES OF N IN MANNINGS FORMULA	8
3	PARAMETERS INVESTIGATED IN DOMESTIC WASTEWATER CHARACTERIZATION PROGAMS	23
4	SAMPLE PRESERVATION	25
5	ESTIMATED MINIMUM DAILY SEWAGE FLOWS IN IMPERIAL GALLONS	31
6	SEPTIC TANKS - WORKING CAPACITY FOR HOUSEHOLD SYSTEMS	35
7	GENERAL SOIL CHARACTERISTICS	44
8	CONCENTRATION OF CONTAMINANTS IN SEPTIC TANK SEPTAGE	64
9	SOME INORGANIC CONSTITUTENTS IN SEPTIC TANK SLUDGE	65
10	HEADWORKS UNITS	82
11	CHARACTERISTICS OF SOME AERATION EQUIPMENT	126
12	ESTIMATED EFFLUENT QUALITY FOR TRICKLING FILTERS	131
13	OPERATIONAL CHARACTERISTICS OF VARIOUS TREATMENT PROCESSES	147
14	ESTIMATED EFFLUENT QUALITY FOR VARIOUS TREATMENT PROCESSES	148
15	ESTIMATED ANNUAL MANHOUR REQUIREMENTS FOR VARIOUS TREATMENT PROCESSES WITH A DESIGN FLOW OF 450 m^3/d	148
16	ESTIMATED CAPITAL COSTS FOR ALTERNATIVE TREATMENT PROCESSES WITH A DESIGN FLOW OF 450 m^3/d	149
17	ESTIMATED OPERATION AND MAINTENANCE COSTS FOR VARIOUS TREATMENT PROCESSES WITH DESIGN FLOWS OF 450 m^3/d	149
18	ESTIMATED TOTAL AND UNIT COSTS FOR VARIOUS TREATMENT PROCESSES WITH A DESIGN FLOW OF 450 m^3/d	150

Table		Page
19	OPERATIONAL PROBLEMS AND CORRECTIVE MEASURES FOR VARIOUS TREATMENT PROCESSES	151
20	TYPICAL SLUDGE QUANTITIES AND CONCENTRATIONS	153
21	METAL CRITERIA FOR SEWAGE SLUDGE APPLICATION	165
22	SLUDGE AND LANDFILL SITE CONDITIONS	169
23	TYPICAL CHLORINE DOSAGES FOR DISINFECTION	173
24	COST SUMMARY FOR CHLORINATION EQUIPMENT	186
25	CHEMICAL REQUIREMENTS FOR DECHLORINATION	187
26	ALKALINITY LOSS AND RESTORATION BY CHEMICAL ADDITION	188
27	TYPICAL VALUES OF CROP UPTAKES OF NITROGEN	194
28	VALUES OF RUNOFF COEFFICIENT	199
29	ACTUAL COSTS FOR A 455 m^3/d SPRAY IRRIGATION EFFLUENT DISPOSAL SYSTEM IN THE U.S.A.	204
30	SUMMARY OF CAPITAL COSTS OF TREATMENT ALTERNATIVES FOR CASE STUDY	260
31	PRESENT VALUE AND TOTAL COST ESTIMATES FOR THE WASTEWATER TREATMENT ALTERNATIVES FOR CASE STUDY	261

1 INTRODUCTION

Small sewage treatment systems serving communities of a few hundred people often present design and operational problems not encountered with large-scale urban works, although the general processes used for treatment may be similar. Many of the unique problems encountered with small systems are caused by the use of a system unsuitable for the specific wastewater treatment requirements. Plant upsets also occur because of poor operation and maintenance practices.

This report presents information on various treatment processes which are available for small wastewater treatment systems. The selection and design of such standard rural wastewater disposal systems as septic tank - tile fields are examined and discussed, as are "package" treatment plant design and operation. Although comprehensive in content, the intention of this document is not to develop "instant experts" in the field of wastewater treatment for small communities, but rather to inform concerned individuals of the importance of in-depth study of problem situations and to describe methods of investigating waste management alternatives. The detailed design of a treatment system may become much more complex than this manual suggests; therefore, it is recommended that final design, or review of final design, be undertaken by qualified persons with experience in this field. The long-term benefits from experienced advice and review will far outweigh the costs incurred for such a service.

1.1 Scope

"Small wastewater treatment systems" are defined in this manual as the collection, treatment and disposal facilities associated with domestic wastewaters generated by individual homes, apartment complexes, restaurants, institutions, rest areas, and communities with populations of up to 2500 people. Basic information and recommended practices are provided for the design, implementation, and operation of systems that will be most economic, efficient and trouble-free in particular situations. The manual is intended for engineers, architects, technologists and contractors who are frequently confronted with the task of providing small-scale sewage treatment facilities, but who are not waste treatment specialists.

The selection of a particular waste management system for a small community involves review of three principle alternatives: 1) on-site treatment and disposal of wastes; 2) installation of a small package system to treat community wastes collected via a central collection system; and 3) formulation of a regional waste management plan in which one central treatment facility receives wastes from two or more population centres. In many instances, the selection of one of these major alternatives may become obvious upon cursory review of regulatory requirements, inspection of site location and conditions, and/or review of regional development plans. Otherwise, the relative merits of each alternative must be reviewed on the basis of cost/effectiveness, i.e., the unit cost of a system relative to such tangibles as public health protection, the protection afforded the receiving environment, aesthetics, energy conservation, reliability and operational requirements. Obviously, the rationale used in the decision making process will vary from situation to situation. This manual is intended to provide sufficient information to permit the analysis of waste management problems with an awareness of the solutions that are available.

Of ever-increasing importance in the design and selection of waste management systems is the conservation of energy. Although this factor is not dealt with specifically in this report, the conscientious designer should, in all cases, develop systems which minimize the use of electrical power while maintaining an acceptable standard of effluent quality and operational simplicity. This may entail the selection of a system with design modifications over an otherwise obvious off-the-shelf package. Energy conservation is of particular importance in isolated areas where power must be generated on-site by means of fossil fuel consumption.

1.2 Objectives

The specific objectives of this manual are:

a) to provide information on various small wastewater treatment systems and to define the constraints, limitations and supplementary considerations when selecting and/or designing such systems;

b) to provide data on capital expenditures and operation and maintenance costs for various small wastewater treatment systems;

c) to identify applicable governmental regulations and requirements, and outline the steps and procedures involved in obtaining approval for installation of small wastewater treatment systems.

1.3 Organization

Sections 2 to 6 of this manual discuss design considerations for various alternative small wastewater treatment systems; sections 7 and 8 outline rationales and criteria that can be used in making a selection from the various alternatives. Terminology used throughout the manual is consistent with general practice in wastewater management; a glossary is provided as Appendix A. Appendix B lists federal and provincial regulatory agencies.

2 WASTEWATER CHARACTERIZATION

Wastewater characterization is the grassroots of process design and selection in a waste management system. Characterization studies provide information on flow variation, waste loading fluctuations, and treatment efficiency and flexibility.

The characterization of wastewater generated within an existing sewered community or installation involves the implementation of a monitoring program. In the case of completely new systems, the designer must rely on empirical values identified by regulatory agencies or found in literature. This section provides detailed information on initiating and conducting wastewater characterization programs at existing installations, as well as the utilization of empirical values for estimating wastewater loadings for new facilities.

2.1 Wastewater Monitoring Program

The objective of a monitoring program is to provide an understanding of the characteristics of water-borne waste materials. Good planning is necessary to establish a monitoring program which is inexpensive, convenient and effective, and does not interfere with normal activity at the installation under study. Steps involved in the preparation of a monitoring program include:

a) development of collection system flow sheet,
b) selection of sampling stations,
c) coordination of laboratory support,
d) selection of monitoring equipment.

2.1.1 Development of Collection System Flow Sheet. The first step in the development of a wastewater survey is a review of all existing facilities and services. This is best accomplished by on-site inspection to compile information on waste sources and waste collection techniques. Included among facilities and services within the planning area which could have a significant affect on a treatment system are: combined sanitary and storm water collection systems; institutions such as schools and sports complexes; restaurants; hotels; and industries. Wastewater from industrial complexes should be immediately identified on this initial inspection and brought to the attention of laboratory personnel involved in analyzing collected samples. It would also be advantageous to identify the operations carried out within any industrial area to determine possible waste constituents in a discharged effluent. Meetings with industrial plant managers are advisable to secure this information.

Upon completion of the site visit, the inspector should have a reasonable grasp of the layout of existing facilities and services and be capable of developing an up-to-date sewer map showing water, sanitary and storm drain lines. Where applicable (i.e., at proposed monitoring sites), the map should specify pipe size and location, direction of flow, and location and depth of manholes, catch basins, pumping stations and outfalls. Where wastewater collection systems have been previously installed, much of the required information should be available in "as-built" drawings at municipal offices.

2.1.2 Selection of Monitoring Stations. Careful selection of monitoring sites is crucial. Ideally, the sites should include all pertinent substreams and be located only where the wastewater is well mixed. Unfortunately, many areas to be sampled are far from ideal. When a preferred sampling site is located under several feet of concrete, or when an outfall is below the water level of the receiving stream, compromises must be made. Sometimes it is necessary to sample several contributory streams which make up the flow at the inaccessible site and combine these samples in proportion to their flow contribution. This procedure can result in a reasonable approximation of the actual wastewater.

Other considerations in the selection of sampling sites are safety of sampling personnel, accessibility, and security of the sampling instruments from vandalism.

2.1.3 Coordination of Laboratory Support. Before finalizing the monitoring program, the investigation should be discussed with the laboratory support group. At such time, scheduling of the program, together with duration and intensity of the study, will be established. Information concerning analytical interference from contaminant input into the waste stream (i.e., industrial waste) should be given to laboratory personnel and preventive measures discussed. Local regulatory agencies should be able to assist project initiators in the selection of suitable analytical laboratories for wastewater monitoring programs.

2.1.4 Selection of Monitoring Equipment. To fulfill the purposes of a raw waste characterization program, monitoring equipment must be capable of reflecting truly representative data. The principle concerns in a monitoring program are flow measurement and wastewater sampling.

2.2 Flow Measurement

The method of flow measurement selected depends upon the following:

a) type of flow, i.e., pressure or gravity flow,

b) quantity of flow,

c) accuracy required,

d) duration of collection,

e) diurnal variation of flow variability.

The accuracy of flow measurement equipment is perhaps the most difficult variable to define in the selection of monitoring devices, primarily because of its relation to so many other variables in the system. Accuracy may be defined as the estimated standard deviation of repeated measurements, which is expressed as a percentage of the adopted value, usually the mean. The required accuracy of flow measurements depends on the eventual use of the collected data. For design purposes, an accuracy of 10 to 15% is acceptable, considering other incertainties involved (1).

Flow measurement techniques are examined in succeeding sections with regard to their applicability in characterization of raw wastewaters for small wastewater treatment systems. A summary of flow measuring devices is given in Table 1.

2.2.1 Fill and Draw Devices. Two simple methods of measuring flow in a small wastewater treatment system survey are the pump rating method and the bucket-and-stopwatch method. When wastewater in a sewer system is being pumped out of a reservoir, an estimate of flow may be obtained by recording the duration of pumping and the capacity of the pump at the discharge pressure, using head versus capacity curves supplied by the pump manufacturer, or by recording the time of pumping and knowing the volume of wastewater pumped out of the reservoir. The bucket-and-stopwatch method can be used to determine instantaneous flow from a pipe. Effluent from the pipe discharges into a container of known volume and the time required to fill the container is observed and recorded.

The application of fill and draw devices for characterization of wastewater flows is limited to individual homes, small institutions and sources providing uniform flow rates, or at least flow rates of small variability. Variations in flow rate are common and therefore the accuracy (and usefulness) of fill and draw devices is limited.

TABLE I SUMMARY OF FLOW MEASUREMENT TECHNIQUES

Technique	Application: outfall	Application: man-hole	Application: sewer pipe	Estimated Accuracy	Cost Range *
1) Depth Measurement only**:					
Dipping probe	x	x		15 - 20%	$2400-$2800
Float liquid sensors:	x	x		15 - 20%	
Float & counter-weight	x	x		15 - 20%	1000-1200
Scow float & counterweight	x	x		15 - 20%	1200-1350
Scow float & pivot arm	x	x		10%	2000-2800
Bubbler sensor	x	x		10%	1500-2000
2) Weirs:					
Rectangular	x	x		5%	50-100
V-notch	x	x		5%	50-100
3) Flumes:					
Parshall	x			5%	500-700
Leopold-Lagco	x	x		5%	400-600
Trapezoidal	x	x	x	5%	400-600

* 1976 manufacturers' costs (average); prices will vary from manufacturer to manufacturer, and with size of sewer.

** Complete with flow recorder and accessories.

2.2.2 Depth Flow Measurement. Depth flow measurement consists of measuring the depth of flow at a suitable cross-section in the sewer system and substituting the recorded depth into an equation for uniform flow. Most frequently the Manning equation is used in this calculation (2):

$$Q = A \frac{1}{n} R^{2/3} S_f^{1/2}$$

where:
- Q = flow rate (m^3/s),
- A = cross-sectional flow area (m^2),
- n = Manning's roughness coefficient,
- R = hydraulic radius (m),
- S_f = the friction slope ($S_f = S_o$, bottom slope, for steady uniform flow).

When this method of flow measurement is used, its limitations with respect to accuracy must be recognized. For example, the estimates of Manning's n in the equation are an unavoidable source of error; the values given in various handbooks vary significantly (Table 2). Furthermore, the friction coefficient also varies with the depth of flow and may be influenced by local form losses, such as losses at sewer pipe junctions, manhole structures or poor joints.

TABLE 2 VALUES OF n IN MANNING'S FORMULA (2)

	n	
Nature of Surface	Min	Max
Neat cement surface	0.010	0.013
Wood-stave pipe	0.010	0.013
Vitrified sewer pipe	0.010	0.017
Metal flumes, smooth	0.011	0.015
Concrete, precast	0.011	0.013
Cement mortar surfaces	0.011	0.015
Common-clay drainage tile	0.011	0.017
Concrete, monolithic	0.012	0.016
Brick with cement mortar	0.012	0.017
Cast iron	0.013	0.017

A variety of liquid level sensors for liquid depth measurement are available commercially. Three of these instruments are considered applicable for depth flow measurement in small wastewater treatment system installations.

a) Dipping probes (1). A dipping probe (Figure 1) is a thin stainless steel probe which is lowered to the liquid surface on a wire controlled by a precision motor. The probe makes a contact with the surface of the liquid and retracts slightly, then repeats this cycle until stabilization is reached just above the liquid. Any change in the liquid level will result in a shortening or lengthening of the unwound wire (thus no fouling of probe with suspended material). Changes in cable length are translated to the rotation of a measuring wheel which in turn, through an electronic servo system, positions a pen to indicate instantaneous flow rate on a recording chart.

b) Float liquid level sensors (1). Mechanical level monitors which use floats as sensors are the oldest instruments used for liquid level monitoring. They are very reliable, inexpensive, and easy to maintain. Included among the disadvantages of these instruments are the need to construct a stilling well and the fouling of the floats by suspended materials in the sewage. Figure 2 illustrates a mechanical water level recorder which operates by means of a cylindrical float in a stilling well. The instrument accuracy is better than 3 mm (0.01 ft) using a standard float size; it can be further improved by using an oversized float. The instrument recorder is driven either by a weight, or by a synchronous motor.

   In many monitoring situations, space limitations do not permit the installation of a stilling well. Conventional float level recorders must then be modified by replacing the standard float with a scow float (see Figure 3). Commercial electronic products using quasi-spherical floats (and other shapes) as scow floats are available (Figure 4). In such cases, the scow float is held in place by a pivoted arm. The float movement induces angular movement to the shaft on which it is supported. Movement of the shaft is converted into an electronic (or pressure) signal which activates the recording instrument. Such an arrangement is more reliable (and expensive) than the cable-suspended scow floats.

c) Pneumatic water level gauges (air bubblers) (1). Pneumatic level gauges are frequently used for measurements in sewers. These instruments are relatively simple, do not obstruct flow, and the sensor can be installed several hundred feet from the recorder. The method of operation involves forcing gas (air or nitrogen) from pressurized cylinders through a thin tube (dip tube) which is immersed into the

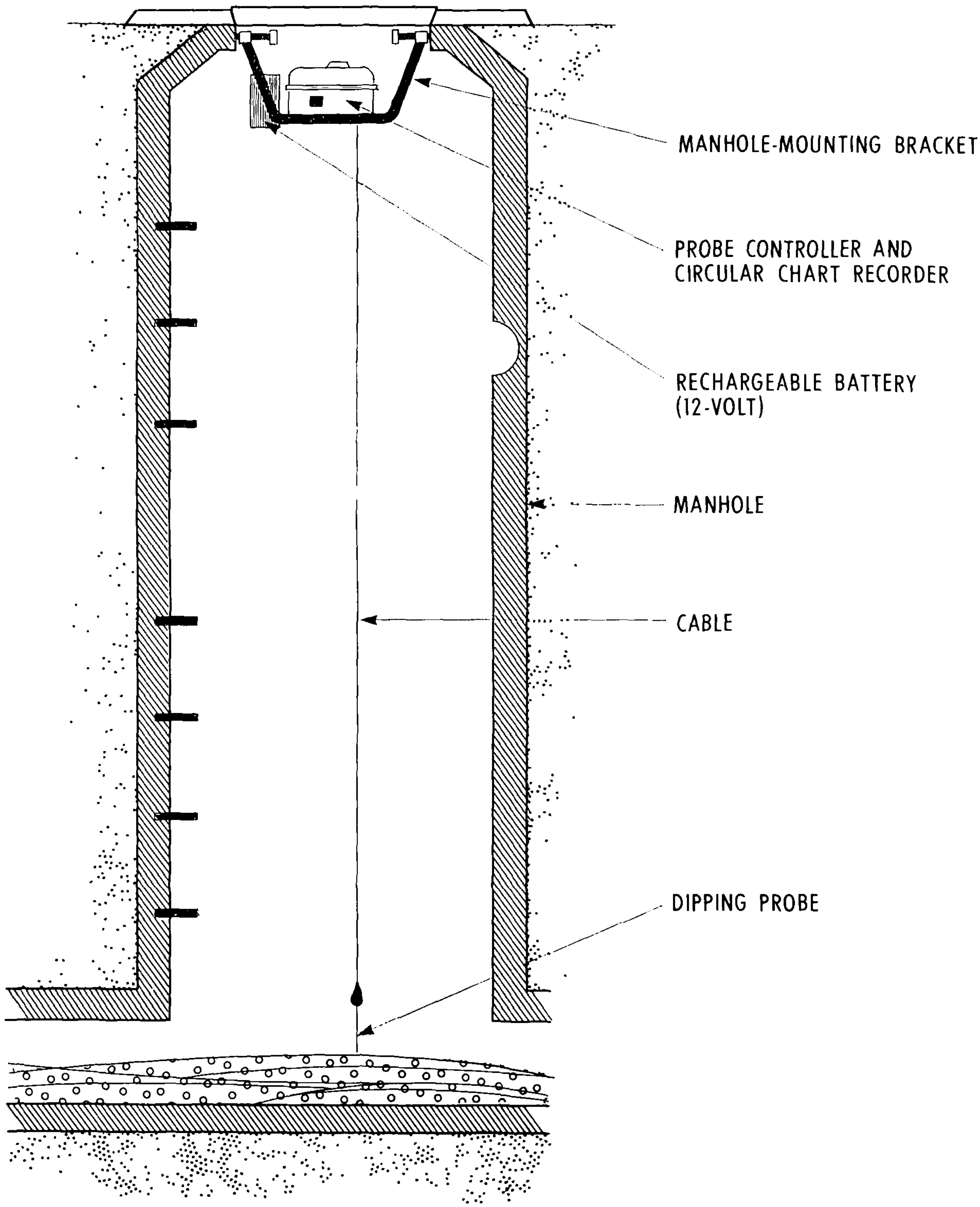


FIGURE 1 DIPPING PROBE AND RECORDER
(From Manning Environmental Corporation)

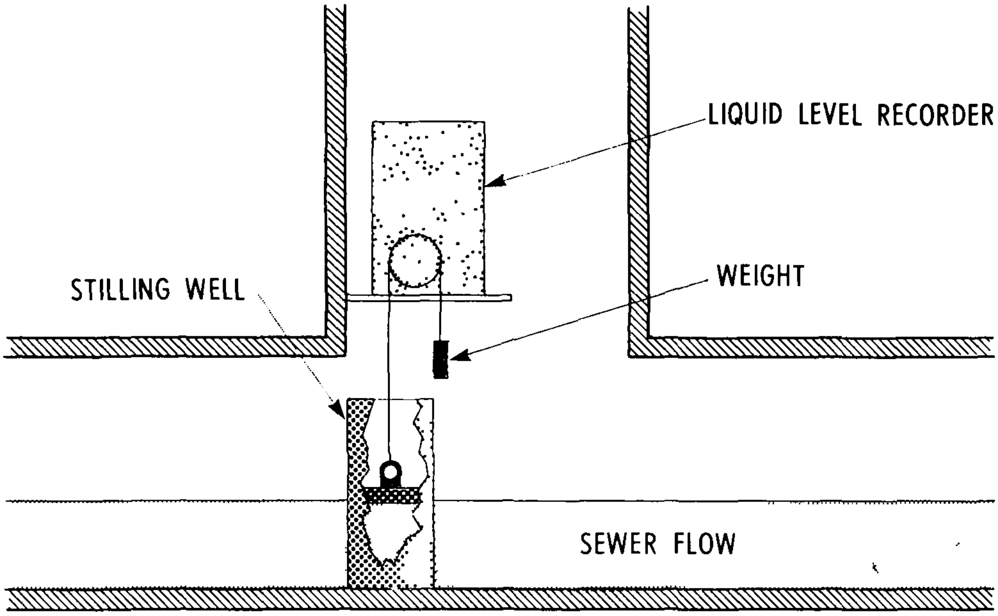


FIGURE 2 CYLINDRICAL FLOAT INSTALLATION (1)

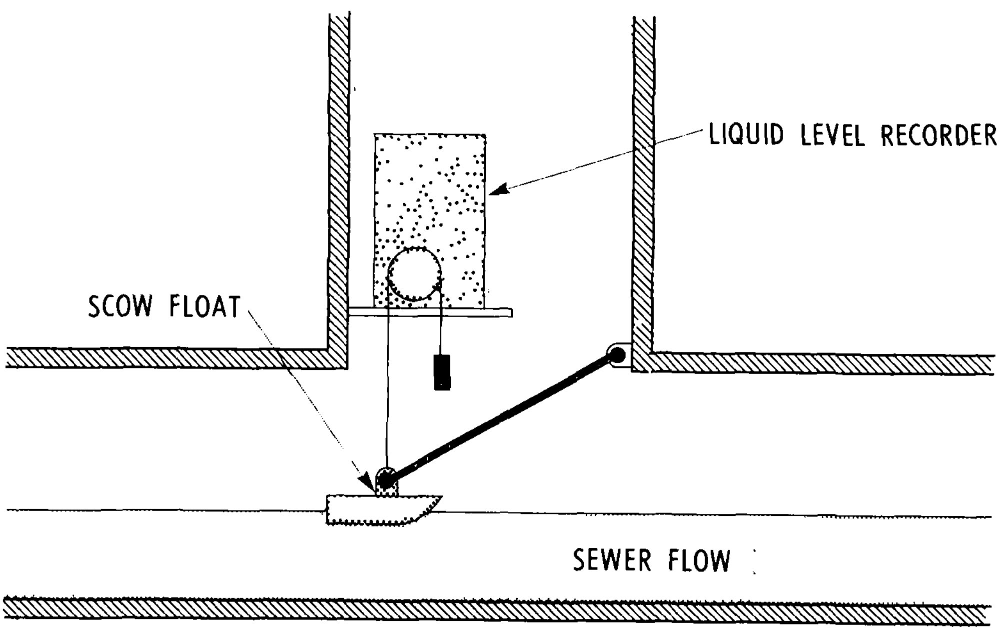


FIGURE 3 SCOW FLOAT INSTALLATION (1)

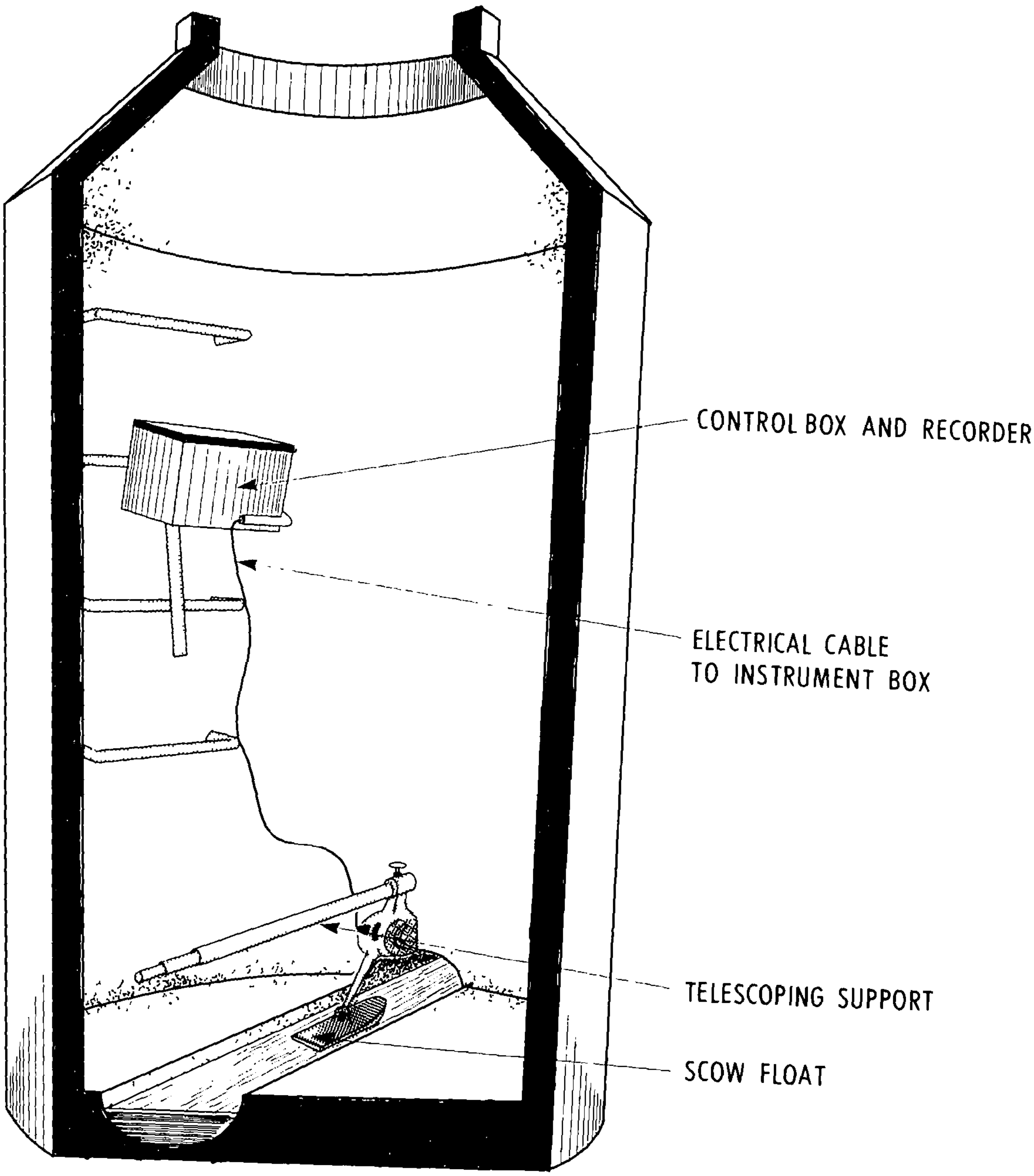


FIGURE 4 ELECTRONIC MANHOLE METER
(From N.B. Products Inc.)

liquid being measured. Gas pressure at the end of the dip tube equals the static pressure due to the weight of liquid column above the point of gas discharge, and thus is a measure of the depth of the liquid. The transmitter output (pressure) is either recorded directly, or is first converted to an electronic signal and then recorded. For fast flowing liquids, e.g., liquids with a velocity of 0.5 m/s (2 ft/s), it is recommended that the dip tube be protected by a simple "stilling well" in the form of a concentric sleeve pipe (see Figure 5). Without this protection, the bubbler readings could be affected by the velocity of a fast flowing media (i.e., the dynamic pressure).

2.2.3 Measuring Weirs. The measurement of small wastewater flows in gravity sewers or open channels can satisfactorily be achieved with the use of weirs. These primary measuring devices are relatively accurate (2-5% except at very low flows), quite reliable and of reasonable cost. The following types of weirs have been used in sewer studies:

a) rectangular weir,
b) V-notch weir,
c) vertical slot weir,
d) trapezoidal weir (without bottom part).

Among the disadvantages of weir use are the reduction in sewer pipe capacity, upstream accumulation of solids for weir-types a) and b), the possible distortion of flow due to the presence of the weir, and a limited operational range because of surcharging and submergence (1).

Installation of weir devices will differ with each situation. The basic intent is to provide a barrier or bulkhead across the stream so that water will overflow the barrier through a notch in the center of the structure (3). The weir bulkhead may be made from a thin piece of aluminum or stainless steel cut to the approximate shape of the channel cross section. The height of the structure must be adapted to on-site conditions. A plastic sheet and sandbags can be used to seal the edges of the structure to reduce leakage around the bulkhead. The velocity of approach to the weir should be negligible; in some cases, baffles may be necessary.

The only practical method of recording discharge head over a weir is usually to install a liquid level sensor with a chart recorder upstream of the weir. As illustrated in Figure 6, the minimum distance that the depth sensor should be located from the weir is

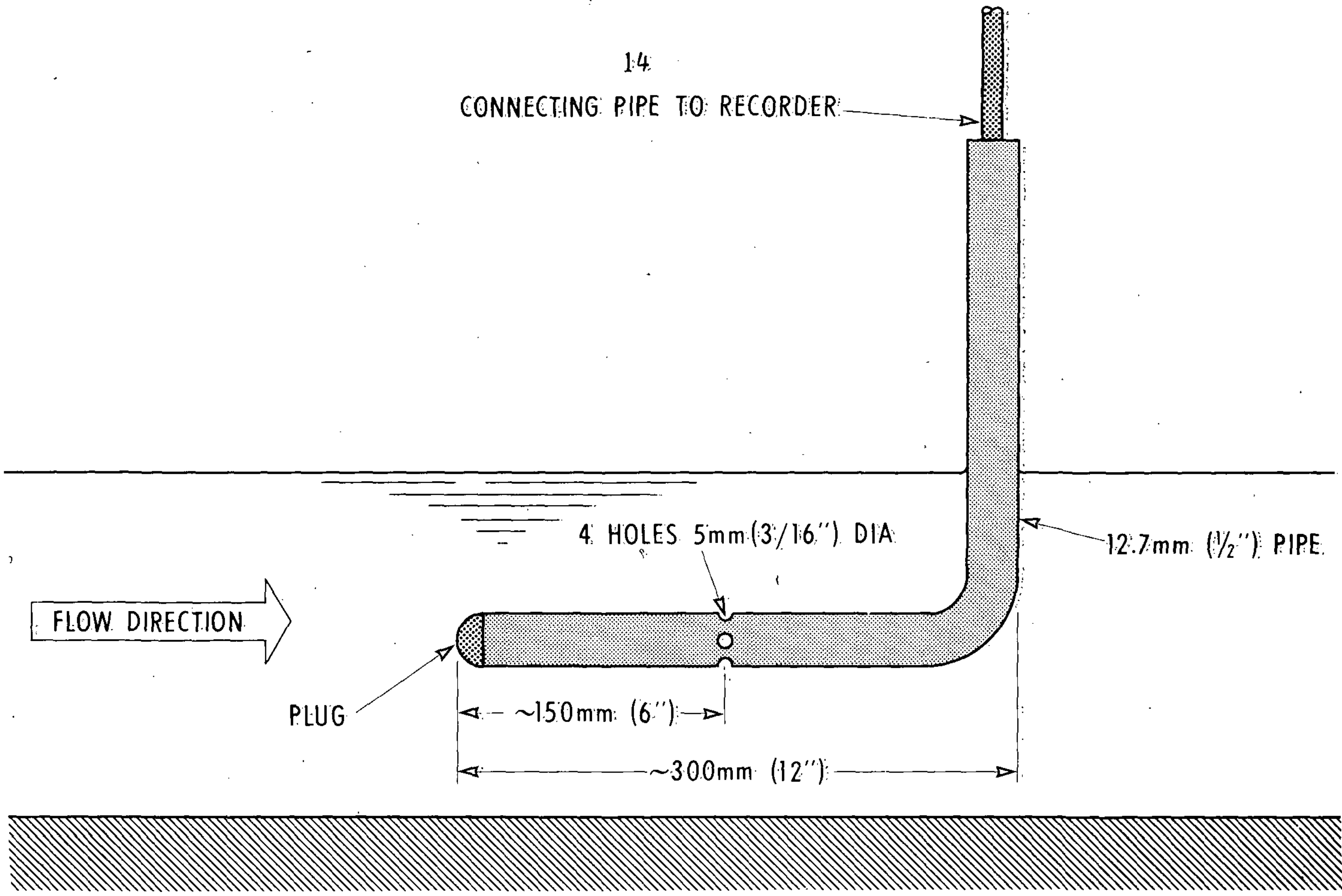


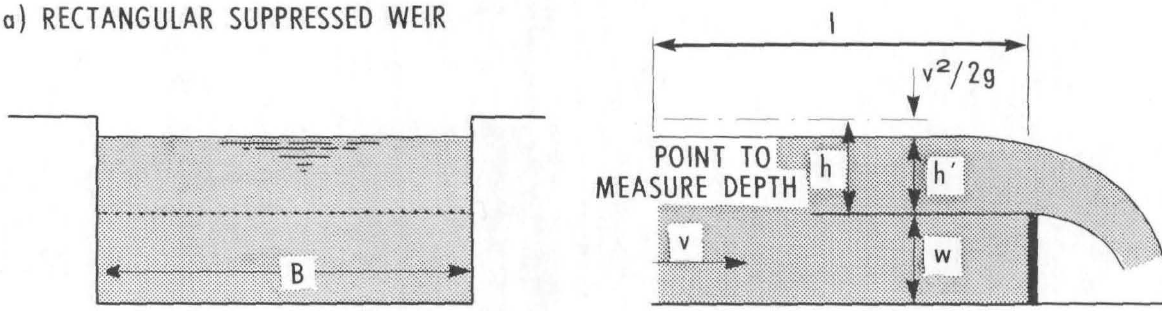
FIGURE 5 AIR-BUBBLER INSTALLATION (1)

defined to avoid the drawdown effect that occurs at the weir crest. If liquid level sensors are not available, continuous manual recording of the weir discharge head may be undertaken using gauges as illustrated in Figure 7. A brief description of weir-types a) and b) and their applicability for flow measurement follows. For further information reference (1) should be consulted.

a) Rectangular weirs. The rectangular weir is the simplest measuring device to construct. Two problems encountered with this type of weir, however, are that it considerably reduces the pipe capacity, causing an accumulation of solids immediately upstream of the device, and it is insensitive to low flows. The former problem can be solved by constructing the weir at a sewer outfall or, if this is impossible, cleaning the weir daily to remove deposited solids (flushing with hose). The lack of sensitivity to low flows would negate the use of this device in situations where this condition is anticipated. A typical installation is shown in Figure 6 a).

b) V-notch weirs. A V-notch weir creates more constriction of the flow than a rectangular weir. This not only provides good measuring sensitivity at low flows, but also reduces the pipe capacity considerably. The damming of sewage flows and

a) RECTANGULAR SUPPRESSED WEIR

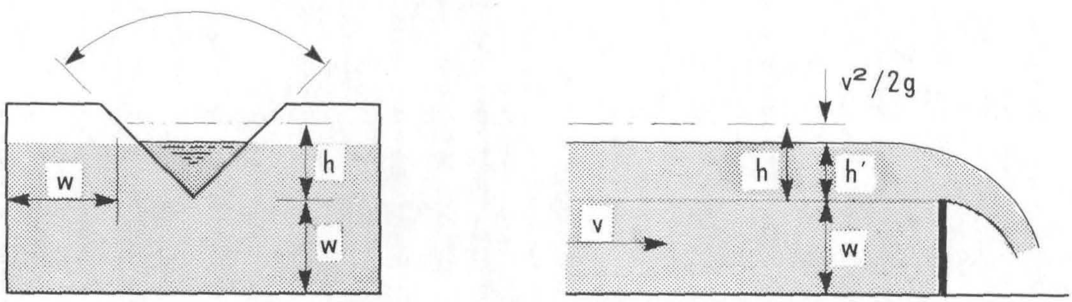


RECOMMENDED GEOMETRY: $w=2h_{max}$, $l=4h_{max}$

FLOW RATE: $Q(m^3/s/m)=2.953(0.604+.081h/w)(h+.001)^{1.5}$
WHERE h,w (m)

$Q(cfs/ft)=5.347(0.604+.081h/w)(h+.0034)^{1.5}$
WHERE h,w (ft)

b) 90° V-NOTCH WEIR



RECOMMENDED GEOMETRY: $w=2h_{max}$

FLOW RATE: $Q(m^3/s)=1.34h^{2.48}$ (m)

$Q(cfs)=2.48h^{2.48}$ (ft)

FIGURE 6 RECTANGULAR AND V-NOTCH WEIRS (1)

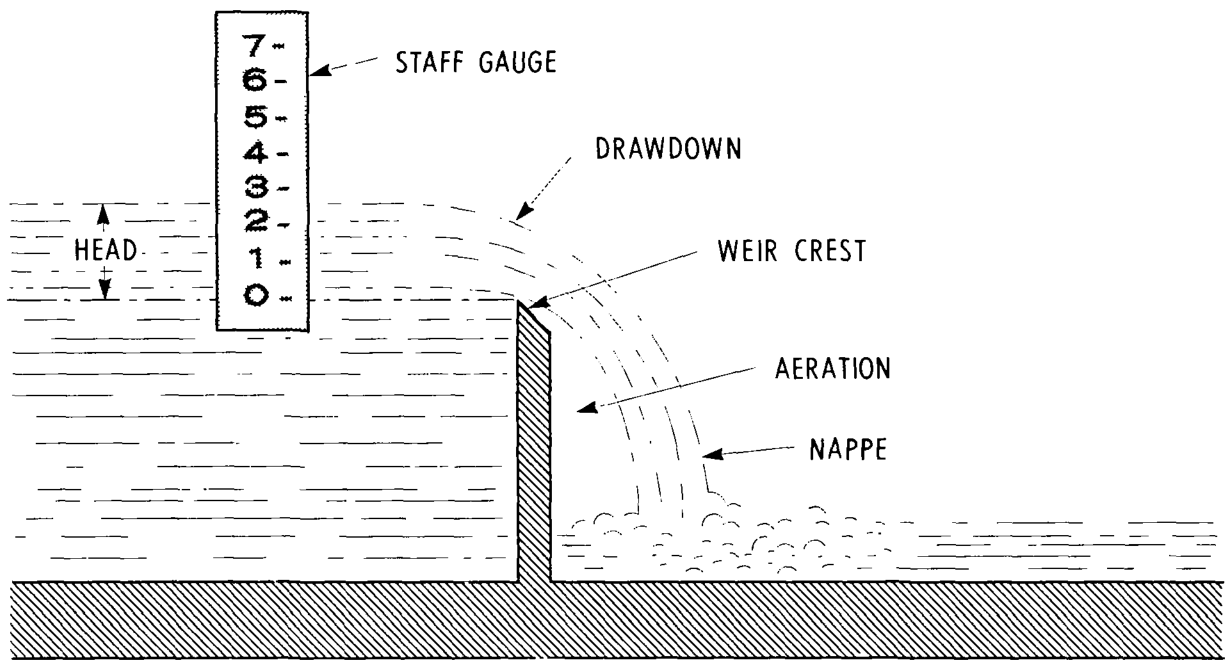


FIGURE 7 MANUAL RECORDING OF WEIR DISCHARGE LEVELS (3)

solids deposition upstream of weir are major drawbacks. Consequently, precautions similar to those cited in the previous discussion are required when using V-notch weirs. There are four sizes of V-notches that are commonly used: 90°, 60°, 45° and 22-1/2° (3). The size selected will depend upon the expected flow range and the allowable depth over the weir. A 90° V-notch is used for large flows, with the allowable minimum depth of flow over the weir being about 6 cm (0.2 ft). With smaller weir sizes, such as 45° and 22-1/2°, the minimum depth of overflow can be somewhat lower, but should not be less than 3 cm (0.1 ft). At low heads, the overflow nappe may cling to the weir, rendering the relationship between head and flow unreliable. Figure 6 b) illustrates a 90° V-notch weir installation.

2.2.4 Measuring Flumes. The flume is a commonly applied flow measuring device which operates using the Venturi principle to determine flows, i.e., if downstream flow conditions do not affect the flow through the flume, then a definite relationship exists between the upstream liquid depth and the flow through the facility. Flumes have the same reliability and accuracy as weirs, but offer the advantages of being self-scouring and having minimal effects on upstream hydraulic characteristics. Flumes, however, are considerably more expensive than weirs.

Flumes of various geometries are used. Some flumes have bottom contraction (hump); others do not. The former can be easily inserted into sewers and held in place, and the error in head caused by standing water is eliminated.

Three types of flumes are commonly used:

a) Parshall flume. A Parshall flume, illustrated in Figure 8, consists of a converging section, a rectangular throat and a diverging section. The level of floor in the converging section is higher than the floor in the other two sections. Parshall flumes are well known and have been used extensively in open channel flow measurement (e.g., sewer outfalls, entrance to sewage treatment plants) in conjunction with depth measuring devices. The use of Parshall flumes in manholes or sewer pipe is not advised due to space limitations.

b) Leopold-Lagco flumes. Leopold-Lagco flumes are manufactured commercially. The flumes are made of fibreglass and are installed directly in the sewer line (see Figure 9), normally at a standard straight-through manhole. The flumes cause small head losses and can be installed on grades up to 2%. Leopold-Lagco measuring flumes are made in nominal sizes from 0.15 to 1.8 metres (6 to 72 inches), with maximum discharge ranging from 0.007 to 3.35 m^3/s (0.25 to 118 cfs), respectively.

c) Trapezoidal flumes (Palmer-Bowlus). Trapezoidal flumes are quite popular for sewer flow measurement. They are manufactured commercially (or can be custom made) and can be installed directly into a sewer line. The principal advantages of trapezoidal flumes are: i) the small reduction in the cross-sectional flow area; and ii) by selecting the throat width and the slope of sidewall, the flume can be designed to give specific heads at two points within the flow range. Figure 10 illustrates the installation of a Palmer-Bowlus flume.

In the operation of a flume, the upstream flow depth (head) is measured at a specific point which is exactly defined for each type of flume. The previously discussed liquid level sensors and recorders can be used in conventional installations. The output signal of level sensors can usually be converted into a flow-proportional signal by electronic components or mechanical cams, and recorded on a chart recorder (1).

When selecting the location for a flume, the flow characteristics in the adjacent upstream and downstream reaches should be examined. The flumes should be installed in sections which do not surcharge frequently, where the incoming flow is

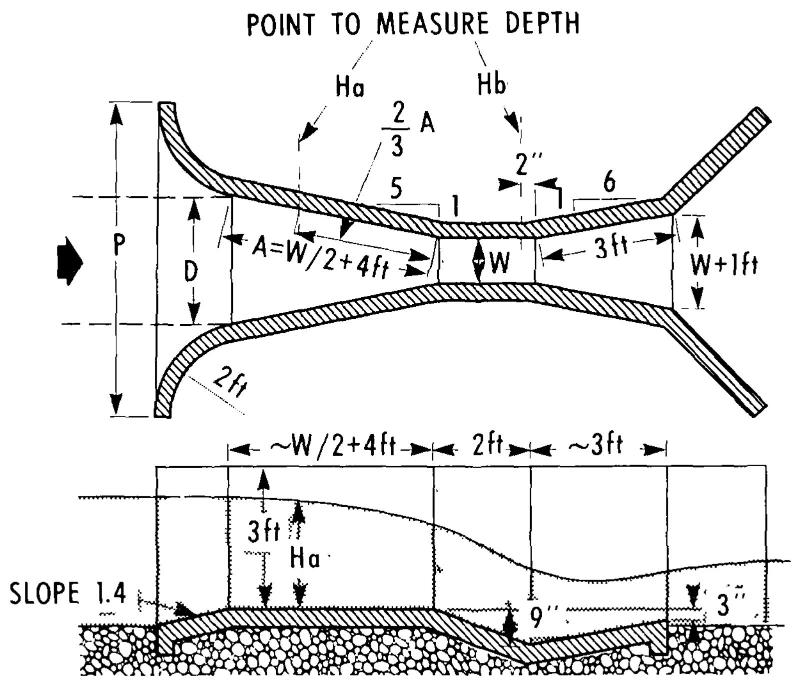


FIGURE 8 TYPICAL PARSHALL FLUME (1)

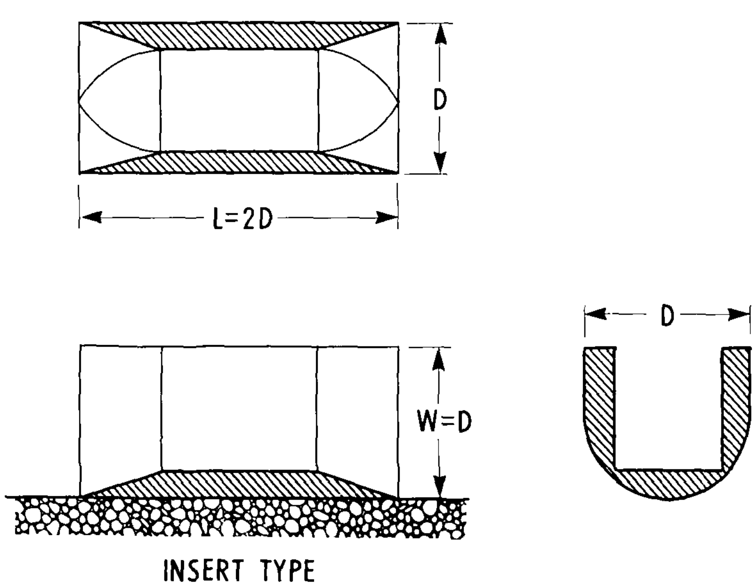
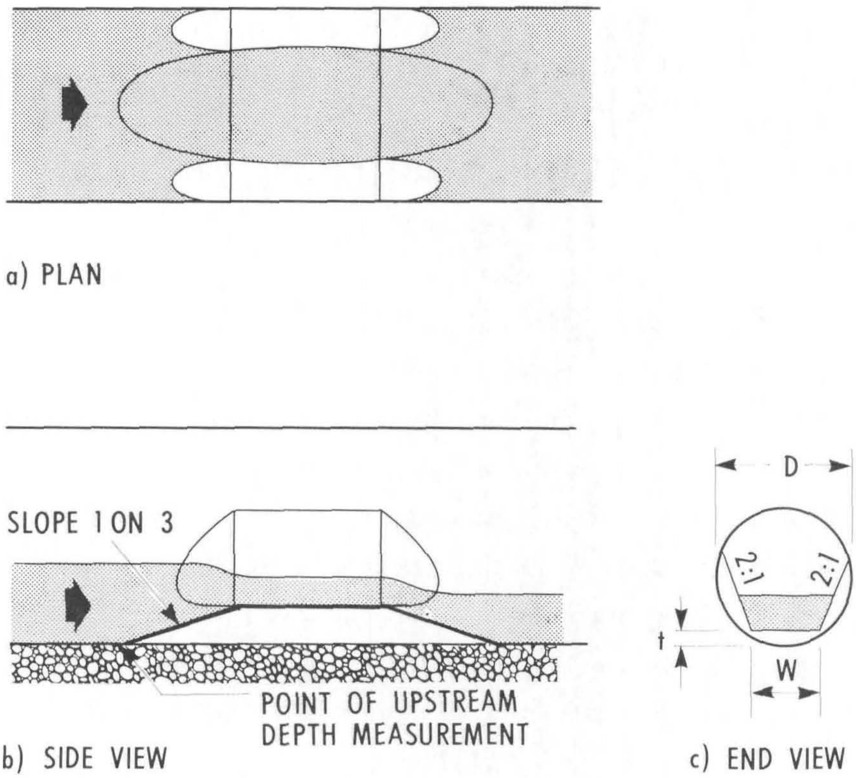


FIGURE 9 LEOPOLD LAGCO FLUME (INSERT TYPE) (1)



FLUME GEOMETRY W=5D/12, t=D/12, SIDE WALL SLOPE 1.5:1 OR STEEPER

FIGURE 10 TRAPEZOIDAL FLUME (PALMER-BOWLUS) LAYOUT (2) subcritical (sewer slope $S \leq 2\%$) and no backwater effects occur. For an incoming supercritical flow ($S > 2\%$), the flume has to create a sufficient constriction of the flow cross-section to cause a hydraulic jump, preferably far upstream from the flume. A hydraulic jump in the proximity of the flume would impair the measurement of the hydraulic head. The installation of conventional flumes on supercritical slopes (typically $S > 2\%$) should be avoided, since the transition from the shooting to the tranquil flow will frequently result in the sewer pipe surcharging. The capacity of the downstream reach should be checked for the occurrence of backwater effects which could create submergent conditions at the flume (1).

The overall accuracy of flumes is typically 5% of the measured value through the full operational range, with the exception of very low flows (1). For very low flows, the error in the rating curve, as well as the relative error in the measured head, becomes quite significant.

2.2.5 Summary. Fill and draw devices are simple units devised to manually determine instantaneous flow from a pipe. The application of such devices is quite limited and measurements obtained should be considered only rough estimates of uniform sewage flow. Depth flow measurement may be accomplished using a variety of liquid level sensors. However, major sources of error are encountered with this method of flow measurement due to non-uniformity and unsteadiness of flow, as well as the uncertainty of the value of Manning's "n". Weirs are inexpensive flow devices, but are faulted for frequently interfering with the transport of solids and causing energy loss in the liquid stream. Flumes are more expensive devices which allow the passage of solids and cause relatively minor energy losses. A summary of the flow measurement methods was presented in Table 1.

2.3 Wastewater Sampling

Another important aspect of raw wastewater characterization is the determination of the quality of treatment plant influent. The quality of sewage is assessed by performing laboratory analyses on representative samples collected in the field. Major errors will occur in treatment plant design if the accuracy of the sampling program is not ensured. The following details are particularly important.

a) The samples must be truly representative of the wastestream.
b) Proper sampling techniques must be used.
c) The samples must be protected until they are analyzed.

The following discussion outlines accepted procedures and equipment utilized in wastewater sampling programs. More specific information may be obtained from local regulatory agencies as required.

2.3.1 Types of Samples. The selection of a particular sampling technique depends upon the nature of the facility being monitored, the degree of pollutant concentration variation during the monitoring period, and the parameters to be measured. Two frequently used sampling techniques are grab samples and composite samples.

a) Grab Sampling. Grab samples are collected either manually or automatically from a waste stream and represent the quality of the wastewater at the specific time and location at which the sample is taken. Grab sampling is usually performed when the following conditions prevail (5):

   i) The waste to be sampled does not flow continuously.
   ii) The waste characterictics are constant.
   iii) It is desirable to determine whether or not a composite sample obscures extreme conditions of the waste. (A classic example is the possible variation of pH).

   Grab samples are also required when analyzing wastewaters for parameters such as dissolved gases, residual chlorine, temperature, and pH.

   A variation of this sampling technique is sequential grab sampling where discrete grab samples are collected at constant or variable intervals, depending on the nature of the automatic sampling device. If the sampler is connected electronically to a flow measuring device, sampling intervals will vary as flows vary; if the sampler is time controlled, sampling intervals will be constant. Sequential grab samples may be submitted to laboratories as discrete samples, or manually combined to form a composite sample.

b) Composite sampling. Composite sampling involves combining all samples collected over a specified time into a single container. The composite sample then represents an average pollutant concentration. The main advantage of the composite technique is the low cost of quality analysis, since only one sample has to be analyzed. There are several variations in the make-up of composite samples.

   Simple composite sample (non-weighted) (1): In this sampling technique, constant aliquots of wastewater are withdrawn at regular intervals and collected in one container to constitute the composite sample. The technique is inexpensive and the least sophisticated samplers and flow recorders may be used. The simple sample underestimates the total pollutant yields. The reason for this underestimation is that high concentrations and flow rates are assigned the same statistical weight in the sample composition as low concentrations and flow rates.

Flow-weighted composite sample (1,4): Flow-weighted sample composition requires more sophisticated and more expensive equipment than simple composition. The flow-weighted composition can be done either automatically or manually. Two automatic techniques are available. In the first, a constant fraction of the flow is collected continuously; in the second, samples of constant volume are withdrawn at irregular time intervals, based on a preselected constant quantity of wastewater passing through a flow measuring device. The former automatic technique is not recommended where the stream is high in suspended solids. The latter technique requires the installation of a flow meter with a flow integrator to activate the sampler. Manual flow-weighted sample composition may be achieved by proportioning the volume of individual sequential grab samples relative to the quantity of flow between the collection of two consecutive samples. The proportioning is done upon completion of the monitoring period using a recorded flow chard.

Sequential Composite Sampling (1,4): This technique is a compromise between the composite and the grab sample. Sequential composites require the collection of series of individual samples (flow-weighted) representing specific time periods (e.g., four to eight hours). This procedure is particularly useful in small domestic wastewater systems where the character of waste may vary significantly, depending on the time of day.

2.3.2 **Sample Size.** The sample size is based on the number and types of laboratory analyses to be performed; the minimum volume of a composite sample is about two litres. A list of parameters essential to wastewater characterization studies is shown in Table 3 along with elements which may be considered desirable parameters in such programs. The analytical laboratory should be consulted to determine the required volumes of sample for each individual analytical test; the sum of all required volumes being the size of the sample submitted. Local regulatory agencies should be consulted to verify analyses required in the waste monitoring program.

2.3.3 **Sample Preservation.** Certain parameters cannot be determined accurately if there has been any significant time lapse between sampling and analysis (e.g., of dissolved gases). The analysis of parameters may be divided into two categories: obligatory field analyses; and, those which may be performed in the laboratory.

TABLE 3 PARAMETERS INVESTIGATED IN DOMESTIC WASTEWATER CHARACTERIZATION PROGRAMS

	Frequency of Analysis	
Parameter	Essential	Desirable
Biochemical Oxygen Demand	x	
Chemical Oxygen Demand		x
Total Organic Carbon		x
Total Solids		x
Suspended Solids	x	
Dissolved Solids	x	
Volatile Suspended Solids		x
Volatile Dissolved solids		x
Phosphorus - Total P	x	
Orthophosphate, Hydrolyzable		x
Orthophosphate, Soluble		x
Total Kjeldahl Nitrogen	x	
Nitrogen - Ammonia		x
Nitrite and Nitrate		x
Oil and Grease	x	
Phenols		x
Chloride		x
Organic Chlorine		x
Heavy Metals		x
pH	x	
Acidity-Alkalinity	x	
Temperature	x	

1) Field Analysis Essential: dissolved oxygen, pH, temperature, odour.
2) Laboratory Analysis: biochemical oxygen demand (BOD), chemical oxygen demand (COD), total organic carbon (TOC), suspended solids (SS), etc.

Field analyses should be completed as near the collection site as is feasible and with the least possible delay. Regardless of the nature of the sample, complete stability for every constituent can never be achieved. At best, preservation techniques can only retard the chemical and biological changes that inevitably continue after the sample has been removed from the parent source (5).

As a general rule, preservation techniques should be discussed with personnel in the analytical laboratory. Sample preservation is best achieved by cooling the samples to temperatures near freezing or below. Commercial samplers with built-in refrigeration units or samplers designed with a cavity to store ice can be used. For other parameters however, chemical preservatives must be used. These preservatives may be added to the sample container prior to the sampling period if they do not interfere with other analyses to be conducted on the sample. If analytical tests cannot be completed due to interference, the preservative should not be added to the container. In such cases early removal and segregation of the required volume of sample from the sampling device will be necessary. The segregated sample may have to be "fixed" with the preservative prior to transport to the laboratory depending on the time frame involved. Laboratory personnel will be of assistance in determining these requirements. Certain constituents cannot be protected against biochemical action by any chemical means which does not interfere with the analysis. These constituents are the so-called nutrients, and include all forms of phosphorous and most forms of nitrogen and sulphur. Samples in which these analyses are desired can be preserved by cooling, or by adding H_2SO_4 to a pH of less than 2 and then cooling.

Table 4 lists various preservation techniques for common parameters in a waste characterization study.

2.3.4 Sample Collection and Handling. Some precautions and general rules for operating a successful sampling program are (4,5):

a) Samples should be taken at a place in the collection system where the wastewater is well mixed, such as near a Parshall flume or at a location in a sewer with hydraulic turbulence (e.g., close to the lateral inflows or vertical drops). Weirs tend to enhance the settling of solids immediately upstream and the accumulation of

TABLE 4 SAMPLE PRESERVATION (5)

Parameter	Preservative	Maximum Holding Time
Biochemical Oxygen Demand	Cool to 4°C	6 hours
Suspended Solids	Cool to 4°C	7 days
Total Phosphorous	Cool to 4°C	24 hours
Total Kjeldahl Nitrogen	Cool to 4°C, H_2SO_4 to pH<2	24 hours
Oil and Grease	Cool to 4°C, H_2SO_4 to pH<2	24 hours
pH	On-site determination	---
Temperature	On-site determination	---
Acidity-Alkalinity	Cool to 4°C	24 hours

floating oil or grease immediately downstream. Such locations should be avoided as sample sources.

b) Samples should be taken in the center of the channel of flow where the velocity is highest and the possibility that solids have settled is minimal. To avoid an excess of floating materials, the mouth or head of the sample intake line should be placed a few inches below the water surface.

c) The ability of a sampling device to collect solids is of particular importance, because equipment with a limited capability in this area may lead to an erroneous estimate of pollutant yields. The solids intake capacity of a sampling device is affected by the orientation of the sampler intake and the velocities in the intake nozzle and line. A sampler intake is typically suspended from the top or oriented axially pointing downstream. These intake arrangements are usually necessary because of the logistics of the sampling station and to prevent clogging of the intake. However, the overall effect is that the concentration of solids in the collected samples is underestimated.

Intake line velocity is another factor in the design of a sampler which affects the solids collection capability. The line diameter must be reasonably large to avoid clogging (10 mm or 3/8 inch) and a velocity sufficient to transport particles should be maintained (1 m/s) over the working head of the sampler. Suppliers of commercial samplers should be prepared to supply information of this nature to prospective clients.

d) A sampler should have built-in features which alleviate the possibility of cross-contamination of samples. Two design features are available: a) the sampler air-purges the intake line preceding each sampling cycle; b) separate intake lines are provided for each individual sample.

e) The sample container, intake and line should be clean and uncontaminated prior to initiation of each sampling period.

f) The type of sample containers used in a monitoring program (i.e., material used in the manufacture of container) may affect the analyses being performed. Laboratory personnel should verify the use of acceptable sample containers.

g) Samples submitted to the laboratory for analyses should be clearly labelled with the following information:

 i) designation or location of sample station,
 ii) date and time of collection,
 iii) indication of grab or composited sample,
 iv) interfering or hazardous constituents.

The duration of a wastewater characterization program is normally a function of the size of the facility being monitored and the variability of waste characteristics. Upon firmly establishing quantitative and qualitative characteristics at an installation, design of a treatment system may proceed. Regulatory agencies often stipulate the number and nature of samples to be collected in a wastewater characterization program, as well as the time and duration of the sampling period.

A wastewater characterization program should last for a sufficient period to establish reliable design criteria. In situations of high flow variability, e.g., tourist or commercial areas, wastewater characterization may have to be undertaken on a seasonal basis to establish operating limits for a system.

When characterizing waste generated in a small community, the sequential composite sampling method is perhaps the most revealing monitoring approach. Utilizing this method, diurnal fluctuations in wastewater loadings may be established, and the system may be designed accordingly. Depending on the variability of the waste stream, 4, 8 or 12-hour composite samples may be collected and submitted for laboratory analysis. The interval between sampling events will be variable if the sampler flow-weights samples automatically (i.e., electronically activated by flow measuring devices). If the sampler is activated by a timing device, the samples should be flow-weighted manually and the sampling frequency should not exceed one hour.

2.3.5 Summary. There are basically two sampling methods, grab and composite, which are used in wastewater characterization programs. Grab samples indicate waste characteristics at a specific time and location in a waste stream, whereas composite samples indicate average pollutant concentrations over a sampling interval. Flow-weighted composite samples are more representative of actual waste conditions than simple composite samples. Sequential composite samples (flow-weighted) give a good indication of diurnal sewage characteristics. Local regulatory agencies may be contacted for specific information on required duration and time of sampling programs and parameters to be analyzed. Analytical laboratories should be consulted regarding the volume of samples required to perform required analyses, acceptable methods of sample preservation, and the type of sample containers required.

Automatic samplers used should be of the refrigerated variety, and designed and operated to facilitate the collection of representative samples. The prices of commercial automatic samplers vary with the degree of flexibility and automation provided. A portable unit with a mechanical timing device and hand evacuation pump costs in the neighbourhood of $500, whereas a refrigerated stationary sampler, complete with electric vacuum pump, flow and time initiated sample switch, purge cycle, adjustable sample volume regulator and 24 polyethylene bottle containers (500 ml) costs approximately $3500.

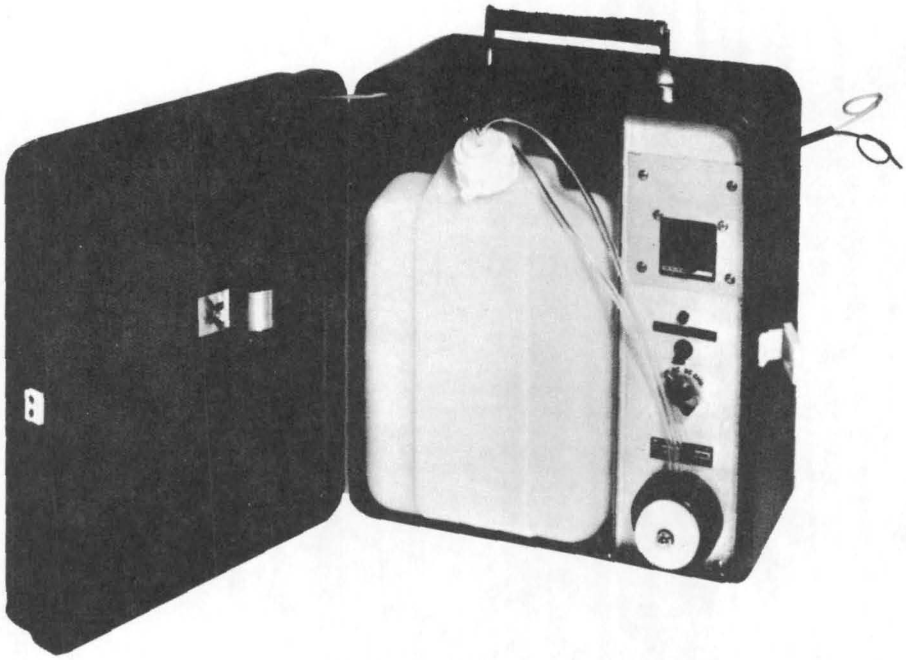
Figure 11 illustrates the variety and type of samplers which are available commercially.

2.4 Estimating Wastewater Loadings

In planning sewage treatment facilities, wastewater loadings must be projected through the useful life of the proposed system (15-25 years). This entails the investigation of population projections, land-use and zoning plans, and economic trends. After determining what is believed to be the ultimate growth projection for the design period, unit wastewater generation rates can be applied and a waste loading projection calculated.

Many tables have been compiled by different authorities to reflect wastewater generation rates. These estimates often differ considerably in magnitude and the designer must exercise judgement when using them. Local regulatory agencies should also be consulted to determine whether design criteria are specified for the location of the planned installation.

Table 5 has been extracted from the sewage disposal regulations of British Columbia (6) and is provided for general information to illustrate typical estimates of wastewater generation rates. Local regulatory agency criteria prevail over any figures presented herein.



a) Simple Composite Sampler With Time Control (from Sigmamotor Inc.)

b) Refrigerated Sequential Sampler (from Isco)

FIGURE 11 SAMPLING DEVICES

c) Sequential Sampler With Flow or Time Control Option (from Manning Environmental Corporation)

FIGURE 11 SAMPLING DEVICES (CONT'D)

TABLE 5 ESTIMATED MINIMUM DAILY SEWAGE FLOWS IN IMPERIAL GALLONS (6)*

Type of Facility	Estimated Minimum Daily Sewage
Apartments + condominiums (having one common entrance)	165 for 1 bedroom unit. 225 for 2 bedroom unit. 250 for 3 bedroom unit.
Houses, duplexes (all other residential units)	250 for 1 and 2 bedrooms. 300 for 3 bedrooms. 375 for 4 bedrooms. 450 for 5 bedrooms. 550 for 6 bedrooms.
Mobile home parks	250 per space.
Hospitals and laundry	250 per bed.
Hospitals without laundry	150 per bed.
Institutions, work camps, rest homes, residential schools	50 per bed.
Nursing-homes	150 per bed.
Motels/hotels	70 per unit. 100 per housekeeping unit.
Camp-sites	100 per unit. 150 per unit (year-round sites)
Theatre/drive-in (food service is limited to single service containers)	5 per space.
Fixed seat assembly (theatres, churches)	2 per seat.
Restaurants, dining-rooms, dinings-lounges	2 per square foot of dining area.
Banquet and meeting rooms	0.35 per square foot of floor area.
Beer parlours, cabarets, neighbourhood pubs	3 per square foot of customer seating area.
Swimming-pools	5 per person based on design bathing load**.
Summer camps	35 per bed.
Office buildings	20 per worker.
Factories, with showers	20 per worker per shift.
Factories, without showers	10 per worker per shift.
Schools, primary and elementary	15 per student.
Schools, high	20 per student.
Service-stations	125 per single hose pump. 250 per double hose pump.
Shopping centres (exclude cafés and laundries)	0.15 per square foot of enclosed sales area.
Laundry	350 per laundry machine.

* Conversion factors: 1 Imp. gallon = 4.5459 L

1000 L = 1 m^3

** Design bathing load is calculated as $\frac{D}{27} + \frac{S}{10}$ where D = area of pool more than 5 feet deep and S=area of pool less than 5 feet deep.

NOTE: The estimated daily sewage flows for facilities not mentioned in this table may be determined by the Medical Health Officer or his delegate. The above table gives minimum estimated daily sewage flows. The Medical Health Officer or his delegate may increase these estimated flows if circumstances warrant this in any specific application.

3 ON-SITE TREATMENT AND DISPOSAL

The treatment and disposal of wastewater in rural or isolated regions of Canada often require facilities which must function almost "operator-free". At the same time, the facility must be capable of operating efficiently and aesthetically under a variety of flow conditions, including hydraulic and organic shock. These two design requirements have traditionally resulted in the construction of what is commonly referred to as an outhouse or, alternately, the installation of a septic tank-subsurface disposal system. Whereas these traditional devices may still be quite acceptable with respect to public health and environmental protection, most regulatory agencies now administer specific rules and regulations for siting, designing and maintaining such systems. Furthermore, these same agencies have approved the use of alternate modes of on-site treatment and disposal of domestic wastes.

Most individual homes, institutions and facilities in rural areas have pressurized water systems and are likely to have indoor plumbing, including flush toilets. The outlets of toilets, sinks, showers and other fixtures normally run into a single pipe which leaves the building and discharges into one of three receptacles: i) directly to a body of water, watercourse or drainage ditch without receiving any treatment; ii) to an on-site treatment plant such as a cesspool, septic tank or aerobic tank; or iii) to a community sewer system which transports the waste to a central treatment plant. Obviously, direct discharge to a body of water is illegal in most jurisdictions. Information related to item iii) and the systems available for sewage treatment in small communities is contained in later sections. The treatment and disposal of wastewater on-site will be reviewed in this section under the following headings: a) septic tanks; b) aerobic tanks; c) subsurface disposal of liquid effluents; and d) treatment and disposal of septage.

3.1 Septic Tanks

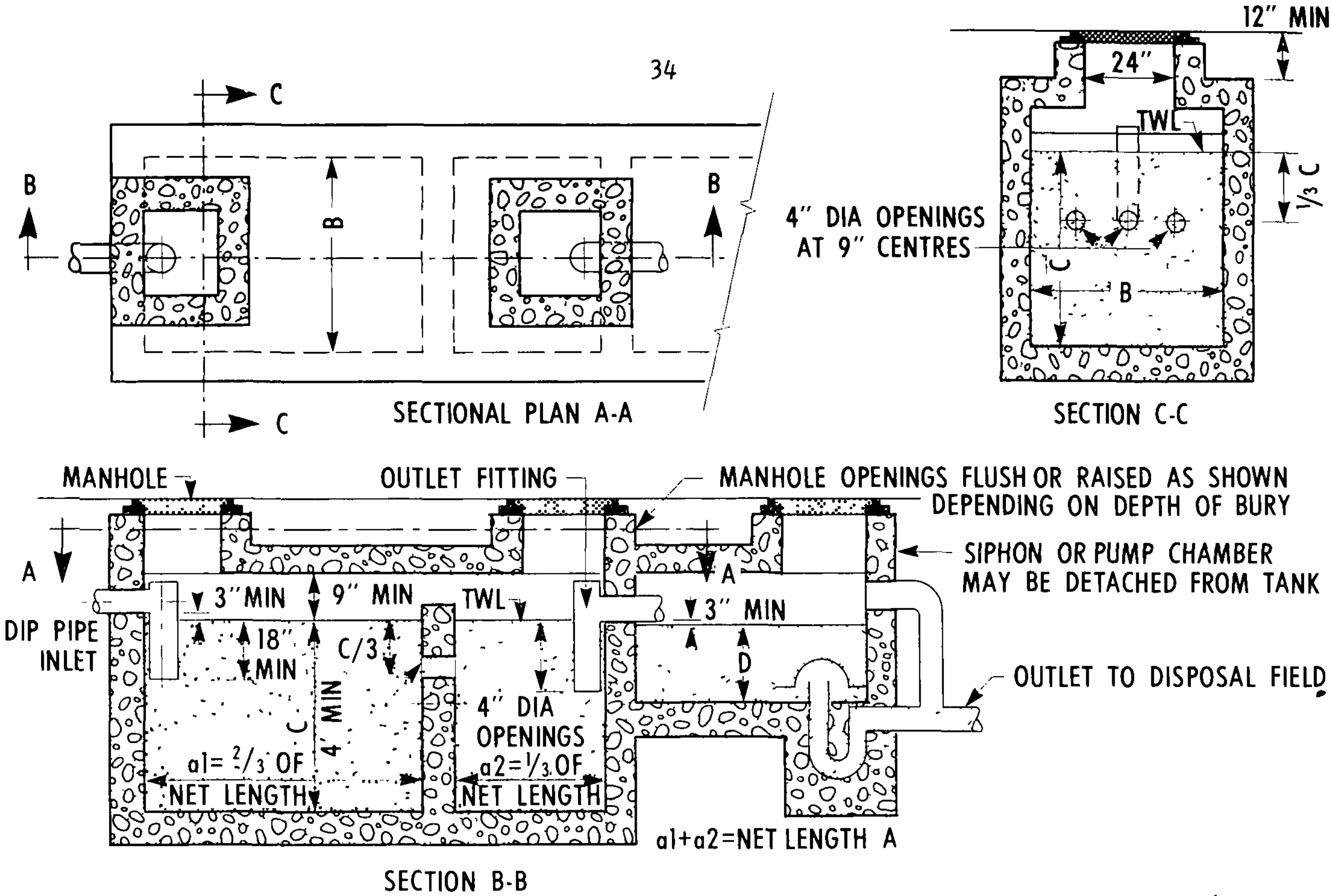
3.1.1 Function of Septic Tanks. The main function of a septic tank is to remove solids from the waste which would otherwise plug soil voids in a receiving subsurface disposal area. The waste liquid is retained in a reservoir for a designed period of time, allowing the heavier sewage solids to settle to the bottom of a tank where they form a blanket of sludge. Lighter solids, including fats and greases, rise to the surface and form a layer of scum. The retained sludge, and scum to a lesser degree, undergoes partial digestion and compaction but otherwise forms a residual in the tank (i.e., septage) which must be removed intermittently by pumping the tank clean (7).

In an efficiently operating septic tank, a liquid effluent with low solids content is discharged to a receiving subsurface soil disposal system. This liquid, highly charged with bacteria and nutrients, continues to biodegrade as it percolates downward through the soil. Physical, chemical and biological reactions within the soil matrix remove wastewater contaminants before the liquid reaches the water table. This important factor is the principal reason for establishing a minimum depth of soil filter above a water table, rock, or impervious soils.

3.1.2 **Construction.** Septic tanks should be watertight and constructed of materials which are not subject to excessive corrosion or decay, such as concrete, coated metal or fibreglass (7). Precast concrete tanks are available commercially in most areas and in several provinces must be constructed in accordance with government regulations. Construction requirements for cast-in-place reinforced concrete tanks are available from regulatory agencies. Details of a typical prefabricated septic tank are illustrated in Figure 12. As shown, this approved tank is rectangular and constructed with two compartments, the first compartment equal to one-half to two-thirds of the total volume. Although many agencies approve single compartment tanks, two compartments provide better suspended solids removal, which may provide better protection of the soil absorption system. Another important feature to note in the diagram is the positions of the inlet and outlet devices with their respective tees (or baffles).

A septic tank may be fitted with a pump or siphon to dose the leaching facility. This component adds a distinct advantage to the system as it permits uniform distribution of liquid to the entire disposal area, which is not otherwise achieved, and also permits the bed to rest between doses. A dosing device has the further advantage that it assists in preventing the system from freezing up where this might otherwise occur. Where a siphon or pump chamber is used, either as a structural extension of the tank or as separate chamber, it should be sized to deliver an acceptable dose of treated effluent to the disposal area. Regulatory agencies will specify the design volume required in each instance.

3.1.3 **Locating Septic Tanks.** The prime consideration in locating septic tanks is the protection of potable water supplies. Setback requirements for septic tank placement are specified in all septic tank standards issued by regulatory agencies.



Conversion factor: 1" = 2.54 cm

NOTES

- Manhole access shall be provided to each compartment located to facilitate servicing of the inlet and outlet.
- Baffles may be used at inlet and outlet of tank instead of dip-pipes. The top edge should be not less than 6" above TWL. and bottom edge not less than 18" below T.W.L.
- Inlet pipe may enter side wall of tank if convenient, but centre-line of pipe must not be more than 6" from inlet end wall.
- The slope of the inlet pipe should be such that inlet velocity does not exceed 3 feet per second (1" in 6' for 4" dia pipe, 1" in 12' for 6" dia pipe)
- Provision should be made for not less than 12" of cover to tank (this may be raised above general ground level when available fall to distribution system is limited)
- A siphon or pump shall be used to dose the leaching bed when more than 500 feet of distribution pipe is required
- Dimension E should be according to siphon manufacturer's requirements.
- Add 9" to dimension C for total internal depth.
- For dimensions A,B,C, see table 6
- Inspect tanks annually. Tank to be cleaned when the level of the bottom of the scum is within 3", or the surface of the sludge is within 18", of the bottom of the outlet fitting

FIGURE 12 TYPICAL SEPTIC TANK CONSTRUCTION DETAILS (7)

3.1.4 Sizing Septic Tanks. The sizing of septic tanks for individual dwellings is specified on a per bedroom or occupancy basis, as illustrated in Table 6. A septic tank must be sufficiently large to provide:

a) a 24-hour retention period for raw sewage,

b) storage for sludge and scum accumulations.

TABLE 6 SEPTIC TANKS - WORKING CAPACITY FOR HOUSEHOLD SYSTEMS (7)*

Number of Bedrooms (2 persons bedroom)	Minimum Total Working Capacity (Imperial gallon)**	Recommended Internal Dimensions Rectangular Tanks**		
		Length A	Width B	Water Depth C Min. 4' - 0"
2 or less	500	6' - 9"	3' - 0"	4' - 0"
3 or less	600	8' - 0"	3' - 0"	4' - 0"
4 or less	570	9' - 0"	3' - 6"	4' - 0"
5 or less	900	9' - 0"	4' - 0"	4' - 0"
6 or less	1080	9' - 6"	4' - 0"	4' - 6"

* The dimensions are calculated for households with an automatic washer but no garbage grinder. If a garbage grinder is used, tank capacity should be increased by 20%. Probable future use of these machines should be taken into account when tank is designed.

** Conversion Factors: 1 ft = 0.3048 m
1 inch = 2.54 cm
1 Imp. gal. = 4.5459 L

Septic tank standards provide tank dimensions and capacity in relation to sewage flow to control the velocity of flow through the unit, and to provide conditions favourable for settling of suspended solids.

For non-domestic systems or domestic systems requiring septic tanks of 4550 L capacity or more, regulatory agencies should be contacted to obtain information. Septic tank standards issued by regulatory agencies do not normally give requirements for these larger systems.

3.1.5 Operation and Maintenance. With good design and careful construction, a septic tank system will need very little maintenance provided it is used properly. With the tank capacities previously given, it should not be necessary to pump out the tank more than once every three years. It should, however, be inspected at least once a year and pumped out when necessary. Failure to pump out a septic tank when required will result in sludge or scum being carried into the leaching bed which in turn may clog and cease to function. Inspection of sludge and scum accumulation is the only way to determine when a tank should be pumped out and this is indicated if either (7):

a) the bottom of the scum mat is within approximately 8 cm of the bottom of the outlet fitting, or

b) the surface of the sludge blanket is within 46 cm of the outlet fitting.

Accumulation of sludge and scum in a septic tank is variable, and may be considerably greater during the first year of operation than at other times. The Province of Alberta advises that the rate of accumulation in a septic tank drops from about 80 litres per person per year for the first year of operation to a fairly constant rough average of 25 litres per person per year (8). These figures, of course, represent averages for a large number of tanks. Individual systems may vary considerably due to factors such as increased soap accumulation from hard water, cooking habits, use of different soaps and detergents, septic tank digestion rate, etc. A study of two-compartment septic tanks in Ontario determined that the typical sludge accumulation rate for individual domestic septic tanks was 0.22 litres person per day or 80 litres per person per year (9). To determine treatment and disposal requirements, the Ontario study suggested a septage (i.e., sludge plus supernatant) contribution rate of 200 litres per person per year. Both studies (8,9) suggest septic tank cleaning at intervals between three and five years.

In most localities contractors provide a pump-out and haulage service at a cost of $35.00 or more, depending on the size of the tank and the location. The means and place of disposal of the solids content of the tank must be approved by appropriate regulatory authorities.

3.2 Aerobic Tanks

3.2.1 Function of Aerobic Tanks. Aerobic tanks produce an effluent of higher quality than that discharged from a septic tank. With the generation of a higher quality effluent, many regulatory agencies stipulate allowable reductions in the size of subsurface disposal fields which receive such discharges. The rationale behind the reduction in disposal field area rests on the basis that aerobic tank effluents promote desirable aerobic biological activity within the soil matrix, keeping soil voids open and enhancing the percolation rate and evapotranspiration properties of the soil (10).

Aerobic tanks for on-site treatment of domestic wastewater are available commercially in one, two or three-compartment designs. Some designs incorporate a "fail-safe" sand filter to provide polishing of final effluent. In general, wastewater entering the aerobic mini-plant accumulates in an aerated chamber for a prescribed period of time, or as controlled by level sensors. After aeration, the wastewater is allowed to settle, either in a separate chamber or by automatic shutdown of aeration devices in the holding chamber. Thereafter the treated effluent (supernatant) is pumped to the disposal field. Some plants incorporate components which permit the recycle of settled sludge from the settling compartment to the aeration compartment.

3.2.2 **Construction.** Many different aerobic tank devices are on the market and available in materials such as concrete, coated-metal or fibreglass. Regulatory agencies across Canada have specifically approved the use of some of these manufactured products, but not all, and contact should be made with appropriate agencies to determine which devices are acceptable. Figure 13 illustrates the general layout of an aerobic tank system complete with a commercially-constructed filter bed.

Raw sewage enters the first part of the aeration chamber, which contains a minimum 1100 litres of mixed liquor. Here the raw sewage is mixed with activated sludge from previous treatment and oxygen from air supplied by an air blower.

The mixed sewage flows by gravity through three holes and a slot in centre baffle plate into a second part of the aeration chamber which is provided with an air diffuser similar to that in the first part of the chamber. Aeration in these two compartments is continuous.

A small amount of air fed to the diffuser header of the aeration compartments is bled off via an orifice to operate the output control lift pump for transferring the liquid from the aeration chamber to the settling chamber. This air lift pump is designed to lift water to a fixed height and to operate automatically. When there is no flow into the system, the aeration tank freeboard capacity is approximately 455 L (100 Imp gal). Briefly, the air lift pump functions according to the following principles. As raw sewage enters the first aeration chamber, the liquid level in the second aeration chamber rises and the output air lift pump starts to operate and continues pumping until the freeboard capacity is again about 455 L. An instantaneous input of 227 L would be necessary before

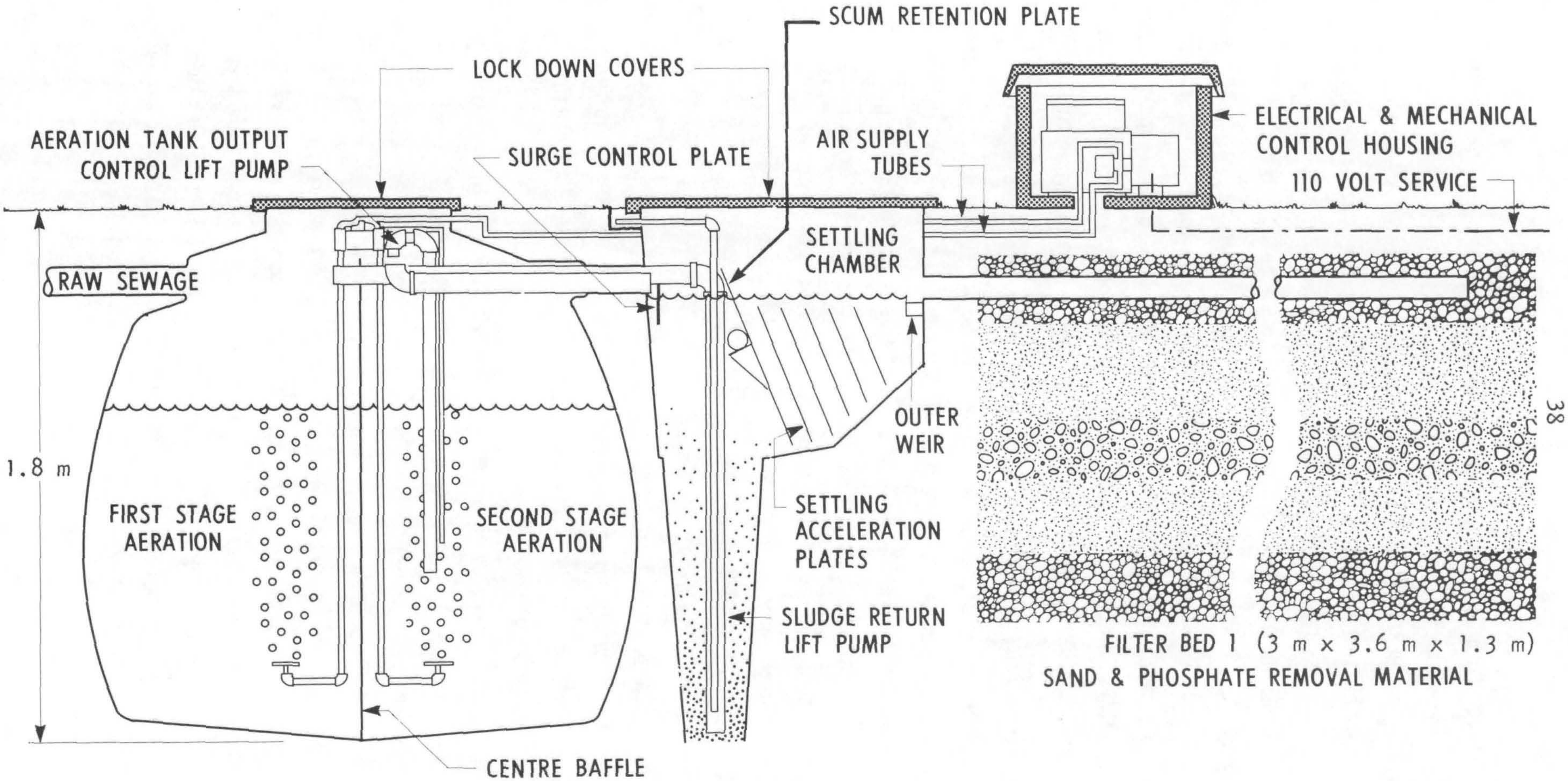


FIGURE 13 AN AEROBIC TANK SYSTEM (From Waltec Industries Ltd.)

the air lift pump would have a peak output to the settling tank of more than 13.6 L/min. The lift pump output decreases as the liquid level in the aeration tank lowers.

The settling tank capacity is 410 L. It is fitted with a surge plate to buffer incoming flow, a retention baffle for floating sludge or scum, five inclined plate acceleration baffles and a sawtoothed overflow weir. At a peak feed rate of 13.6 L/min, the minimum settling time is 30 minutes. In actual household use, settling time in the clarifier could be much longer due to use patterns and the decreasing output of the air lift pump with head reduction in the aeration chambers.

An air lift sludge return pump installed in the leg of the settling tank returns settled solids back to the first stage aeration chamber. On a pre-set periodic basis, the total blower air supply is directed from the diffusers to the air lift sludge return pump by a motorized valve acting on a timer signal. The action simultaneously washes the settling tank, walls and baffling plates, returns 273 L of settled sludge, and breaks up floating sludge. The pumpback requires five minutes and is normally activated every 12 hours.

In the particular design shown in Figure 13, the settling tank effluent passes into a sand filter 11 m^2 in area and 132 cm deep. The effluent is distributed through three plastic, perforated pipes, each 3.05 m long, and through four cross members on top of the filter sand.

Another design in aerobic tanks features a three-compartment unit. The first compartment receives household wastewater and holds it long enough to allow solid matter to settle to the sludge layer at the tank bottom. Organic solids are broken down physically and biochemically by anaerobic bacteria in this compartment. In the second compartment, a mechanical aerator mixes incoming liquid with activated sludge, at the same time injecting large quantities of air into the wastewater. The final phase of the operation takes place in a clarifying compartment where the settling of any remaining settleable material occurs (11).

Yet another design and operational mode for aerobic tanks involves a fill and draw batch treatment process (12). Wastewater entering the plant is accumulated for approximately 20-1/2 hours during which time it is aerated continuously utilizing fine bubble diffused air. Aeration is followed by a period of quiescent settling (three hours).

The supernatant is pumped out of the tank for a 30-minute period and the settled solids are retained. The process is automatic and can be programmed to accomodate hydraulic flow patterns consistent with individual home use patterns. The discharge pump is controlled by the programmed timer and a low level sensor. If the low level of the liquid is reached prior to the shutoff setting of the timer, the level sensor will activate the shutoff of the pump. In no case will pump operation extend beyond the timer setting or into the aeration cycle.

In 1970, the National Sanitation Foundation in Ann Arbor Michigan issued its "Standard No. 40" on individual aerobic treatment units (12). This standard outlines criteria for evaluating individual home units and presents a procedure for testing and certification. Plants are classified according to their ability to produce an effluent of acceptable quality under specific loading and operating conditions. A Class I plant must be capable of producing an effluent BOD of 20 mg/L and SS of 40 mg/L. A Class II plant must be capable of generating an effluent with a BOD of 60 mg/L and SS of 100 mg/L. The mechanical units discussed previously are all considered equivalent to Class II systems. The unit illustrated in Figure 13 is considered a Class I system primarily because of the polishing effect of the sand filter. Of further importance in the testing program is documentation of operation and maintenance practices and problems associated with each of the units.

3.2.3 Locating Aerobic Tanks. As with septic tanks, the prime consideration in locating an aerobic tank is protection of the potable water supply. Regulatory agencies have not, as yet, issued standards respecting the use of aerobic tanks for on-site disposal of wastewater and they should be contacted for a case-by-case assessment of each proposed installation.

3.2.4 Sizing Aerobic Tanks. The sizing of aerobic tanks for dwellings will be on an occupancy basis, as specified by regulatory agencies (e.g., total liquid volume for two days retention). Each agency will stipulate required sizes of aerobic tank devices when approving a system.

3.2.5 Operation and Maintenance. When aerobic tanks are purchased for individual dwellings, regulatory agencies will require the inclusion of a service contract in the purchase order. Such contracts require the manufacturer, or a representative of the manufacturer, to maintain the aerobic tank in operating condition by providing:

a) a specified number of service calls per year,
b) 24-hour emergency service,
c) replacement of parts,
d) pump-outs of the system as required.

The cost for service contracts is in the neighbourhood of $100 per year. Power requirements for aerobic tanks vary depending on the manufacturer and unit model chosen (10 kWh/d range).

Aerobic tanks will require removal of solids (i.e., pump-out) every three to five years, depending on loading conditions. The means and place of disposal of contents of the tank must be approved by the appropriate regulatory authority in each region.

Although long-term operating information on aerobic tanks has yet to be developed, evaluation studies conducted by recognized testing agencies in both Canada (13) and the United States (12) have revealed potential problems:

i) Organic and hydraulic shock loads applied to the system over a short period of time will upset the quality of effluent discharged from the plant. Repeated shock loads could result in clogging of distribution pipes or filter media in the receiving subsurface disposal system.

ii) Prolonged power failures have an adverse affect on the quality of effluent discharging from the plant. Repeated failure could result in clogging of the subsurface disposal system.

iii) Air supply mechanisms, either for aeration of the waste or for the operation of air lift pumps are susceptible to plugging and may require routine maintenance. The air supply system appears to be the component which requires the most attention. Frequent inspection and assurance of prompt service in emergencies is essential in order to keep the mini-plant operating satisfactorily.

iv) Filter systems incorporated into some designs play a significant role in maintaining the quality of effluent discharged to subsurface disposal areas. The useful life of filter systems may be shortened by unexpected increases in the flow of waste for prolonged periods of time or repeated shock loads. (Long-term monitoring programs are required to determine the useful life of sand filters are under normal loading conditions.)

3.3 Comparison of Septic Tanks and Aerobic Tanks

A comparison of septic tank and aerobic tank operation for individual dwellings may be made on the basis of the following parameters:

Function. Both devices are designed to treat wastewater discharged from the household. The septic tank separates solids from wastewater by sedimentation and flotation and partially stablizes retained pollutants. The aerobic tank separates large solids from incoming wastewater, stabilizes large unstable pollutant molecules to smaller, stable molecules through the action of aerobic microorganisms, and enhances desirable predatory-prey relationships between microorganisms in the tank effluent, improving infiltration and evapotranspiration in subsurface absorption systems.

Performance. BOD and SS concentrations from a septic tank are about 150 to 200 mg/L (9), whereas values for aerobic tanks can be expected to be 60 to 100 mg/L. In addition, a DO of about 1 to 3 mg/L may be expected in aerobic tank effluents (11,12,13).

Operation. A septic tank should be inspected frequently during the first year (three times) and annually thereafter. Aerobic tanks should be inspected by the homeowner at least daily during the first few weeks of operation and once every two to three months thereafter. Semi-annual or annual check-ups by qualified technicians should be specified in purchase contracts. Sludge and scum from the unit must be removed intermittently, and, depending on the type of unit and the applied load, pumping may be required every 9 to 12 months to maintain optimum MLSS concentrations in aeration chambers (13).

Cost. The capital costs for septic and aerobic tanks will vary from region to region. The installed cost of a septic tank (2275 to 4500 capacity) varies from 10¢ to 15¢ per litre and the operating cost may be from 10 to 15 dollars per year. Aerobic tanks cost from \$1.10 to \$2.20 per L installed; maintenance contracts vary from \$65 to \$110 per year, and power costs may run \$35 to \$60 per year (2.4 to 7.4 kWh/d at 4¢ per kWh). The installed cost of a sand filter for an aerobic plant is about \$160 to \$215/m^2 and maintenance will cost approximately \$10/m^2 per year. A tile field for a septic tank installation will cost approximately \$5.00 to \$6.50 per metre of tile run. Although the above costs favour septic tank installation, supplementary consideration of the soil absorption systems receiving treated effluents from these two treatment devices may give the advantage to the higher quality effluents produced by an aerobic tank.

3.4 Subsurface Disposal Systems

3.4.1 General. A process which involves land application of treated wastewater by distribution beneath the soil surface through open-jointed or perforated pipes, drains or basins is called a subsurface disposal system. As a first step in the design of subsurface sewage disposal systems, it must be determined whether the soil is suitable for absorption of a treated wastewater effluent and, if so, the land area required for disposal. This evaluation involves a number of factors, including: the ability of the soil to absorb liquid; the slope of the site; the depth to groundwater; the depth to bedrock; the likelihood of seasonal flooding; and the distances to well and surface waters (7,8,14). Tools utilized in the evaluation of potential disposal sites are listed below.

a) Test pit. The highest seasonal elevation of the local water table and the depth to any impermeable strata can be determined by a deep test pit dug in early spring and left open several days to allow the water level to come to equilibrium with the surrounding groundwater table. If the site evaluation must be made at some other time than the wettest season of the year, an estimate can be made of the highest position of the water table by examining the records of wells dug in similar soil in the area, or identification of mottling in the soil profile. Mottles are spots of contrasting colours found in soils which are subject to periodic saturation. The spots (oxides of iron and manganese) are usually bright yellow-orange-red, surrounded by a grey-brown matrix (15,16).

b) Percolation test. Evaluation of the suitability of a soil for subsurface disposal also requires a percolation test. The test, which may vary in procedure from province to province, provides an estimate of soil permeability based on the assumption that the ability of a soil to absorb sewage effluents over a prolonged period of time may be predicted from its ability to absorb clear water. Although this assumption has been strongly criticized, the percolation test remains the only practical method to determine soil characteristics for wastewater disposal in field conditions (15,16,17). Percolation test results are normally reported in minutes per cm, i.e., the time required for clean water to sink one centimeter in a hole dug in the natural soil to the depth of the proposed seepage bed, and in which the adjacent soil has been soaked to near saturation. This measured "percolation rate" is used for sizing the subsurface disposal area, an exercise in which empirical relationships between the percolation rate of the soil and the design wastewater loading rate are employed. Since the percolation rate depends largely on grain and void sizes in the soil, simplified relationships between the types of soils, percolation rates, grain sizes and void volumes can be used, as tabulated in Table 7. Soils with percolation rates of 60 minutes or less are usually considered acceptable for subsurface disposal systems.

c) Biological activity. Percolation rate is one parameter used to establish the required area for seepage beds; another important factor in system design is biological activity in the upper part of the undisturbed natural soil. If aerobic conditions are maintained in a seepage bed, in particular at the interface with natural soils, through the discharge of an aerobic effluent to the bed and/or by constant ventilation of the seepage bed, aerobic bacteria, protozoa, rotifers and nematoda maintain their pore-openings, permitting liquid effluent to percolate into the soil.

TABLE 7 GENERAL SOIL CHARACTERISTICS (11)

Percolation Rates (minute/cm)		Ranges of Particle Sizes and Voids (mm)				
		All particles 0.1 - 3.0	50% of particles 0.3 - 1.6	Average size	Size voids	% voids
0.4	(coarse sand)	0.1 - 3.0	0.3 - 1.6	0.8	0.35	33-36
2	(gritty medium sized sand)	0.06 - 2.5	0.2 - 1.0	0.5	0.30	35-40
4	(fine sand, some medium sand)	0.04 - 2.0	0.1 - 0.6	0.3	0.22	38-48
6	(fine sand and silt, some loam)	0.02 - 1.5	0.06 - 0.4	0.15	0.09	38-42
8	(silt-sand mixture, some loam)	0.01 - 1.1	0.03 - 0.3	0.09	0.05	40-45
12	(silt with some loam and sand)	0.005 - 0.8	0.016 - 0.16	0.05	0.03	40-45
18	(loam-silt mixture; heavy fertile soil)	0.002 - 0.5	0.008 - 0.1	0.02	0.01	45-50
24	(loam, with some clay and silt)	0.0015 - 0.35	0.004 - 0.06	0.01	0.007	45-50
35.5	(clay-silt mixture, and loam)	0.001 - 0.2	0.002 - 0.03	0.005	0.005	45-50
47.25	(clay with some silt)	0.0007 - 0.15	0.0015 - 0.02	0.003	0.003	50-55
71	(heavy clay with some loam)	0.0005 - 0.1	0.001 - 0.01	0.002	0.002	55-60

Some researchers claim that when the subsurface system is permanently inundated or ventilation is prevented, anaerobic bacteria accumulate as organic material in soil pores and cause system failure (11). Other researchers state that hydraulic gradient and hydraulic loading rate are the prime factors to consider in the design of subsurface disposal system (18).

Subsurface disposal systems receiving an anaerobic effluent (e.g., effluent from a septic tank) can be stimulated to recover aerobic conditions by designing a siphon or pump chamber into the system so that periods of loading and resting alternate. When inundated after loading, the disposal system becomes anaerobic, and the infiltration rate goes down. During rest periods, the distribution pipes of a well-designed seepage bed function as ventilating pipes and aerobic microorganisms become at least partially effective. Although more research is required to determine the best way to maintain aerobic conditions in the soil interface, it is known that sufficient rest for drainage and restoration of soil pore oxygen is required. The duration of the required rest period has not been well-defined, but several authors have suggested that two or three daily drainfield loadings allow sufficient rest (11,15,16).

d) Evapotranspiration. Evapotranspiration is an other consideration in the design of subsurface disposal system construction details. In seepage beds, part of the discharged wastewater moves upwards into plants and into the air and is influenced by two phenomena:

i) evaporation, and
ii) transpiration, where water is taken up by plant roots, used for plant growth and finally transpirated into the air.

To facilitate evapotranspiration in a subsurface disposal system, two important actions must occur:

i) capillary rise of water, which occurs best through sand with a grain size of 0.5 to 1.0 mm, and
ii) uptake of wastewater by plant roots.

In a well-designed seepage bed, the capillary rise will account for a 20 to 30 cm (8 to 12 inch) moisture lift, provided proper construction details are followed and sunlight is allowed to reach the bed surface to promote evaporation. A good growth of grass should be encouraged on the surface of the disposal area to permit the absorption of moisture by plant roots (15,16).

3.4.2 Tile Fields. All regulatory agencies with septic tank standards detail design information for effluent disposal systems. In most cases, the system described is a tile field (or seepage bed) consisting of perforated (or open jointed) pipes laid in trenches below the surface of the ground. Details of the design differ from region to region in Canada, as well as with the application (i.e., individual home versus commercial facility such as restaurant). Important considerations in the design of a tile field, which will be specified by regulatory agencies, include (7,8,14,17):

a) Depth of soil mantle. For the leaching bed to work satisfactorily, the maximum elevation of groundwater table, or any rock formation or layer of impervious material should be a specific depth below the bottom of the absorption trenches. This depth is specified in the range of 0.9 to 1.2 m (3 to 4 ft) throughout the country. Raised disposal fields are sometimes permitted by regulatory agencies where the soil mantle is not of sufficient depth or the soil is not suitable. Each agency will detail the design of such raised beds.

b) Lot size. Past history indicated that disposal fields will eventually fail due to soil clogging. For this reason a replacement field of an area equivalent to 100% of the initial field area must normally be reserved.

c) Soil suitability. The suitability of soil for absorbing a liquid effluent is determined by various soil characteristics. The number and type of tests to be carried out to verify acceptable conditions and determine design parameters will generally be limited to three percolation tests. In marginal situations, more tests may be required.

d) Locating tile fields. Regulatory agencies specify minimum setback requirements for tile fields. Precautions must be taken to ensure that the distance between the boundary of the tile field and any buildings, property boundaries, wells, surface waters, cuts or embankments, etc., is as recommended by these agencies.

e) Tile field design and construction. The design and construction of the installed tile field will be characterized by the following details, as specified by regulatory agency standards:

   i) location of tile fields,
   ii) interpretation of soil percolation rates,

iii) use of distribution box or header pipeline,
iv) depth, grade and spacing of laterals,
v) interconnection of lateral ends,
vi) allowable pipe sizes and pipe runs for gravity and dosing systems,
vii) maximum allowable length of absorption trench for gravity flow,
viii) construction details for absorption trench or absorption area,
ix) use of pumps or siphons as dictated by length of absorption trenches and/or topography of site,
x) quality of treated effluent discharged to the bed,
xi) quantity of treated effluent discharged to the bed.

Figure 14 illustrates a typical arrangement of a septic tank - tile field system serving a single-family dwelling. Other acceptable layouts and designs are detailed by regulatory agencies.

3.4.3 Leaching Cesspools. Cesspools (or seepage pits) are covered, open-jointed, walled pits dug into the soil. Cesspools receive raw sewage or treated effluent and discharge effluent directly to the soil in a concentrated area surrounding the pit. Since surrounding soil is kept saturated, there is little likelihood that aerobic organisms will be present. Therefore, consideration must be given to the nuisance factor (e.g., odour) and the expected life of such systems (i.e., soil clogging). As with tile fields, the likelihood of contaminating groundwaters is of prime importance in designing the system.

Site selection and evaluation should follow the same procedures as defined for tile fields. Design and construction specifications for cesspools are specified by regulatory agencies on much the same basis as tile fields. Cesspools are sized according to the type of soil (percolation rate) and the volume of wastewater. Figure 15 illustrates a typical seepage pit (14).

3.5 Non-Water Carriage Systems

Rural or isolated installations without pressurized water systems, or with pressurized water systems but unsuitable soil conditions for subsurface disposal systems, must use methods of wastewater disposal other than those previously cited. Under these circumstances, self-treatment or containment of toilet wastes so that no pollutant load is placed on the surrounding environment is employed. If water carriage of wastes is available, a holding tank in conjunction with low water-use or chemical toilets can be installed. If water carriage of wastes is not available, alternatives such as privies,

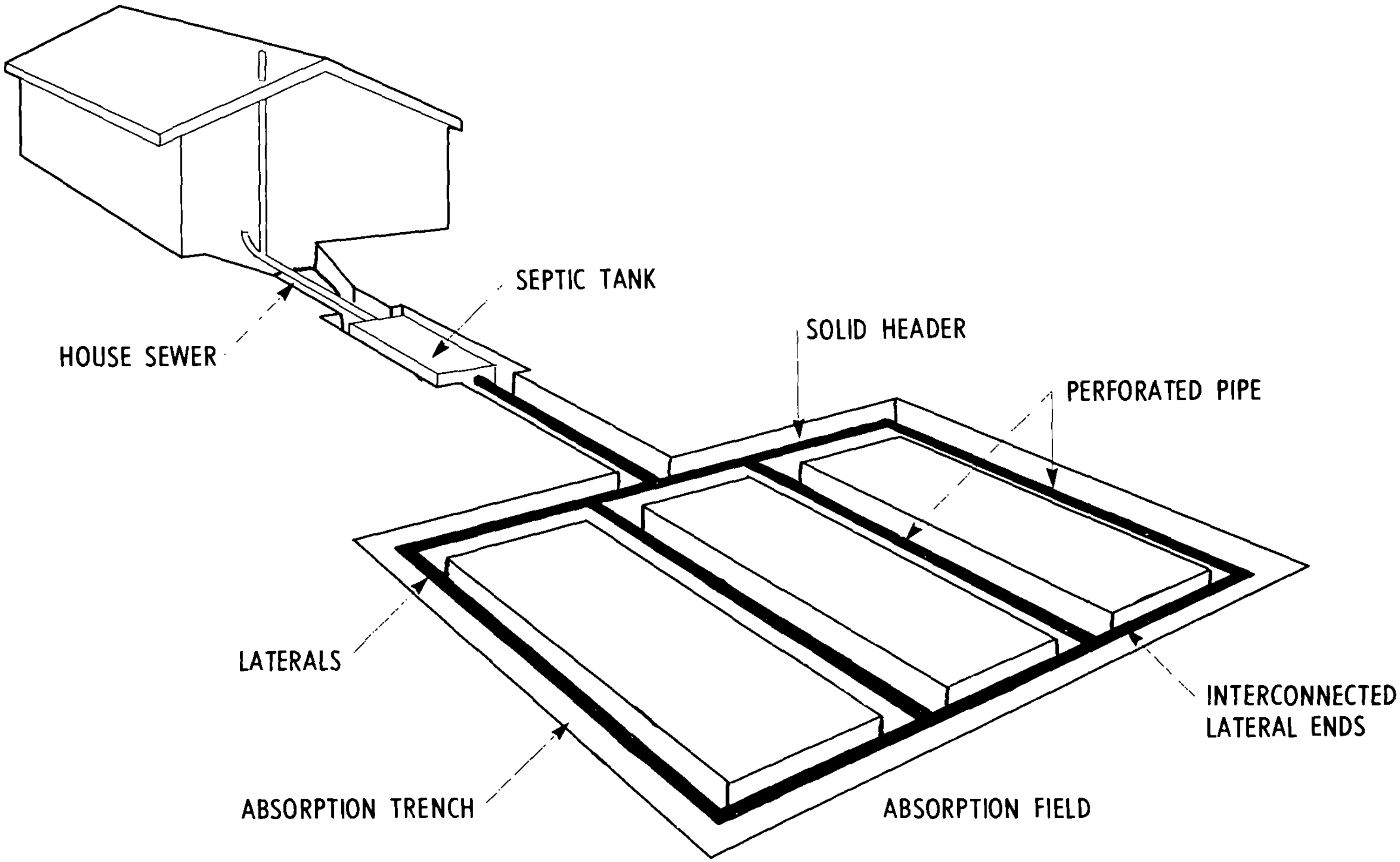


FIGURE 14 TYPICAL SEPTIC TANK TILE FIELD LAYOUT

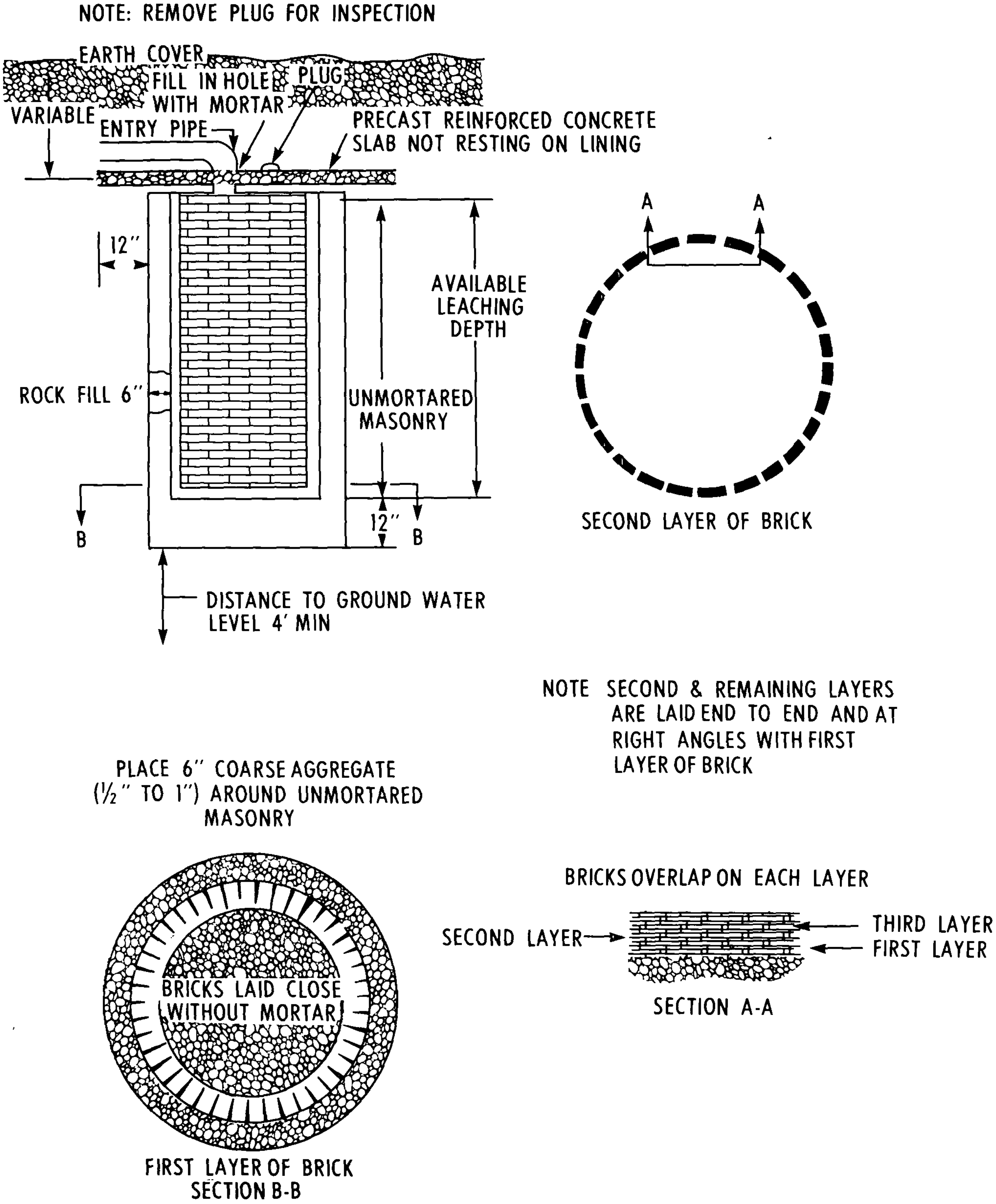


FIGURE 15 TYPICAL SEEPAGE PIT CONFIGURATION (14)

Conversion factor 1" = 2.54 cm

compost toilets, chemical toilets and incinerator toilets should be explored. The following discussion examines the alternatives available in self-contained systems.

3.5.1 Earth Pit Privy. The earth pit privy offers a method of sanitary waste disposal in an unserviced environment. It is a device constructed for the sole purpose of disposing of human wastes in a pit in the earth. The pit is covered by a structure affording privacy and shelter, and containing a covered seat with an opening into a pit.

The capacity of the pit is such that it may be used for several years without requiring the privy be moved. A minimum volume of 1.5 m^3 (50 ft^3) is recommended. The depth of the pit should be no more than four to five feet below grade. The site should be accessible to the user, ordinarily not less than 15 m (50 ft) and not more than 45 m (150 ft) from the occupied buildings. Consideration should be given to the direction of prevailing winds to reduce possible fly and odour nuisances (19,20).

The location of the privy should also minimize the danger of contamination of water supplies. The safe distance from springs or wells depends upon the nature of the soil. Under ordinary conditions the privy should be located at least 15 m from any building line or fence. The distance should be greater where the soil is sandy and less where the soil is clay. In limestone regions where crevices are prevalent or potentially present, distances should be greater.

Insects, animals, and surface water should not have access to the pit to prevent the spread of intestinal diseases. On level ground, the area around both privy and water supply should be mounded with earth. The earth mound should have a level area extending 46 cm (18 inches) away from the privy floor level in all directions (19).

Building practices for the superstructure and pit, as well as maintenance criteria, are available from federal and provincial authorities.

When any doubt exists as to the safety of the water supply, or there are other potential health hazards associated with the pit, other types of disposal should be considered.

3.5.2 Vault Privy. A vault privy is essentially a pit privy with an impervious lining and provision for removal of excreta. The unit is used where the groundwater table is close to the ground surface, or where it is necessary to prevent contamination of nearby water courses, wells, and springs.

Vaults should be constructed of concrete and should be watertight to keep out groundwater and prevent leakage. A minimum volume of 1.5 m^3 (50 cu ft) is recommended (19).

A readily accessible clean-out door is necessary. This door should be constructed to prevent access by flies, animals and surface water to the vault contents. Pump-out of the vault should be completed on a regular basis. Vault contents should be either disposed of in a public sanitary sewer system or other federally or provincially approved process.

3.5.3 Compost Toilets

Function. Composting is a process in which the microbial breakdown of organic material takes place in the solid phase. It differs from fermentation or digestion, which are liquid phase processes, in that pores or passages in the solid phase material allow the movement of gases and air into and out of the material. Composting can proceed anaerobically, aerobically or in a combination of the two processes. In general, anaerobic composting proceeds slowly with the evolution of small amount of heat. The end products are odour-causing compounds such as methane and hydrogen sulphide. Aerobic composting proceeds at a faster rate, releases more heat of reaction, and does not produce foul odours. The breakdown of organics is much more complete than in the anaerobic process with the main end products being carbon dioxide and water (21).

Compost toilets are a relatively new concept in self-contained disposal systems in Canada with little long-term operating information available. Developed in Europe, the unit basically consists of an impervious container with access chute for deposition of waste, a clean-out hatch or tray for removal of decomposed waste, a draft tube for maintenance of an odour-free environment, and a storage area in which deposited wastes are biodegraded.

Two basic design concepts are involved in the layout of compost toilets. The unit illustrated in Figure 16 is designed with chutes connecting the container to both the kitchen and toilet. A draft tube connected to the container and vented to the roof pulls air through the deposited material, providing the oxygen required for aerobic composting and evaporating excess moisture.

Temperatures within the unit must be maintained at 35° to 40°C to effect the most efficient operation (20). In an adverse environment, a heated room, or insulated and internally heated housing must be provided for the unit. As the organic material decomposes, it slides down the inclined slope to the storage chamber where it remains until it is manually removed. Because of the size of the storage chamber, and the length of time of decomposition, removals from the unit are only required once every two or three years, even when the unit is in constant use. The large size of the container and

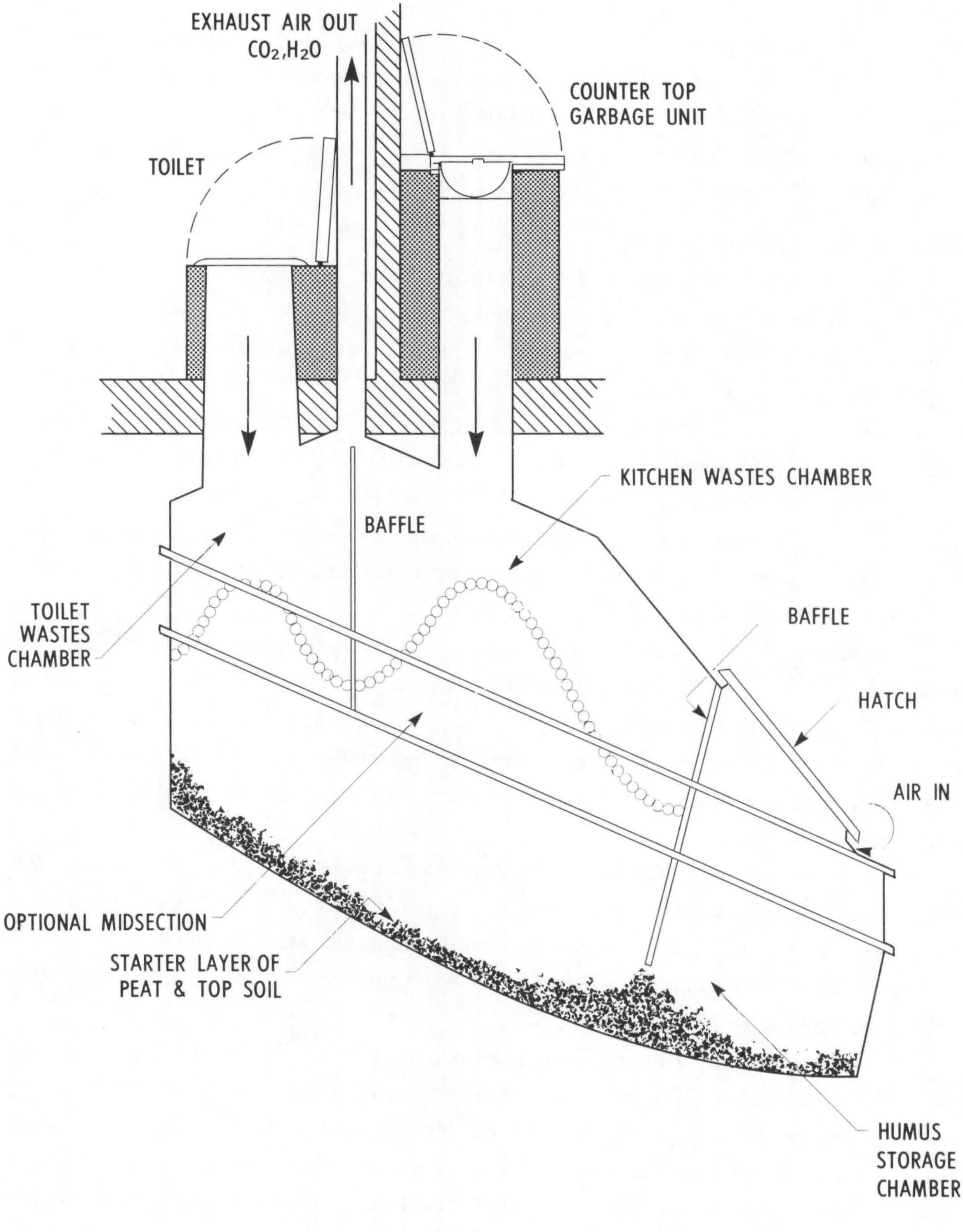


FIGURE 16 COMPOSTING TOILET (From Clivus Multrum U.S.A.)

volume of material retained allow this type of unit to function under wide loading fluctuations. However, size can also be a major drawback, since the composting container is too large to install in homes already constructed and presents difficulties in those which lack a basement (20,21,22).

Figure 17 illustrates a smaller compost unit which utilizes electric heating coils and forced ventilation to promote rapid decomposition of waste. The heating coils located in the waste storage area maintain an optimum temperature during operation and the process is kept aerobic by air intakes at the bottom of the unit and an induced draft fan installed in a draft tube connected to a roof vent. The theory behind this design is that, by providing an ideal environment for the microbial process, the waste can be stabilized in a relatively short time, and the size of the composting unit can be kept small (21). The units are, in general, sized to fit in a normal bathroom. Installation is relatively easy since they sit on the floor and require only an exhaust vent and a power source. Most of the units require only slightly more floor space than a standard water closet.

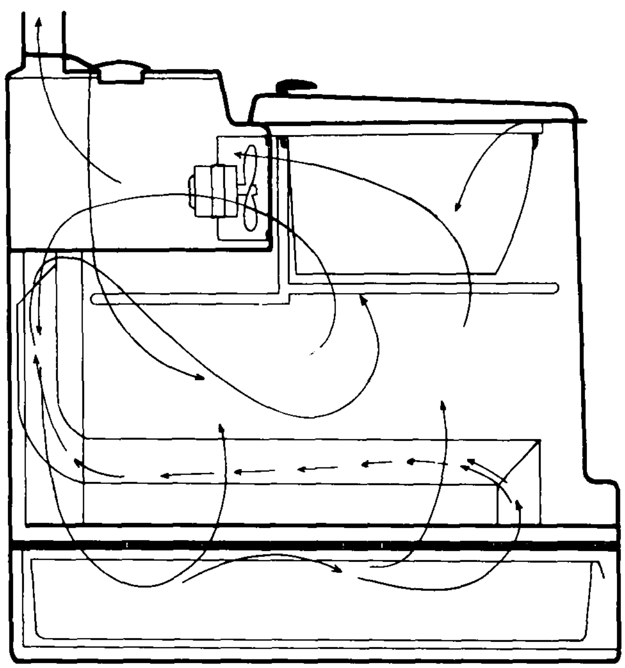


FIGURE 17 SMALL COMPOSTING TOILET (From Humus Toilets Corp. Ltd.)

Construction. Compost toilets are made of durable plastic and are watertight. The larger compost units (for human excreta and kitchen waste disposal) require approximately 4.5 m^2 of floor area (divided over two levels), while the small units require 0.7 m^2 of floor area on one floor level (20).

Various manufactured models are available. Family-sized compost toilets may be purchased with sufficient capacity for daily use by three to four persons. Larger units with capacities up to 15 persons per day are also available.

Operation and maintenance. The major area of concern with compost toilet devices is their ability to stabilize upon start-up and remain stable during operation. Monitoring conducted on a number of units at various locations in western Canada has indicated that the small electrical toilets are able to function well under ideal conditions (21,22). However, because of the limited capacity of the composting container, they are easily upset by organic and liquid shock loadings. Since the units rely on evaporation to remove moisture, excessive input of urine, such as might occur during a party at the home, can easily saturate the compost preventing oxygen penetration. Once anaerobic conditions are established, unpleasant odours are produced and eventually become a problem within the home (more problematic on small units). A toilet that has become septic requires considerable time to re-establish aerobic conditions.

Excess humidity within the waste containers in some small units has caused corrosion of electrical connections, resulting in failure of fans and heating equipment (20). Under-used toilets do not generally have problems as long as the compost is moist enough to keep the microbial populations alive. If the compost dries out the microbial action slows or stops.

Other problems which may occur with composting units include (21,22):

1) Uneven distribution of moisture in the humus, which causes portions of the compost to dry out and cake. Under these conditions, the microbial action ceases in the dry portions of the compost and the compost heap continues to build up instead of breaking down and falling through screens into the humus collection pans.
2) Sealing materials between bonded sections of small units can break down, allowing liquids and odours to seep out of the unit into the house.
3) If vents are not properly installed and insulated in the attic and above the roof, condensation will occur, and evaporated moisture will return to the unit and saturate the humus. Also, improper sealing of the exhaust vents will allow odours to escape into the house.

It has not yet been determined whether units which continually receive pathogenic organics can produce compost which can be disposed of without special precautions. This factor prevents the endorsement of compost toilets for widespread use.

Cost. The large units illustrated in Figure 16 cost approximately $1750 to $2500. Small electrical units are in the medium price range ($500 - $1000). Maximum power usage in the small units ranges from 1.2 to 8.75 kWh/d.

3.5.4 Chemical Toilets.

Function. Chemical toilets are self-contained holding units designed for use in areas where water supply is limited and/or disposal of wastewater by subsurface disposal systems is unacceptable.

Some chemical toilets utilize strong caustic solutions to stabilize human waste. Basically, waste is received in a holding tank charged with a strong lye solution, and the high pH of the chemicals solubilizes the waste and destroys bacteria, preventing decomposition and to some extent reducing odours.

A variation of the chemical toilet is the recirculating chemical toilet. These toilets may be activated electrically, by compressed air, or mechanically, depending on the make and model, and utilize a chemical solution (odour control) or chemical concentrate as flush liquid. The units are designed so that a designated number of flushes may occur before removal of stored waste and replenishment of flush liquid is required. The number of flushes accommodated is dependent upon the size of holding tank (from 4 to 2400 L) and the component equipment provided.

Chemical toilets may be portable (Figure 18) or installed as permanent fixtures (Figure 19). Chemical toilets must be used in combination with a haulage system, whereby waste is removed from the holding tank and transported to an acceptable disposal site.

Chemical toilets offer considerable advantages in remote or isolated areas. Considerations such as the requirement for limited water-use, the storage and recirculation capabilities of the unit, and the absence of any requirement for process control or power supply give the chemical toilet advantages over other on-site disposal systems. Disadvantages associated with chemical toilets include the necessity for users to maintain a supply of chemicals for frequent charging of holding tanks, to clean and/or replace filters in recirculating toilets, and above all, to have wastes removed from the toilet for disposal.

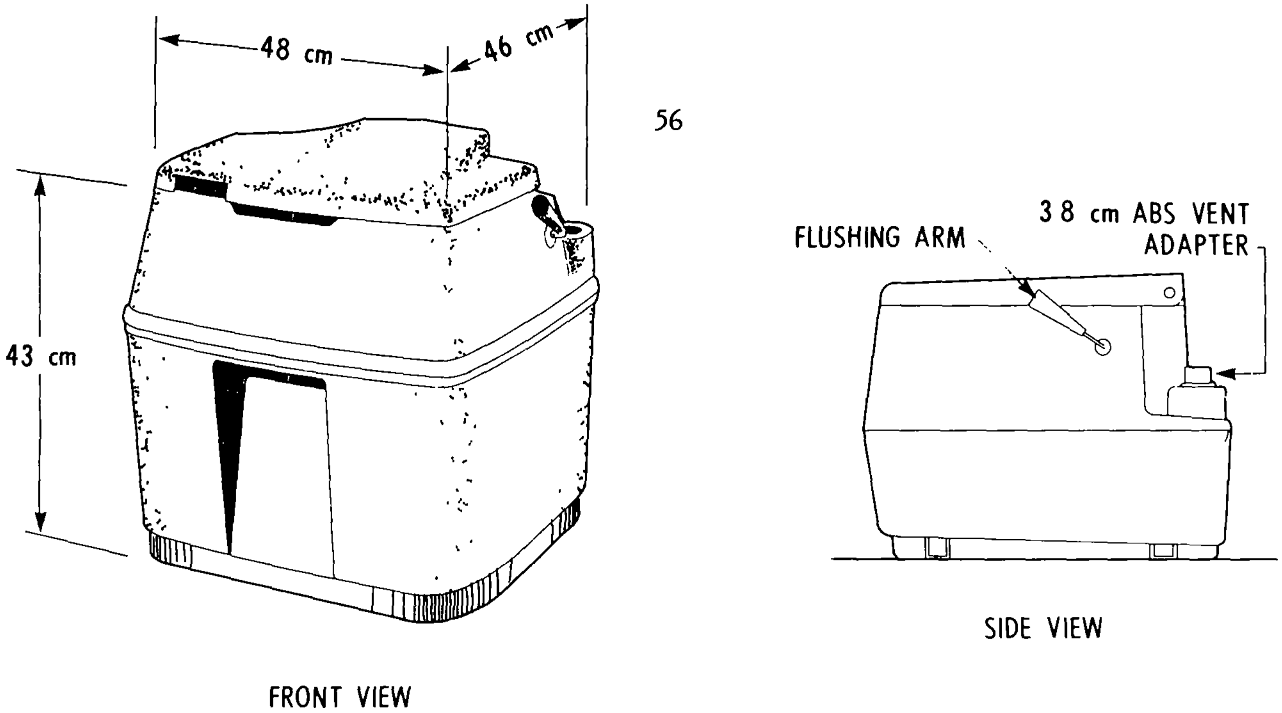


FIGURE 18 PORTABLE CHEMICAL TOILET (From Sanitation Equipment Ltd.)

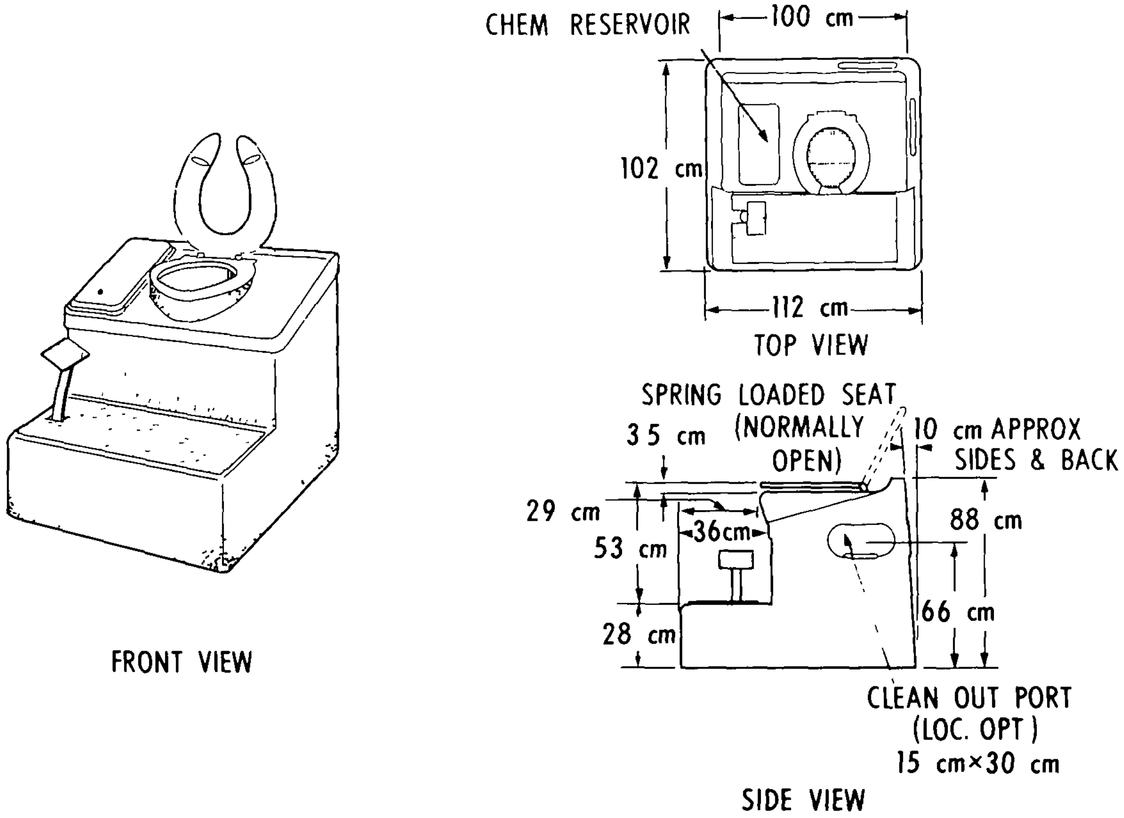


FIGURE 19 RECIRCULATING CHEMICAL TOILET (From Monogram Industries Inc.)

Construction. Chemical toilets are watertight units constructed of non-corrosive materials such as hardened plastics or stainless steel. Portable, fixed, and fixed recirculating toilets are available commercially. Portable units weigh between 4.95 kg and 7.46 kg, and do not require external water pressure or power, nor an external water supply tank. Fixed units may weigh up to 93.3 kg, depending on the make and model (20,21,23).

Sizing of chemical toilets. Chemical toilets are sized according to the number of available flushes or uses, i.e., they are designed with a specified holding tank and chemical charging reservoir capacity (including chemical toilets with recirculation systems). When selecting a particular model of chemical toilet, the number of people using the service, as well as the duration of the service, must be considered. Toilets that can accommodate up to 1000 flushes before a pump-out are available.

Operation and maintenance. Chemical toilets require inspection daily to ensure proper operation and availability of flush fluid. Stored wastes must be removed periodically and chemicals replenished. Depending on the size of the holding tank, waste material must be removed either manually or via a mobile pump-out container. Electric chemical toilets may be operated on a 12-volt DC battery or 110-volt AC power outlet.

An interesting and problematic aspect of chemical toilets is the disposal of collected wastes. In most cases, disposal of wastes from chemical toilets may be undertaken on the same basis as disposal of septage from a septic tank. The discharge of chemical toilet wastes into a biological wastewater treatment system can affect the efficacy of the system (24). The effect of odour control chemicals on biological sludge has been evaluated by monitoring the rate of substrate removal in terms of soluble organic carbon in the presence of chemical additives. Although conclusions were general, it was noted that the biological treatment process was most sensitive to heavy metal compounds. All chemical formulations had deleterious effects on the activated sludge process when present in sufficient quantities. It was also noted that shock loadings of these chemicals adversely affect the settleability of sludge in a municipal waste treatment plant (24).

Costs. Portable chemical toilets are available commercially throughout Canada. Depending on the size of the unit, prices will fluctuate between $30 and $100. Chemicals used with these units are available in 55-g packages (granulated) and 0.5-L bottles (chemical concentrate). Costs for chemicals will range from 50¢ to 90¢ per holding tank charge depending on the size and nature of the toilet.

Chemical toilets of the permanent variety cost anywhere from \$750 to \$1000, again depending on the size (45 to 273 L) and complexity of the unit. Chemicals are available in 0.2 m^3 and 25 m^3 containers (chemical concentrate) or in cartons of 1-L bottles (i.e., 12 1-L bottles per carton). Chemical costs per holding tank charge will range from 50¢ to \$7.00.

3.5.5 Incinerator Toilets.

Function. Incinerator toilets use either electricity or hydrocarbon fuels to incinerate human excreta to a minute, inert ash, which must be removed periodically from the unit. An air inlet and an exhaust system are required to oxidize the organics and evaporate moisture. Otherwise, the unit requires no holding tanks, pump-out facilities or water supply system. The operating cycle of incinerator toilets is activated automatically by closing the seat cover or by pressing a button on the unit (20,25).

Incinerator toilets are practical at installations where a supply of power and/or fuel is available and where the disposal of an unstabilized waste material is a problem (i.e., poor soil conditions, no acceptable disposal site in the area). In remote or isolated installations, the operating cost will be a major concern because power must be generated on-site and/or fuel must be transported (21).

Careful installation of an insulated ventilation system is required in all incinerator toilet installations. Some manufacturers provide carbon filters as part of the exhaust system to control the venting of odours generated by the oxidation process (20,21).

Construction. All components of the incinerator toilet are constructed of stainless steel. Exterior casings are fibreglass or porcelain enamel. Figure 20 shows the various components of an incinerator toilet.

Sizing of Incinerator Toilets. Incinerator toilet combustion cycles (i.e., from time of waste deposit to termination of oxidation) average approximately 45 minutes per cycle. Based on this parameter, the daily capacity of one unit is in the range of four to six persons (21).

Operation and maintenance. Routine inspection is an important operational consideration when employing the incinerator toilet. Unless the unit is kept in peak operational service (i.e., complete oxidation of organics, evaporation of moisture) odour problems can be expected within the home. This is one of the disadvantages of this type of unit; the user should be mechanically capable of servicing the incinerator toilet, or emergency servicing

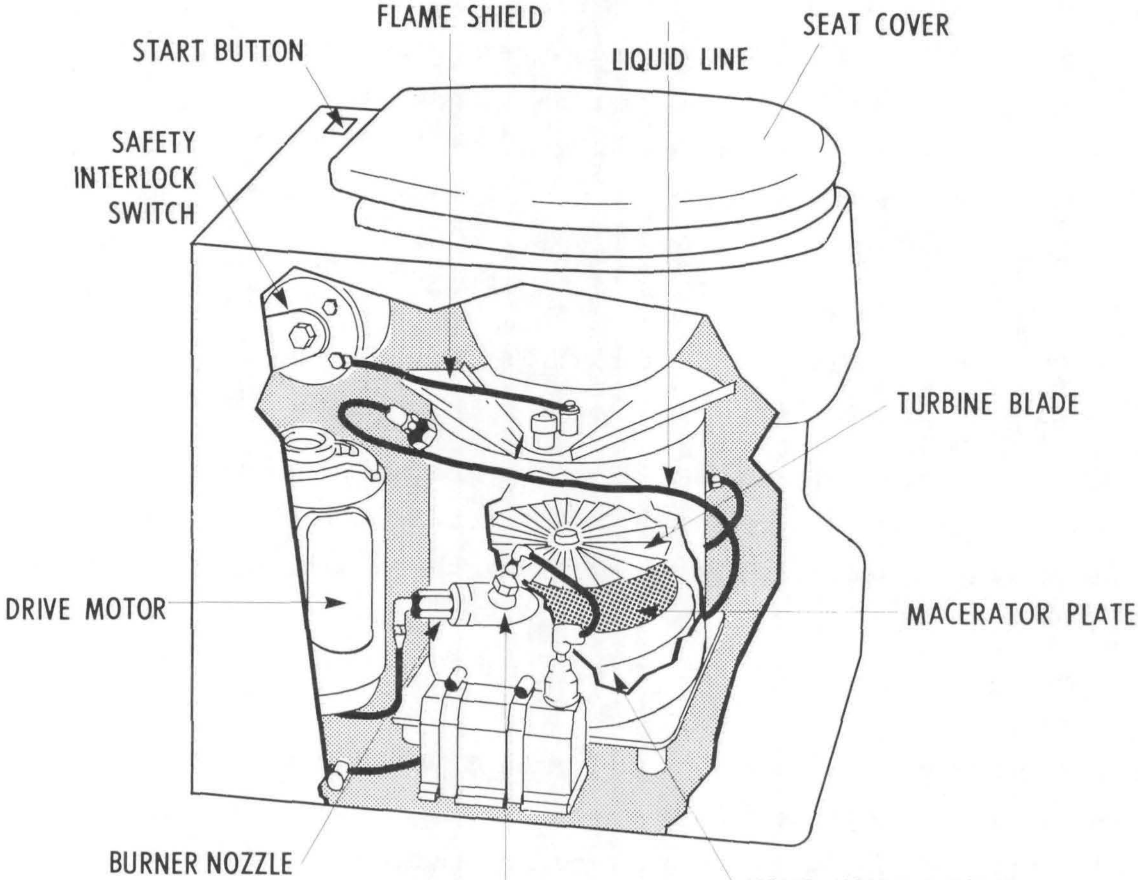


FIGURE 20 INCINERATOR TOILET (From Tekman Corp.)

should be readily available in the event of breakdown. Cleaning the unit and removing accumulated ash will be required periodically (21).

Costs. Unit costs of incinerator toilets are in the \$700 to \$900 range. Operating costs include electrical, gas or oil, and servicing costs. The operating cost has been estimated at 4 to 8 cents per cycle (21), but these estimates have not been substantiated.

3.6 Holding Tank and Sewage Haulage Systems

3.6.1 Function. A holding tank is used to store sanitary sewage until a transport vehicle removes the wastewater to an authorized disposal site. In other words, the holding tank takes the place of a sewer system by receiving and storing liquid waste until it can be collected by a pump-out vehicle. Holding tanks are often combined with low water use toilets to minimize the quantity of water and, as a result, the frequency of waste collection. As discussed in a previous section, chemical toilet systems may also be used in individual premises when considering the employment of a sewage haulage system (21,25,26).

3.6.2 Holding Tank Construction. To perform satisfactorily, the holding tank must meet the following standards (25,26):

a) The tank must be adequately sized to receive the amount of sewage produced between removals. Reserve capacity should be provided for greater than average volumes of wastewater or for delays in the normal removal schedule that can reasonably be expected to occur.

b) The tank must be structurally able to withstand all pressures and forces it may experience.

c) Adequate anchorage must be provided to prevent movement of the tank. If the tank is above ground, forces caused by wind, waves and/or accumulated snow and ice must be withstood. Below-ground tanks must be anchored to counteract buoyancy forces in cases of a high water table.

d) It must be possible to seal all fittings and openings on the tank to withstand internal pressures of at least 34.5 kPa (5 psi). Overflows of the tank which would act as pressure relief systems cannot be permitted for reasons of public health and environmental protection. Hence, each system must be checked to determine the maximum pressure which could be developed due to overfilling. For example, where the holding tank is located at a much lower elevation than the premises and vented through the premises' plumbing system, overfilling the tank could cause back-up in the inlet sewer and increased pressures on the tank. Similar problems could arise with pumped systems when a tank is pumped full and the air vent is automatically closed. This pressure could equal the difference between the discharge head of the pump and the static head between the premises and tank.

e) The tank must be sufficiently durable to function adequately for the design life of the system. Corrosion-resistant materials suitable for the internal condition of septic sewage and the external exposed condition must be used to construct the tanks or to coat less resistant materials used for construction. The design life of the tank may vary with the location. The fact that a holding tank may be readily replaced or duplicated, especially if located above-ground, and that holding tank systems may not be regarded as the ultimate solution in many situations, suggests that there will be a market for a shorter life and presumably less costly tank, as well as for durable tanks. Six types of materials have been used for holding tank construction: steel, concrete, fibreglass, reinforced plastics, rubber impregnated synthetic fabrics, and polyethylene. The advantages and disadvantages of each material must be compared for each situation and location.

3.6.3 Holding Tank Transfer Systems. Obviously, the most desirable method of transferring sewage to a holding tank would be a gravity system. However, some premises may be located where the topography and ground conditions prevent the installation of a gravity system, or where access by a transfer vehicle is impossible. In such instances, a small lift station may be required to transfer the sewage from the premises to the holding tank (26).

A number of systems have been used to transfer sewage from a holding tank to a transfer vehicle, including:

a) pumping wastewater from the holding tank to the transfer vehicle with a sewage pump;
b) pumping sewage by means of a pump on the transfer vehicle;
c) pressurizing the holding tank;
d) vacuum pumping by means of a vacuum pump on the transfer vehicle;
e) systems based on the removal of a full tank and replacement with an empty tank.

3.6.4 Piping, Couplings and Valves. All piping used in a holding tank sewage haulage system must be watertight, have no easy means of disconnection, and be protected against freezing and vandalism. The hose connecting the holding tank and the transfer vehicle should be a reinforced, rubberized line (7.6 cm or 3-in diameter), capable of withstanding either pressure or vacuum stress. Quick coupling fittings are useful to form joints between the holding tank piping and flexible hose of the transfer vehicle. Any valve used in the system must be rugged and able to withstand all weather conditions. The valve recommended is a plug valve that can be operated with a wrench (26).

3.6.5 Holding Tank Controls. Controls are required on the holding tanks to perform two basic functions (26):

a) A device is required to indicate when the holding tank should be emptied and to set off an alarm to inform the owner; the device must also shut off the pump and lock it out of operation.

b) A method to determine the volume of sewage in the holding tank is also required to check the quantity of sewage removed by the haulage contractor. A calibrated dip stick is a simple method of determining this volume.

3.6.7 Size of Holding Tanks. The required size of holding tank can be expressed by the equation:

$$Q = f \times q \times p$$

where:
Q = size of holding tank,
f = safety factor providing reserve capacity beyond established requirement,
q = estimated total quantity of wastewater, produced daily from the premises based on types of facilities, habits and number of inhabitants,
p = maximum normal period of time between emptying of holding tanks expressed in days.

The value of the safety factor "f" is related to the reliability of the estimated sewage production rate and the extent to which it could be exceeded, as well as the reliability of the removal service. Arbitrarily, a delay in the regular removal schedule of five days may be allowed for in reserve tank capacity. This reserve capacity also allows a reasonable time for the repair or replacement of damaged equipment (26).

The estimated daily wastewater production rate "q" is dependent upon many factors, including the average number of people in the premises, their water use habits and the plumbing fixtures involved. Case-by-case evaluation is the only method of properly assessing the wastewater production rate. It is assumed that the average values obtained in this manner will be accurate, except for occasional brief periods of higher rates resulting from a party or guests at the premises. Reserve capacity for this type of occurrence need not be included in addition to the five-day reserve for haulage equipment breakdown. It is considered unlikely that all unusual events will occur simultaneously.

The factor "p", representing the normal period of time between emptying of holding tanks, is dependent upon a number of variables, including the number of premises requiring pumpout service, capacity of the holding tank on the haulage vehicle, distance from premises to wastewater disposal site, accessibility of premises, etc. Basically, the factor "p" can be determined through cost analysis of these variables, and others which may be peculiar to the situation (26).

3.6.8 Operation and Maintenance. The major cost in operating a holding tank sewage haulage system is the cost for providing the pump-out and transfer service. Two modes of transfer are normally used: trucks and barges. A tank of up to 9100 L can fit onto a single-axle truck, a 9100 to 13650 L tank on a tandum rear-axle truck, and a 18200 L tank or larger on a tractor trailer. Self-propelled barges may be fitted with pump-out tanks of a maximum size in the 54550-68200 L range, depending upon draft limitations. Costs for transfer of sewage wastes will be based on a cost per litre of sewage transported and will vary greatly from location to location (26). As an estimate of cost, in a rural area the rate charged for removing and hauling collected wastes will be approximately 0.5 cents/L.

3.7 Treatment and Disposal of Septage

The treatment and disposal of septage in an economically feasible and environmentally acceptable manner is of increasing concern to regulatory agencies throughout Canada. Septage waste characteristics, current disposal practices and some recent research on the subject are summarized in the following sections.

3.7.1 Septage Characteristics. Septage is a highly variable anaerobic slurry containing large quantities of grit and grease, solids, organic matter, and possibly a significant concentration of heavy metals. It has a very offensive odour, the potential to foam, and poor settling and dewatering characteristics. Table 8 shows the values of common physical and chemical parameters which characterize septage (9). Although

TABLE 8 CONCENTRATIONS OF CONTAMINANTS IN SEPTIC TANK SEPTAGE (AVERAGE DATA) (9)

	Location of Septic Tank		
Contaminant	Farmhouse	Hospital Residence	Experimental Station
Total phosphorus (as P)	281	100	96
Soluble phosphorus	6	22	13
Total solids	10 780	18 610	17 136
BOD	2 747	7 810	5 496
TOC	NT*	NT	1 660
COD	NT	21 450	9 490
pH	7.0-8.7	6.8-7.3	6.2-7.4
Ammonia (as N)	89	77	27
Total Kjeldahl (as N)	1 072	390	416
Nitrite (as N)	0.01	0.03	0.16
Nitrate (as N)	0.58	0.10	0.15
Chlorides (as Cl)	77	103	122
Sulfates (as SO_4)	37	29	57
Aluminium (as Al)	3.5	6.7	4.9
Iron (as Fe)	72	31	173
Calcium (as Ca)	59	60	120
Magnesium (as Mg)	8	35	23
Sodium (as Na)	61	86	54
Potassium (as K)	32	24	10
Hardness (as $CaCO_3$)	NT	293	413
Alkalinity (as $CaCO_3$)	NT	1 465	488
Total coliform (org/100 ml)	0.63×10^6	8.4×10^5	4.7×10^5
Fecal coliform (org/100 ml)	0.5×10^6	3.7×10^6	3.7×10^6

Note: All data except pH and coliform organisms are given in mg/L.
* Not tested.

the data base is limited, an indication of the variations in septage with the source of the waste is evident. Table 9 allows comparison of some inorganic constituents found in both septage and digested and activated municipal sludge (on a percentage of dry weight basis). The inorganic components present in septage are partially affected by the characteristics of the water supply to the premises, whereas the concentration of inorganics observed in the municipal sludges may be caused by metals and other chemicals entering the plants with industrial wastewaters.

TABLE 9 SOME INORGANIC CONSTITUENTS IN SEPTIC TANK SLUDGE (% OF DRY WEIGHT) (9)

Chemical Constituents	Septic Tanks			Municipal Sludge	
	Farmhouse	Hospital Residence	Experimental Station	Digested	Activated
Chlorides (as Cl)	0.23	0.25	0.44	-	0.50
Aluminum (as Al)	0.02	0.04	0.03	3.6	1.70
Iron (as Fe)	0.68	0.17	1.05	3.8	5.00
Calcium (as Ca)	0.29	0.19	0.38	4.8	1.20
Magnesium (as Mg)	0.04	0.12	0.09	1.1	1.10
Sodium (as Na)	0.25	0.27	0.18	-	0.72
Potassium (as K)	0.10	0.08	0.04	0.36	0.72
Phophorus (as P)	2.61	0.52	0.56	0.88	1.36

NOTE: The concentrations of the components in the sludge are weighted averages of the concentrations in both of the compartments of two-compartment septic tanks.

Septage contains predominately gram-negative, nonlactose fermenters. Many of these microorganisms, such as Pseudomonas, are considered aerobic and have been found in septic tanks. Numerous obligate anaerobes are present but only spore-forming types, including Clostridium lituseburence and Clostridium perfringens, have been recovered.

The presence of aerobes in a septic tank can be explained by either the dissolved oxygen of the incoming sewage providing sufficient oxygen to allow limited aerobic growth, or by chemostatic displacement of effluent by the influent furnishing a relatively constant number of aerobic microorganisms. It is fortunate that Pseudomonas and similar aerobic bacteria are found in septic tanks, as they add limited lipid and detergent degradation capabilities (9).

3.7.2 Septage Disposal/Utilization on Land. All septage land disposal/utilization alternatives require evaluation of soil characteristics, seasonal groundwater levels, neighbouring land use, groundwater and surface water conditions, climate, and other site conditions such as land slopes and storm water runoff characteristics.

Other requirements may include availability of storage facilities for times when land application is inadvisable, crop management techniques, odour control procedures, and loading criteria. Loading criteria generally are determined as a result of agricultural considerations that restrict organic and heavy metal loadings (27).

In most agricultural areas, the nitrogen (N) available in the soil is far below the levels needed for optimum crop yield. Artificial sources of nitrogen, such as commercial fertilizer, are usually added to the soil. Septage is rich in available nitrogen and may be considered as a supplement to commercial fertilizer. The plant-available nitrogen in sewage sludge, as determined from the following formula, is important in the calculation of the application rate.

$$\text{Available N} = NH_4^+ + NO_3^- + 20\% \text{ of organic N}$$

It has been suggested that 15 to 20 percent of the organic nitrogen is converted to plant-available N in the first year of application, and 3 to 10 percent of the remaining organic nitrogen is released the second year. Decreasing amounts of organic nitrogen are released each subsequent year. All inorganic nitrogen is assumed available for plant uptake.

The reason for applying sludges at the nitrogen utilization rate of the crop is to minimize groundwater contamination due to nitrate leaching. Nitrate concentrations above 10 mg/L in drinking water may cause health problems, in particular infant methemoglobinemia (nitrate cyanosis). Nitrate pollution in surface waters will cause accelerated eutrophication of lakes and streams.

The amount of plant available N added to soils in sludge is influenced by the application method used. If sludges are applied and allowed to dry on the soil surface, from 20 to 70 percent of the NH_4-N applied can be lost to the atmosphere as NH_3. The exact proportion of NH_4-N lost through volatilization depends on soil, sludge and climatic conditions and is, therefore, difficult to predict. No NH_3 volatilization losses are assumed for sludges applied to soil by injection or incorporation methods. As a result, the rate of sludge applied to satisfy a crop's N requirement will be greater for surface than incorporation application procedures.

A more detailed discussion of sludge application rates, based upon allowable nitrogen loadings and the potential for heavy metal contamination of crops and soils, is found in Section 4 of this report.

The natural digestion process in a septic tank does not always result in a pathogen-free material; salmonella and other potentially dangerous organisms have been found in septage. For this reason, care must always be taken in handling this waste material.

Soil removes pathogens by various mechanisms, predominately filtration, soil inactivation, and die-off. Pathogen travel is usually restricted to a number of meters from point of application unless runoff or channeling occurs, possibly polluting surface and groundwater (28). Local regulatory agencies will have the necessary information concerning approved disposal practices in the province.

Methods of septage disposal on land include surface application, subsurface application and burial.

a) Surface application. There are three methods of surface application of septage on land:

   i) land spreading,
   ii) ridge-and-furrow,
   iii) spray irrigation.

   Land spreading in small communities is frequently accomplished by the same vehicle that pumps out the septic tanks. It is prudent to provide intermediate holding facilities to store the septage during or just before precipitation to prevent runoff of contaminated water. In Canada, land application of septage is limited to unfrozen surfaces to prevent runoff during thaws.

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

The ridge-and-furrow application method has been used to dispose of sludges on relatively level land, usually limited to 1.5 percent slopes. Although this method can be used to distribute septage to row crops during their growth, these crops are normally not for human consumption.

Spray irrigation of septage necessitates storing the waste in a lagoon before disposal. Portable pipes and nozzle guns are used rather than fixed or solid units. Because the septage must be pumped at 550 to 690 kPa (80 to 100 psi) through 2 to 5 cm (3/4 to 2 in) nozzle openings, a screening device at the lagoons' pump suction is mandatory. Spray irrigation also offers the greatest potential for offensive odours; thus a knowledge of wind patterns and a well located site are important during design stages.

b) Subsurface application. Soil incorporation offers better odour and pest control than surface spreading and reduces the likelihood of inadvertent pathogen contamination. Disadvantages include full incorporation of all nitrogen because ammonia volatilization is eliminated. This reduces any nitrogen-loading safety factor from ammonia loss in surface spreading. Costs are greater than for surface spreading because a storage lagoon or tank and subsurface injection equipment are necessary. A resting period of one to two weeks is required before equipment can be driven over the waste-incorporated land. Three methods have been used to inject septage into the land:

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

ii) Sub-sod injection. This technique uses a device that injects a wide band or several narrow bands of septage into a cavity 15 to 20 cm (6 to 8 in.) below the surface. Some equipment forces the injection swath closed.

iii) Terreator. This is a patented device that drills a 9.5 cm (3.75 in) hole with an oscillating chisel point. A tube places the septage as deep as 50 cm (20 in) below the surface at a rate of 24.8 L per linear metre (2 gallons per linear foot).

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

Site selection is important, not only to prevent odour problems but also to minimize potential groundwater and surface water pollution problems. Well sampling and groundwater monitoring may be required as operational checks.

i) Trenches. Disposing septage in trenches is similar to disposing it in lagoons, except that trenches are usually a smaller scale alternative. Septage is placed sequentially in one of many trenches in small lifts of 15 to 20 cm (6-8 inches) to minimize drying time. When a trench is filled with septage, 61 cm (2 ft) of soil is placed as a final covering, and a new trench is opened. Some agencies recommend trenches be a maximum of 2.1 m (7 ft) deep. Sufficient room is normally left between trenches for movement of heavy equipment. The trench-and-fill technique is often used at sanitary landfills.

ii) Sanitary landfill. When a sanitary landfill accepts septage, leachate production and treatment must be investigated. Septage should be prevented from entering landfills in areas with more than 890 mm/annum (35 in/yr) rainfall if leaching prevention and control facilities or an isolated hydrogeological rock stratum are not present. A 15-cm (6 in) earth cover should be applied daily to each area that is dosed with septage. A 61-cm final cover should be applied within one week after the placement of the final lift. Many designers suggest a maximum cell height of 2.44 m. Specific operational guidelines for landfills receiving septage are available from local regulatory agencies.

iii) Disposal lagoons. Disposal lagoons are usually a maximum of 1.8 m deep and allow no effluent or underdrain system. These lagoons require low application rates (15 to 30 cm) and sequential loading of lagoons for optimum drying. Series or series-parallel lagoons with two years capacity each may be sufficient to effect drying. After drying, solids may be bucketed out for disposal in a sanitary landfill, or 61 cm of soil may be placed over the solids as a final cover. Odours are a problem with this type of facility and may be controlled partially by placing the lagoon inlet pipe below the liquid level.

3.7.3 Septage Treatment at Separate Facilities. Alternatives for treating septage at a separate treatment facility include aerated lagoons, composting, the BIF Purifax process, chemical treatment, and the anaerobic/aerobic process (27).

Aerated lagoon. Aerated lagoons can be used to treat septage if the aerators have sufficient oxygen transfer capacity and create enough turbulence to prevent solids deposition.

Brookhaven, Long Island, N.Y., using lagoon treatment of septage, experienced reductions of 62.5 percent in BOD, 51 percent in total solids (TS), and 49 percent in suspended solids (SS) from influent strengths averaging 5600 mg/L, 3700 mg/L, and 2700 mg/L, respectively (27). Without equalization facilities, this process was prone to biological upsets. Grit and scum chambers and three large settling lagoons now buffer flow to the 230 m^3/d septage system. The effluent from a final settling lagoon is chlorinated and discharged to sand recharge beds. Accumulated sludge is removed to a nearby landfill.

Composting. Composting offers good bactericidal action and a 25 percent reduction in organic carbon. In aerobic composting, septage is mixed with dry organic matter for moisture control and easier air penetration so that aerobic conditions can be maintained. Aerobic composting is generally recognized as superior to anaerobic composting because it provides better odour control, higher temperatures for pathogen control, and requires shorter periods for stabilization.

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

Purifax. The BIF Purifax process oxidizes screened, degritted, and equalized septage with dosages of chlorine, from 700 mg/L to 3000 mg/L, under moderate pressure (27). Chlorine replaces oxygen in organic molecules, rendering this material unavailable to bacteria as a food source, thereby stabilizing and deodorizing the septage. The Purifaxes septage changes colour from black or deep brown to straw. The process initially releases CO_2, thus separating liquids and solids by causing the solids to float.

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

Locations using the Purifax process for treating septage and sludge and lagoons for liquid solids separation have had periodic solids separation and odour problems. Sand drying beds appear to be the most efficient method of liquid-solids separation of Purifaxed septage. Adequate ventilation of covered sand drying beds is mandatory to prevent operators from inhaling any nitrogen trichloride (NCl_3) released.

Anaerobic/Aerobic Process. The anaerobic/aerobic process uses an anaerobic lagoon or digester prior to an aerated lagoon.

Chemical treatment. Raw septage is chemically treated with lime and ferric chloride at an Islip, Long Island, N.Y., facility (27). After the septage is screened, degritted, and equalized, about 95 kg of lime per tonne dry solids and 210 L per tonne dry solids of a standard strength ferric chloride solution are flash-mixed with the septage. The solids-liquid separation step occurs in a clariflocculator. The liquid fraction is chlorinated and discharged to groundwater recharge beds, and the underflow solids from the clariflocculator are vacuum filtered. Long-term relative stability of the lime, ferric chloride, septage mixture is unknown.

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

Septage frequently is considered a high strength wastewater and is dumped into an upstream sewer or placed directly into various unit processes in a treatment plant. At some facilities, septage is considered a sludge because it is the product of an anaerobic settling/digestion tank, and it has approximately the same TS concentration as raw municipal sludge. The septage application points, if treated as a sludge, may include sludge stabilization, sand bed drying, or a mechanical dewatering process. The decision where to apply the septage should be determined after a statistically significant sampling and analysis of the septage, including:

- solids loading,
- oxygen demand,
- toxic substances,
- foaming potential,
- nutrient loading (N and P), where required.

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

When septage is added to an upstream sewer or discharged at a treatment plant, there should be a suitable hauler truck discharge facility. It should include a hard-surfaced ramp that leads to an inlet port and is able to accept a quick disconnect coupling directly attached to the truck's outlet. This significantly reduces odour problems. Washdown water should also be provided for the hauler so that spills can be cleaned up. Recording the time and volume and the name of the hauler is vital for operation and billing purposes.

Pretreatment. Treatment plants handling septage have experienced better operation when septage is pretreated. Pretreatment generally includes screening, using bar screens with 1.9 to 2.54 cm openings; grit removal; and pre-aeration or prechlorination if it is an aerobic process. Usually, separation of inorganic matter larger than 150 mesh is sufficient. Equalization/storage tanks with two days' average septage flow and mixing capability should also be provided. To control odours, the storage tanks should be enclosed. Pumping equipment should be used to apply a continuous dose of septage to the desired process. Operators report slug or intermittent doses of septage are difficult to treat and may seriously upset biological treatment systems.

Primary treatment. A report by the U.S. EPA (29) indicates that neither natural settling nor adding lime or polyelectrolytes resulted in consistent liquid-solids septage separation. Another study (30) characterized raw septage as relatively non-settleable, as determined by a settleable-solids volume test (from 0 to 90 percent with 24.7 percent as the average volume). In general, unless chemicals are added to it, septage settles very poorly.

Activated sludge. The following must be considered when contemplating septage addition to an activated sludge process:

a) aeration capacity in the plant;
b) current hydraulic and organic loading on the plant;
c) capacity available for pumping and process waste sludge;
d) characteristics of the septage which could interfere with or inhibit operations at the plant, including metals content and foaming potential.

Septage may be discharged into an activated sludge process either in the form of a slug or a metered discharge. Slug dumping is possible if the increase in mixed liquor suspended solids (MLSS) within the process is limited to 10 percent or less (27). Higher loadings, and the resulting higher sludge wasting rates from the system, affect the biomass within the process, creating a sludge with poor settling properties. It is possible that severe temporary changes beyond the 10 to 15 percent MLSS increase from a septage dump could cause a total loss of the system's biomass. Figure 21 shows suggested loading rates for septage dumps into activated sludge systems of various sizes.

A study conducted for the U.S. Forest Service (31) determined that package treatment plants with a design capacity of less than 455 m^3/d (100,000 Imp. gal/day) should not accept septage for slug dumping. Further, it was determined that package treatment plants could treat septage at approximately 0.1 percent of the hydraulic design of the plant, whereas activated sludge plants, with or without primary settling, could treat septage at at least twice the rate of a package plant (31). This comparison is illustrated in Figure 21.

Plants with septage holding and metering facilities can treat considerably greater quantities of the waste than those practicing slug dumping. Figure 22 is based on the literature and represents continuous septage addition to facilities with fully acclimated biomass. Obviously, initial septage feed to an unacclimated system must be much less than shown, possibly about 10 percent of the indicated values. Gradual increases in daily septage loading over a two to three week period should bring the loading up to or near the amounts shown. Monitoring of the process should be continuous, particularly for oxygen capacity and sludge age.

Figure 23 shows the additional oxygen requirements when septage is added to activated sludge treatment plants. Treatment facilities should be analyzed to determine if oxygen requirements or mixing requirements are controlling factors. Because septage has higher oxygen demands than raw sewage the additional oxygen supply requirement for activated sludge plants accepting septage and having primary treatment facilities would be in the order of 5 kg of O_2 per cubic metre of septage added. For plants without primary treatment, an additional 10 kg of O_2 per cubic metre of septage added should be provided. Package treatment plants have an oxygen requirement similar to plants without primary treatment.

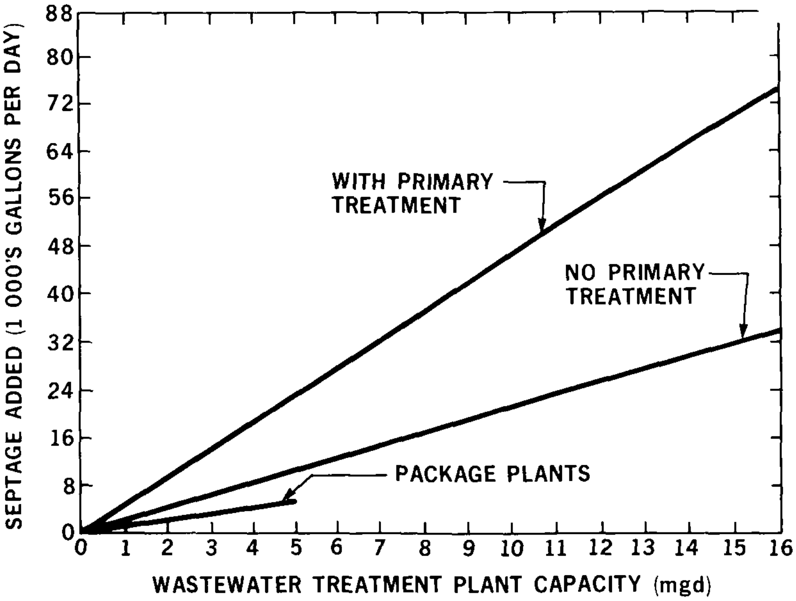


FIGURE 21 SEPTAGE ADDITION BY SLUG DUMPING TO ACTIVATED SLUDGE WASTEWATER TREATMENT PLANTS (31, 32)

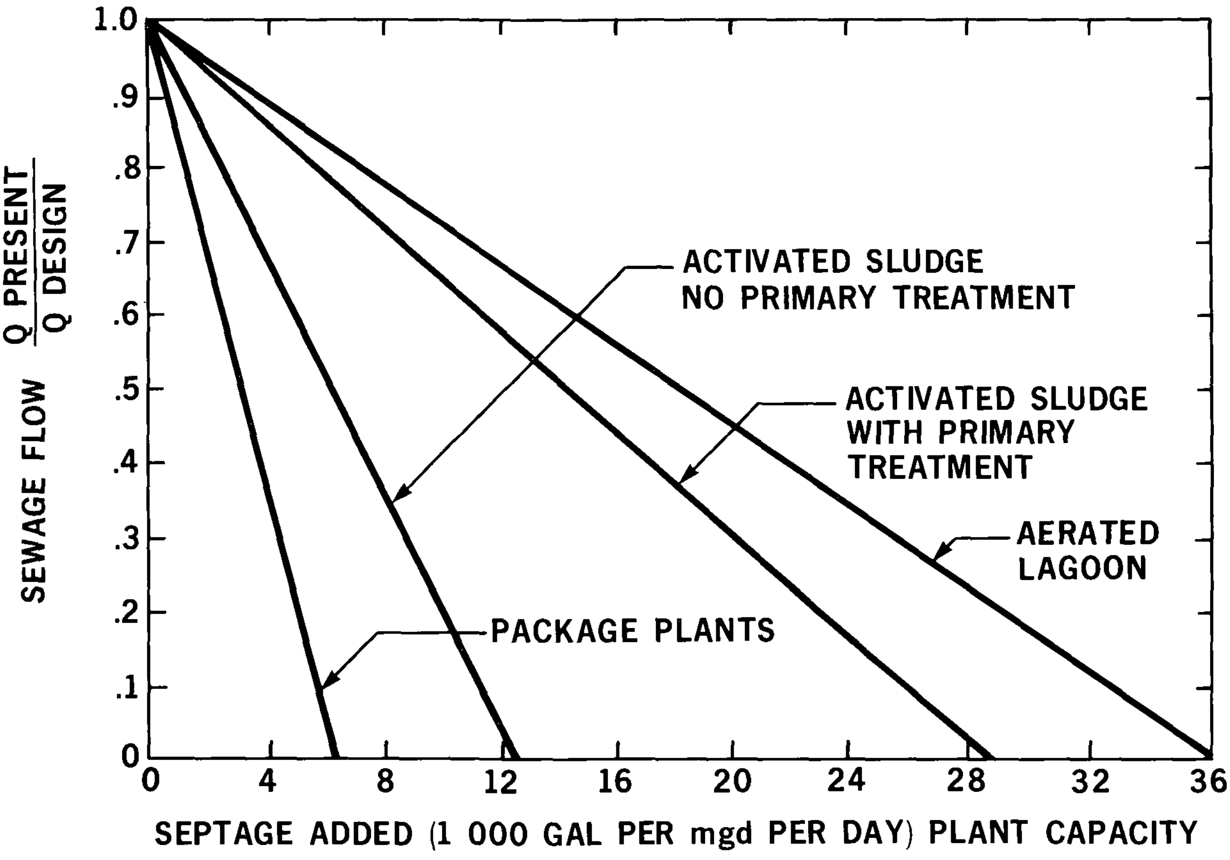


FIGURE 22 METERED SEPTAGE ADDITION TO WASTEWATER TREATMENT PLANTS (27)

Conversion factors: 1000 U.S. gallons = 3.785 m^3
1 mgd = 3785 m^3

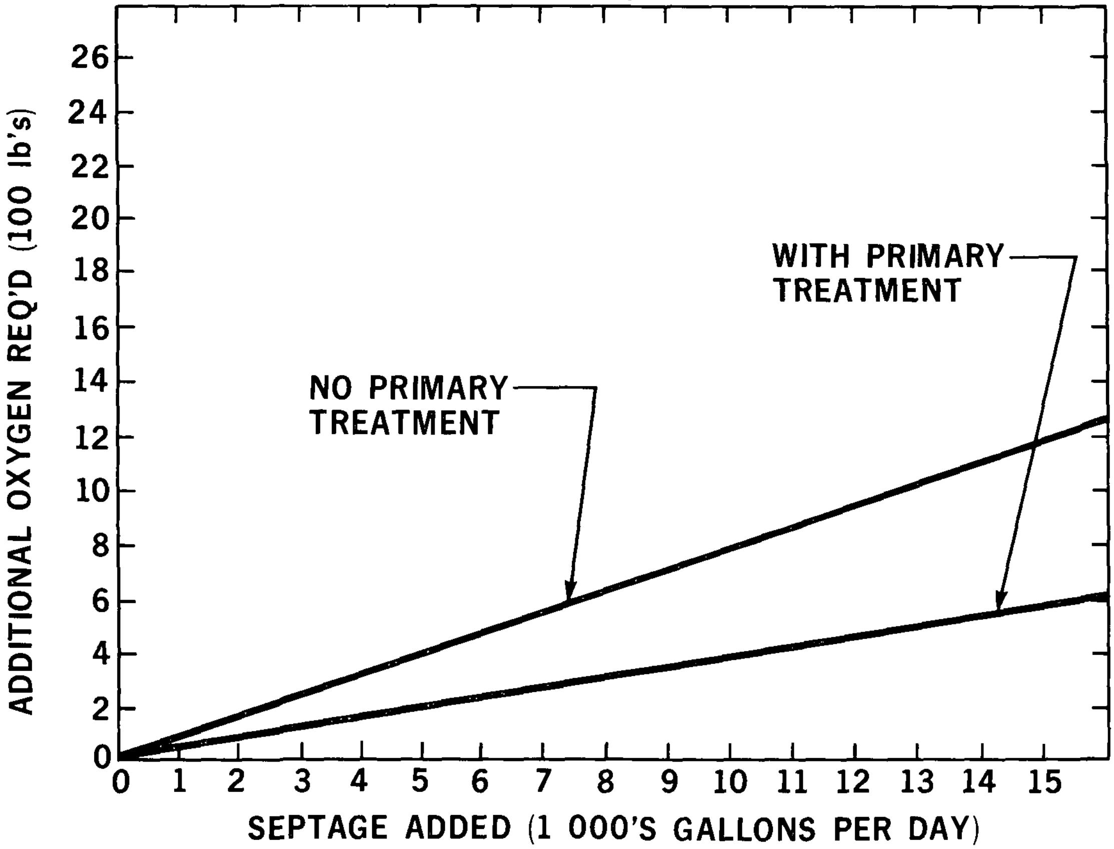


FIGURE 23 ADDITIONAL OXYGEN REQUIRED FOR SEPTAGE ADDITIONS IN ACTIVATED SLUDGE TREATMENT PLANTS (31, 32)

Conversion factors:	100 lbs	=	45.359 kg
	1000 U.S. gallons	=	3.785 m^3

Attached growth systems. Systems that use attached growth aerobic treatment processes, such as trickling filters and rotating biological contactors, are usually resistant to upsets from changes in organic or hydraulic loadings and are quite suitable for septage treatment.

In trickling filters, additional recirculation has been shown to adequately dilute septage concentrations and diminish plugging of the media. At Huntington, Long Island, N.Y., 114 m^3 of septage are treated daily at a 7200 m^3/d facility. BOD_5 reductions of 85 to 90 percent have been observed concurrent with SS reductions of 85 percent (27).

Rotating biological contactors (RBC) use a long detention time and a continually rotating biological medium that is reportedly resistant to upsets. At Ridge, Long Island, N.Y., flow equalization of a low strength septage and a surface loading of 81.5 $L/d/m^2$ resulted in a BOD reduction of 90 percent, a COD reduction of 67 percent, and a total suspended solids reduction of 70 percent by an RBC unit (27).

Aerobic digestion. An alternative to considering septage a concentrated wastewater is to assume it is the product of an unheated digester and, therefore, a sludge. Many researchers have reported good results in aerobic digestion of septage or septage sewage mixtures.

One study (30) reported good septage biodegradation in an aerobic digester with a 10-day aeration time, with a BOD reduction of 80 percent and a VSS reduction of 41 percent. Another researcher (33), treating anaerobically digested septage with an aerobic digester, reported 36 percent VS removal at 40 days' aeration under a VS loading of 25.6 $g/d/m^3$ of digester capacity. After 22 to 63 days aeration, a 43 percent VSS reduction and a 75 percent COD reduction has been observed (34).

When considering septage addition to aerobic digesters, recommendations should include screening, degritting, flow equalization, and analyses of excess digestion capacity and peripheral effects on other processes such as solids handling. An initial septage addition should be limited to approximately five percent of the existing sludge flow. Further septage additions should be gradual (27).

Anaerobic digestion. One report concluded that, in an anaerobic digester with a detention time of 30 days and VS loading of 1280 $g/d/m^3$ digester capacity, a maximum septage addition of 2.1 m^3 for each 14.5 m^3 of sewage sludge added per day per 1000 m^3 digester capacity would not affect the process (27).

Septage should be screened, degritted and equalized before addition to sludge-stage anaerobic digesters. Monitoring of digester performance includes long-term evaluation of volatile acid/alkalinity ratios and gas production. Mixing is vital to prevent a sour digester from developing point source failure from a septage load containing high volatile acid concentrations (27).

In systems with multiple tanks, all the preceeding suggestions apply. Spreading the septage among a number of digesters reduces septage concentrations. Recycling material from the bottom of a secondary digester, or from another well-buffered primary digester, at a rate of up to 50 percent of the raw feed per day has been found helpful. Temperature and mixing should also be adjusted for maximum performance (27).

Sand drying beds. Sand drying has been used to dewater septage with varying success. Anaerobically digested septage is reported to require two to three times the drying period of aerobically digested sludge. After treatment in aerated lagoons and batch aerobic digesters, dewatering simulation studies yielded a septage capillary suction time (CST) in the order of 200 seconds versus about 70 seconds for sewage treatment plant sludges. (A lower CST can be correlated to a faster dewatering time.) The CST's of raw septage were found to range from 120 to 825 seconds; the mean was 450 seconds (27). Adding lime to septage before sand bed dewatering has vastly improved dewatering characteristics. Adding 90 kg of lime per tonne dry solids, or 3.6 kg per m^3 as septage, based on 40 000 mg/L TS, raised the pH to 11.5 and dried to 25 percent solids in six days and 38 percent solids in 19 days (29). An application depth of greater than 20 cm was not recommended because of the slow drying process. Filtrate analysis showed that most heavy metals were tied up in the solids, fecal coliforms were killed effectively, fecal streptococci were more resistant than fecal coliforms, and odours were significantly reduced. Filtrate quality was generally good, but further treatment before discharge was recommended (29).

Other chemicals have worked well in modifying the ability of septage to dewater (35). From a mean initial CST of 450 seconds, septage showed a dewatering ability of 50 seconds after adding an average of either 1360 mg/L ferric chloride, 1260 mg/L alum, 1360 mg/L Puriflox C-31, or 2480 mg/L Puriflox C-41.

The effects of freezing on dewatered samples of septage after treatment in aerated lagoons or batch aerobic digesters have also been studied. Freezing lowered the CST from 225 seconds to 42 seconds, an 80 percent decrease in dewatering time (35).

If septage is to be placed on sand drying beds, treatment to a consistent CST range of 50 to 70 seconds is recommended (27). Further treatment of underdrainage will be required in most cases.

3.7.5 Costs. Of all the alternatives investigated, land disposal is reported to have the lowest operation and maintenance costs, from 40¢ to \$1.32 per m^3 exclusive of the cost of the land. Lagoon treatment is reported to cost between \$1.32 to \$2.64 per m^3. The cost of septage treatment in sewage treatment plants varies widely, but typically runs about \$4.00 per m^3 (27).

4 CENTRAL WASTEWATER TREATMENT SYSTEMS FOR SMALL COMMUNITIES

This section provides design information for treatment systems for small communities (less than 2500 population) and identifies general problems associated with such systems. Because the performance of a treatment plant is closely related to the quality of operation and maintenance, process control and plant upkeep are also included. Cost comparisons for alternate treatment processes are presented to assist in the selection of the most economical system. The data presented are applicable to the treatment of typical domestic sewage with 200 mg/L BOD and 200 mg/L SS.

Design criteria for the various processes are given as ranges of values. The specific value or range of values to be used will depend on the quality and quantity of the untreated wastewater and the effluent quality objectives.

Facilities at wastewater treatment plants in small communities may be classified on a functional basis into the following categories:

a) Flow Equalization. A flow equalization tank modifies fluctuations in pollutant concentrations and flow so that wastewater can be fed into the treatment system at a relatively constant rate. Because small plants are often subject to wide fluctuations in flow and sewage strength, the equalization tank is highly recommended to promote consistent and efficient performance.

b) Solids Removal. Screening, grit removal, comminution and sedimentation remove coarse material, sand and other settleable and suspended matter from influent wastewater.

c) Biological Treatment. Biological treatment removes dissolved and colloidal organics from the wastewater. This process is usually accomplished in a suspended growth system, such as the activated sludge process, or in an attached growth system, such as the trickling filter process.

d) Physical-Chemical Treatment. Physical-chemical treatment employs flocculation, coagulation, sedimentation, filtration and/or carbon adsorption to remove solids, organics and/or phosphorus from wastewater.

e) Sludge Treatment and Disposal. Sludge treatment and disposal involves dewatering and digestion of sludge produced in the above-mentioned processes and, upon stablization of the sludge, disposal of such material in an acceptable manner.

f) Disinfection. Disinfection involves the application of chlorine to a treated wastewater effluent to kill pathogenic organisms prior to discharge to a receiving environment.

Each of these functions are discussed in detail in the following subsections.

4.1 General Design Considerations

Before entering into detailed discussion on the technical aspects of various treatment processes, some of the more basic considerations which should be given recognition as being pertinent to the design and function of a small wastewater treatment plant will be discussed.

4.1.1 Flow Measurement. Every plant should provide flow measurement of the incoming wastes and a record of the flow rate. Many small plant flow meters are inaccurate because they are infrequently checked or because they provide little means for operator calibration. An open channel flow measurement device, such as a Parshall flume, is a most suitable flow measuring device because the operator can zero the meter and manually check the depth, calculate the flow, and compare it to the metered reading. The operator can also check the hourly flow and with a few calculations determine if the totalizer is working properly (36). The design of a Parshall flume and other open channel flow measurement devices is discussed in detail in Section 2.

4.1.2 Sampling. Almost all small plants use manual sampling for operational control and to determine performance results. Eight-hour composites are usually obtained. Because manual collection of samples is time-consuming, automatic compositing samplers are justifiable for at least the plant influent and effluent samples. There are many compositing samplers on the market today. The cost range for these devices is $2000 - $5000 per sample point which, when interconnected with the flow meter, produce an excellent composite sample. The use of composite samplers not only relieves the operator for other duties, but results in more accurate data than manual sampling (36). Specifics regarding this equipment are included in Section 2. Sampling requirements (normally grab samples) for process control have been included in the following text on alternate treatment processes.

4.1.3 Access to Mechanical Equipment. There are many examples of poor layout of mechanical equipment in small community plants. It would appear that in the past little thought was given to removal of pumps, valves, or other equipment, let alone access to such devices by maintenance personnel in the plant.

During design, the designer should keep in mind minimum aisle clearances, adequate spacing between equipment, and other passage access space required for personnel. It is also important to work out procedures to remove equipment from structures or basins in the event replacement is required. The life of the structure probably exceeds the equipment life by four times and future plant expansions may require larger equipment while retaining the use of the structure (36).

4.1.4 **Buildings.** More attention should be directed by designers to building layout and design. The following more salient features should be included:

Laboratory. Many past laboratory designs for wastewater utilities were perfunctory. The lab design should be based on the work areas required for the various analyses; the number of tests, bottles and equipment required; and, the most convenient placement of equipment so that the operator need not go from one end of the lab to the other to perform one analysis. Good lighting and ventilation also are necessary. Even in small labs, safety equipment such as fire extinguishers, eyewashes, emergency showers, etc. should be provided (36).

Maintenance shop. A place for equipment repair should be provided commensurate with the organizational setup of the utility. If maintenance and repair of small parts is to be performed at the plant, a workshop area should be provided (36).

Office/Lunchroom/Records. A room, even though it may be small, should be provided for storage of records and making out reports. This space would also provide a place for the operator(s) to have lunch and coffee breaks away from the lab. A bacteriological/chemical laboratory is no place for lunch (36).

4.1.5 **Plant Site and Landscape.** The planning of the plant site and landscaping also gives the designer an opportunity to minimize maintenance and operation labour and facilitate future expansion of the plant. The plant site should be as compact as possible, but with space for access by cranes or other lifting equipment between structures, as well as access to buried piping. Connecting piping, sidewalks and driveways will be less expensive and more convenient on a compact site layout.

Roadways into the plant and to unloading facilities (such as chlorine cylinders) and loading facilities (such as grit and screenings containers) should be based on the appropriate truck turning radius. This may seem to be obvious; however, the number of small community plants with inadequate vehicle access provisions is legion.

The plant site is often poorly planned for the associated yard work. It is typical for small community plants to have completely fenced properties planted with grass in the enclosed area. The size of the yard and the maintenance required either results in a hit and miss maintenance program or a considerable amount of maintenance labour. As a rule-of-thumb, it takes about 74 MH/year/hectare to maintain a lawn (36).

4.2 Headworks Components

The components of a treatment plant upstream of, and providing pretreatment for, primary clarifiers, flocculators, equalization tanks, or biological units, are considered part of the headworks. Typical headworks components are wet wells and units for screening and comminuting, grit removal, grease and oil removal, and pumping (37).

These components provide preliminary treatment for wastewater to optimize the operation and performance of subsequent treatment processes. Headworks components discussed in this chapter relate to treatment of wastewater that is substantially domestic in origin. Industrial wastewater, it can be assumed, has been pretreated to such an extent that it can be treated as domestic wastewater without loss of plant efficiency (37).

Table 10 lists the units or processes commonly found in headworks and their functions. Under special circumstances, some functions may be combined in one unit (37).

4.2.1 Screening Devices.

Coarse Screen (bar screen). One process common to most treatment plant headworks is screening out larger solids (rags, pieces of wood, dead animals, etc.) that would be unsightly or cause difficulty in downstream processes. For small plants, the screening is usually accomplished by a hand-cleaned bar screen or two bar screens in parallel channels. Sometimes the bar screen is followed by comminution. If the comminutor is down for repair, or if peak flows exceed the comminutor's capacity, the bar screen may constitute the entire pretreatment (37,38). A typical bar screen is shown in Figure 24.

Design criteria for bar screens are as follows (37,38):

Size of openings:	2.5 - 4.5 cm (1 to 1.75 in)
Approaching horizontal velocity:	60 cm/s (2 fps)
Declination of bars:	30 - 60 degrees to the horizontal
Drop of the sewer bottom below the screen:	8 - 15 cm (3-6 in)

TABLE 10 HEADWORKS UNITS

Units or Processes	Functions
Racks and bar screens	Strain out coarse wastewater solids
Communitors and grinders	Macerate and grind wastewater solids into smaller particles
Grit removal	Intercept and remove sand and grit
Skimming (aerated or unaerated)	Remove lighter-than-water particles (such as grease, oil, soap, wood and garbage)
Preaeration	Add oxygen to wastewater, initiate natural floculation, and control odours
Fine screens	Strain out smaller suspended organic matter
Pumping	Add sufficient head to wastewater for gravity flow through plant
Measuring devices	Determine influent flows
Sampling wells	Provide location to sample plant influent
Mixing tanks	Mix influent wastewater, recycled solids, effluents, or sidestreams and chemicals to achieve an homogenous wastewater.

Amount of screenings: approximately 0.008 m^3 per 1000 m^3 (1 ft^3 per MG) of sewage treated.

Screens must be cleaned often enough to prevent sewage flow back-up. Frequency of cleaning depends on the type of sewage, the size of the screen openings and, most important, experience. Cleaning is usually accomplished manually by raking up the screenings to the platform for draining before being removed for disposal.

Screening can be disposed of in a sanitary landfill or in an incinerator. Another disposal method involves the use of a shredder or grinder to reduce the size so

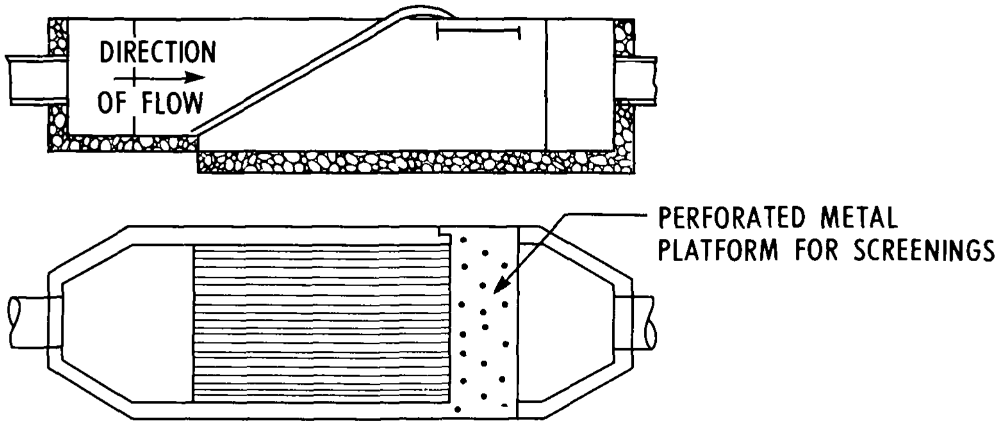


FIGURE 24 BAR SCREEN: CROSS-SECTION AND PLAN VIEW

that they can be returned to the waste streams for subsequent treatment. This method of disposal is normally not employed at small plants.

The annual manhour requirements for cleaning and maintenance of the screening devices for a 450 m^3/d (0.1 mgd) plant is reported to be approximately 250 man-hours (39).

Failure to clean the screens can result in one or more of the following problems (40):

i) back-up of sewage with subsequent deposition of solids in the approach sewer,
ii) shock load to the subsequent treatment units when the screens are finally cleaned.

Screenings usually contain lumps of fecal matter in addition to rags, papers and sticks. Unless promptly removed, septic conditions could occur and attract insects and rodents.

Comminutors (37,38). A comminutor is a device which cuts up any coarse solids not removed from the wastewater stream by a bar screen. Several designs are on the market today, but the basic components of a comminuting unit remain the same, i.e., cutting teeth, shear bars, stationary comb and screen grid. Coarse solids are reduced to particles which are less than 0.6 cm in small comminuting devices, and less than 1 cm in larger units.

Figure 25 illustrates a typical comminuting device. In this case, coarse material is shredded by cutting teeth and shear bars which are mounted inside a slotted revolving drum. The sheared particles pass through the drum slots and are discharged through an inverted siphon into the downstream channel. Manufacturers' data and rating

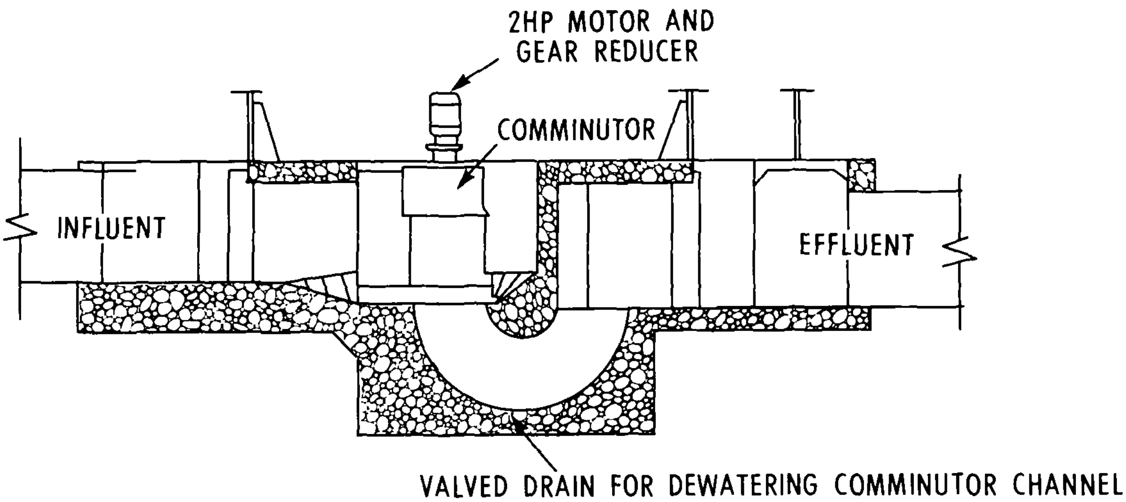


FIGURE 25 COMMINUTOR

tables for these units should be consulted for recommended channel dimensions, capacity ranges, upstream and downstream submergence, and power requirements.

It is advisable to place comminutors or bar screens equipped with shredders after the grit chamber to prevent excessive wear on the blades or teeth. Comminuting devices are also frequently installed ahead of the wet well of a pumping station to protect the pumps against clogging by rags and other large objects, especially in the smaller communities served by separate sanitary sewer systems.

A variety of comminutors designed to suit specific conditions are on the market. They are available in different standard sizes and can be purchased from several manufacturers.

As with all mechanical equipment, comminutors should be lubricated and maintained in accordance with the instructions of the manufacturer. The cutting surfaces require sharpening, and the clearances require periodic adjustment. Stones, sticks and other material should be removed promptly.

The annual manhour requirements for cleaning and maintenance of the comminutor for a 450 m^3/d (0.1 mgd) plant are estimated to be 80 hours (39).

Inorganic matter such as sand, grit, etc. can dull the cutting surfaces, resulting in poor performance of the comminutor (40).

4.2.2 Grit Removal (37,38). Grit removal is included in small wastewater treatment plants to remove inorganic particles of 0.2 mm size or larger. Grit is composed of sand, small gravel, broken glass, cinders, crockery, metal fragments, etc., all of which are heavier than organic particles present in the waste stream. Grit removal units are particularly important if the wastewater contains enough grit to cause faster deterioration and subsequent replacement of equipment such as pumps, centrifuges, and comminutors; to increase the frequency of cleaning of digesters; or to result in excessive deposits in pipelines, channels and tanks.

Grit can be removed from the waste stream through the use of controlled velocity chambers, detritus tanks, or aerated grit chambers. For small wastewater treatment plants, grit removal is normally accomplished with manually-cleaned parallel grit channels (Figure 26), in combination with a downstream control to maintain a uniform velocity of close to 0.3 m/s (1 ft/sec). The velocity must be kept within a range that permits the heavier inorganic grit to settle while lighter organic solids are kept in suspension.

Design criteria for grit removal chambers are as follows:

Horizontal velocity:	0.3 m/s (1 fps)
Detention time:	1-2 minutes
Number of chambers:	2
Storage space for grit at bottom of chambers:	0.02-0.06 m^3 per m^3 (2-8 ft^3 per million gallons) of sewage treated.

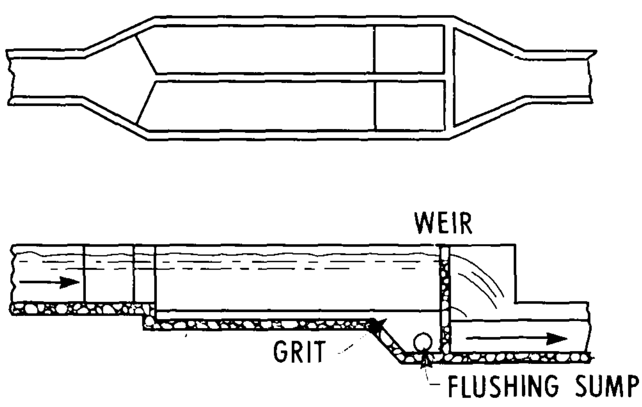


FIGURE 26 GRIT CHAMBER

Control of the velocity in the effective length of a grit chamber is provided by a control section in the grit channel. The control section (or weir) will vary with the cross-sectional area of flow in direct proportion to the flow. The Sutro weir and the proportional weir accomplish this very satisfactorily. The design of each of these weirs is based on the relationship that the theoretical discharge from the weir is directly proportional to the head of the liquid upstream of the weir. The weirs can be adapted for use in rectangular channels where sufficient head is available to permit the sill or crest of the weir to be kept above the downstream water elevation. The sill of the Sutro weir is frequently located 15.0 to 30.0 cm (6-12 inches) above the grit channel invert. This not only provides for grit storage, but also prevents the scouring out of previously settled grit particles. A Sutro weir has the advantages of accurately maintaining an average flow and of enabling relatively simple grit chamber construction. Its disadvantages consist of causing bottom chamber velocity to be greater than top velocity, and loss of velocity control if the weir is submerged. The only difference between the Sutro weir and the proportional weir is that the proportional weir has two curved sides, and the Sutro weir has one curved side and one straight side (Figure 27). The discharge from a proportional weir will be twice that given by the equations in Figure 27.

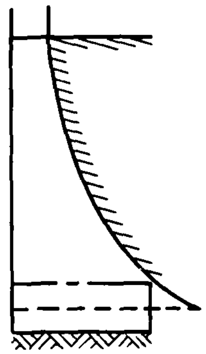
If grit chambers are followed immediately by a Parshall flume, the flume may be designed to control velocity (41,42). Details on the design of the Parshall flume are given in Section 2.

A detritus tank is a grit chamber in which the velocities permit an appreciable amount of organic matter to settle out with the grit. An aerated detritus tank, or aerated grit chamber, is a tank in which the organic matter that would otherwise settle out is maintained in suspension by rising air bubbles or some other form of agitation (37,38).

Aerated grit chambers have the following advantages:

1) Grit removed is clean enough for disposal without further treatment.
2) Variations in flow have little effect on the efficiency of grit removal.
3) The removal of grease, or other floatables, by flotation and skimming can be combined in one chamber with grit removal.
4) The chamber, because of its mixing capabilities, may provide a good location for chemical additions to improve plant solids and phosphorus removal, and for odour control and prechlorination.
5) Preaeration adds DO to incoming wastewater, normally devoid of oxygen, before it is discharged to the next process.

a) SUTRO



$$X = b\left(1 - \frac{2}{\pi}\tan^{-1}\sqrt{y/a}\right)$$

$$Q = b\sqrt{ay}\,\left(h + \tfrac{2}{3}a\right)$$

$$Q_1 = \tfrac{2}{3}\, b\sqrt{2y}\,\left[(h+a)^{3/2} - h^{3/2}\right]$$

WHERE

a&b = CONSTANTS FOR WEIR

y = LIQUID HEIGHT

x = WEIR WIDTH AT LIQUID SURFACE

Q = TOTAL SUTRO WEIR DISCHARGE

Q_1 = DISCHARGE THROUGH RECTANGULAR PORTION OF SUTRO WEIR

b) PROPORTIONAL

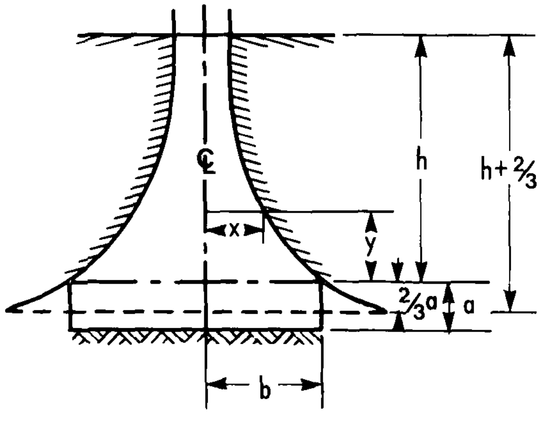


FIGURE 27 ELEMENTS OF SUTRO WEIRS AND PROPORTIONAL WEIRS

The major disadvantage of aerated grit chambers for small treatment plants is that they require more operation and maintenance than do manually cleaned grit channels (37,38).

Grit is generally removed from the chamber by hand or by flushing the grit onto a disposal area bi-weekly. After a heavy rainfall, grit accumulation may be significantly increased, making more frequent cleaning necessary to keep the unit in proper operation.

Grit collected from the chamber usually contains up to 50% organic matter. Unless properly disposed of, this unsightly and odourous material will attract insects and rodents. The method of grit disposal will depend on the amount and the characteristics of the grit, and the availability of disposal sites. For small treatment systems, the most appropriate method is burial. If the organic content is low or the grit is properly washed, it may be used in sludge-drying beds or to make the walkways around the plant.

The annual manhours required for operation and maintenance of a grit chamber for a 450 m^3/d (0.1 mgd) plant are estimated to be 180 hours (39).

Common operational problems encountered with a grit chamber include poor velocity control because of the variation in sewage flow, excessive deposition of grit at the inlet end of the chamber and odours created by improper separation of organics from grit (40).

4.2.3 **Oil, Grease and Floating Solids Removal.** When the amount of fat, oil, grease and floating solids, such as soap, vegetable debris, fruit skins, pieces of cork, etc. is high, removal of this debris may improve the treatability of the wastewater, as well as providing protection to sewers, pumps and downstream treatment components. Pretreatment may include skimming tanks, grease traps, and preaeration of the wastestream at the origin of the flow or at the head of the treatment works.

A skimming tank is a chamber designed so that floating matter rises and remains on the surface of the wastewater until removed, while the liquid flows out continuously through outlets or partitions below the water line. The skimming tank may be a separate unit or combined with primary sedimentation, depending on the process and nature of the wastewater.

Grease traps are small skimming tanks usually located close to the source of the grease. There are a number of proprietary tank patterns; Figure 28 illustrates one particular commercial grease trap design. These units are normally used in small systems.

Preaeration may also be used to remove grease prior to primary sedimentation. Additional benefits of the process include odour control, grit removal, uniform

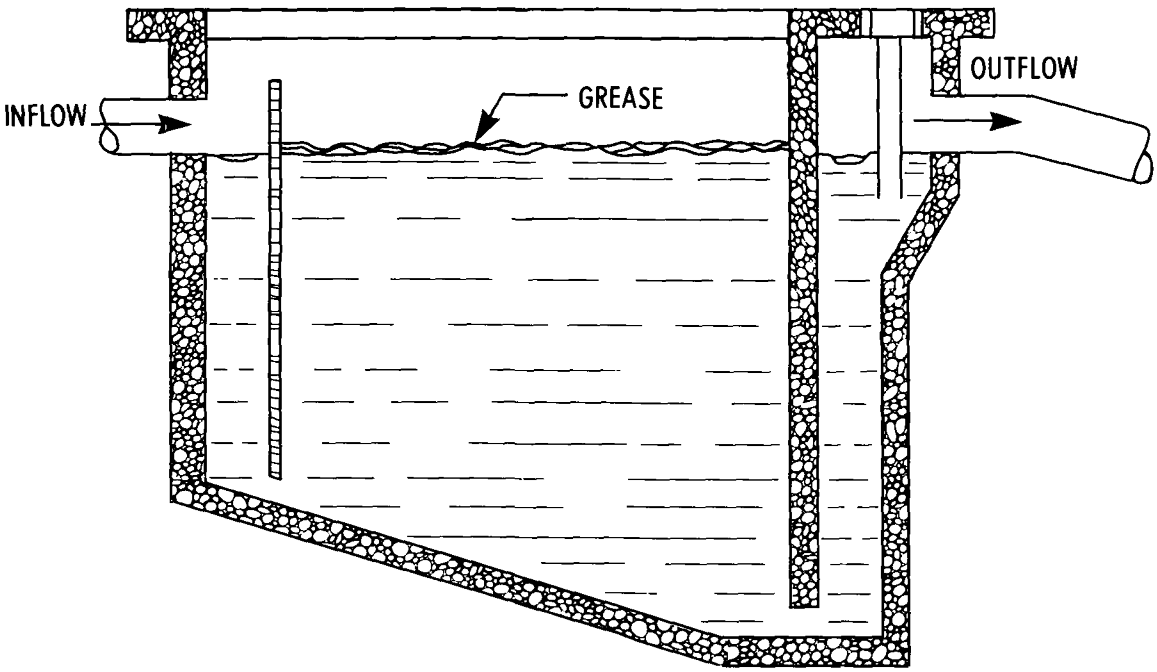


FIGURE 28 GREASE TRAP

distribution of suspended and floating solids, and increased BOD removal. Unfortunately, the cost of maintaining an aerated tank for grit and grease removal in a small system often precludes its use.

The following are design criteria for grease traps:

Detention time:	3-30 minutes
Surface area:	250 $m^2/m^3 \cdot s$ (14 ft^2/mgd)
Slope of influent pipe:	$\geq$ 1 to 30

Grease traps must be large enough to hold and, if necessary, to cool sudden discharges of oily or greasy wastes. Regular cleaning of the trap, at least once a month, is required to maintain efficient operation. These requirements are often not attended to and the result is clogging of the inlet and outlet pipes (40).

The fat and oil, depending on the type, may be recovered and reused, or disposed of in the same manner as screenings mentioned previously.

4.2.4 Flow Equalization. Sewage flows in municipal systems typically exhibit diurnal and seasonal variations, both in quantity and quality. An equalization tank may be used to provide a constant flow of sewage to subsequent treatment processes, and to equalize varying strengths and concentrations of wastewater, providing a homogenous feed of raw waste to the treatment plant. This will effect stable and reliable operation of the plant.

Flow equalization is normally employed after bar screens and grit chambers to avoid accumulation of coarse solids and grit in the tanks, and ahead of primary clarifiers to absorb hydraulic surges which might detrimentally affect the performance of clarifiers. Downstream biological processes benefit from damping of fluctuations in concentrations and flows, and are protected from shock loadings and toxic or treatment-inhibiting substances. Improved process control is also possible where chemical addition is practiced, and sizing of pumps and pipelines is easier.

Disadvantages of flow equalization include increased operation and maintenance costs, potential odour problems and, in cold climates, lower wastewater temperature.

Methods of equalizing wastewater flow include (37,43):

a) A good method for small-flow plants is (after degritting) designing equalization into a treatment process unit, such as an aerated lagoon, an oxidation ditch, or an extended aeration tank, by allowing for variable depth operation and a discharge controlled to near the average 24-h flow rate. The discharge control device may be a proportional controller placed on the discharge pipe of an aeration basin, or a floating effluent weir.

b) Sideline equalization tanks (Figure 29) may be sized to receive and store flows in excess of the average daily flow rate and then to return the stored wastewater at a rate that will raise subaverage plant flow to the average rate. The organic loading variations on the subsequent processes are partially affected, particularly during periods of less than average flow.

c) Inline equalization tanks (Figure 29), sized in the same manner as sideline tanks, equalize the outflow at near the average daily flow rate. This results in significant concentration and mass flow damping.

d) Extra capacity provided in large trunk or interceptor sewers leading to the treatment plant for intermittent storm water inflow may be used for storage of peak flows. This method of equalization is less attractive for small wastewater systems. A variation of this alternative which may be incorporated into a small system is the placement of the equalization facility at a pumping station at the edge

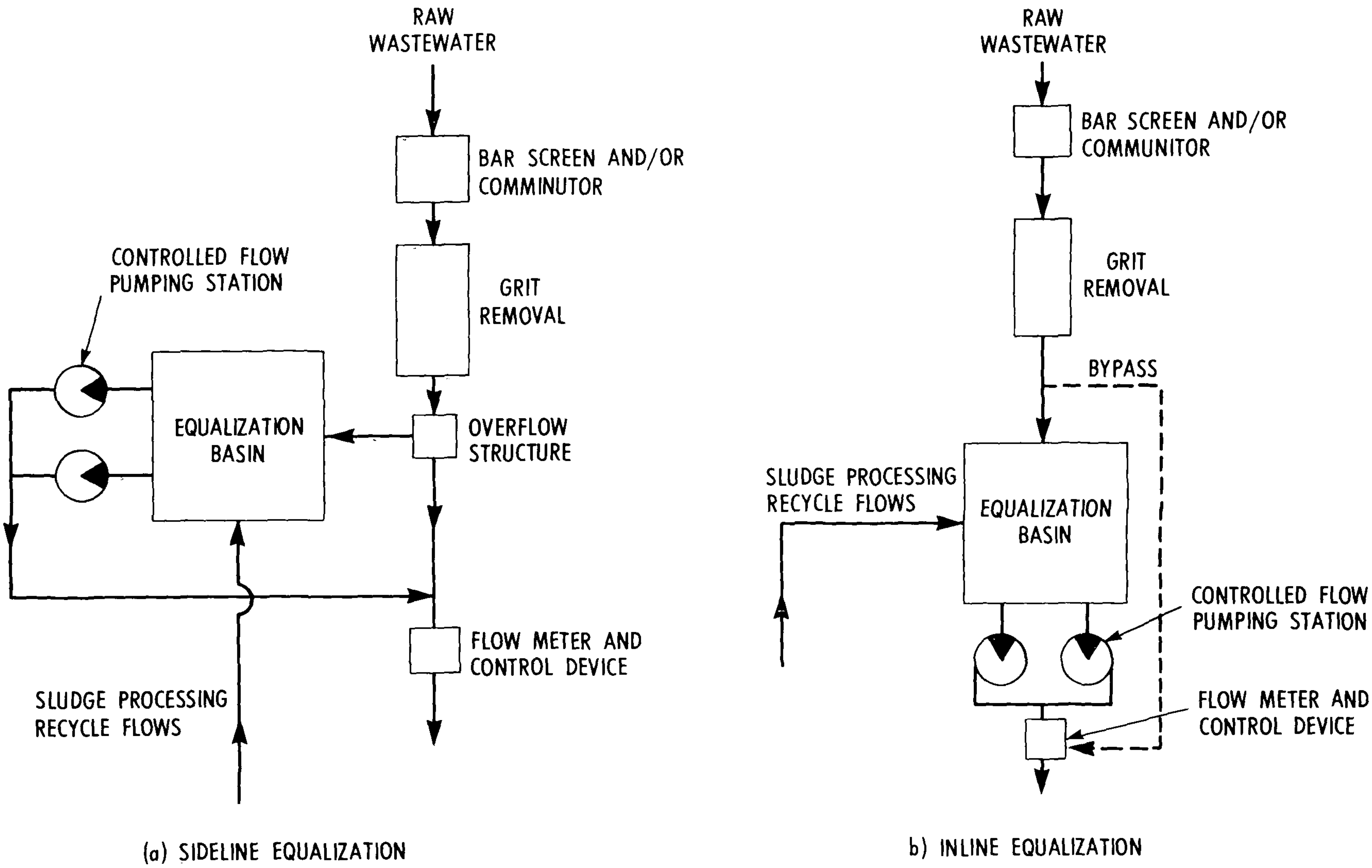


FIGURE 29 SIDELINE AND INLINE EQUALIZATION TECHNIQUES

of the collection works or the head of the treatment works. This will reduce the size of downstream treatment facilities, and possibly the pipeline to the plant.

The maximum 24-hour flow expectancy must first be determined to establish the required equalization storage capacity. Wastewater characterization is also an important factor in the design process.

Figure 30 illustrates typical flow and BOD curves for a maximum daily load at the end of a design period. A mass diagram or hydrograph can be developed from this figure and plotted as in Figure 31. The inflow mass diagram is plotted by first converting the hourly diurnal flows to equivalent hourly volumes and accumulating the volumes over the 24-hour day. A line is then drawn from the origin to the end point on the diagram. The slope of this line actually represents the average flow for the day. Enough equalization volume must be provided to accumulate flows above the equalized flow rate. This normally requires a volume equivalent to 10 to 20 percent of the average daily dry weather flow. To determine this volume, the inflow mass diagram must be enveloped within two lines parallel to the average flow line and tangent to the extremities of the inflow mass diagram. The required volume is represented by the vertical distance between these two lines (37,43).

The actual equalization volume must be greater than that obtained from the hydrograph because:

a) Continous operation of ancilliary aeration and mixing equipment may not permit complete drawdown.
b) Volume may be required to accomodate possible plant recycle streams to the equalization component.
c) Some extra volume should be provided for unforseen changes in diurnal flow.

The final volume selected should take these conditions into consideration, as well as physical and economic restraints in each case.

Factors requiring evaluation in the selection of the type, size and mode of operation of an equalization process are (37,43):

1) degree of flow rate and organic loading equalization required to ensure reliable and efficient process performance;
2) optimum location in the system;
3) type of equalization best suited to items 1) and 2);
4) optimum volume required for equalization;

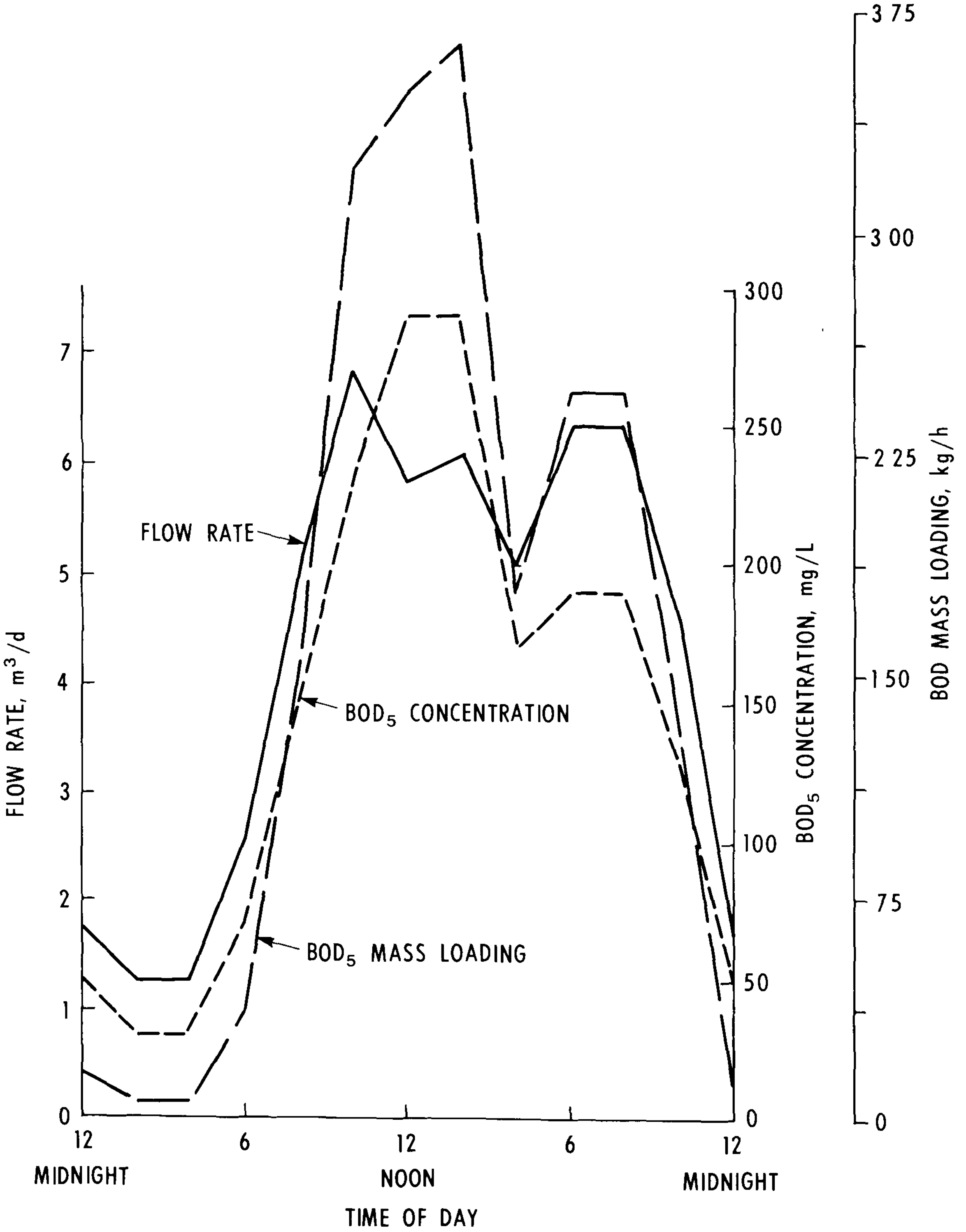


FIGURE 30 TYPICAL DRY WEATHER FLOW AND BOD VARIATION OF MUNICIPAL WASTEWATER BEFORE EQUALIZATION (37)

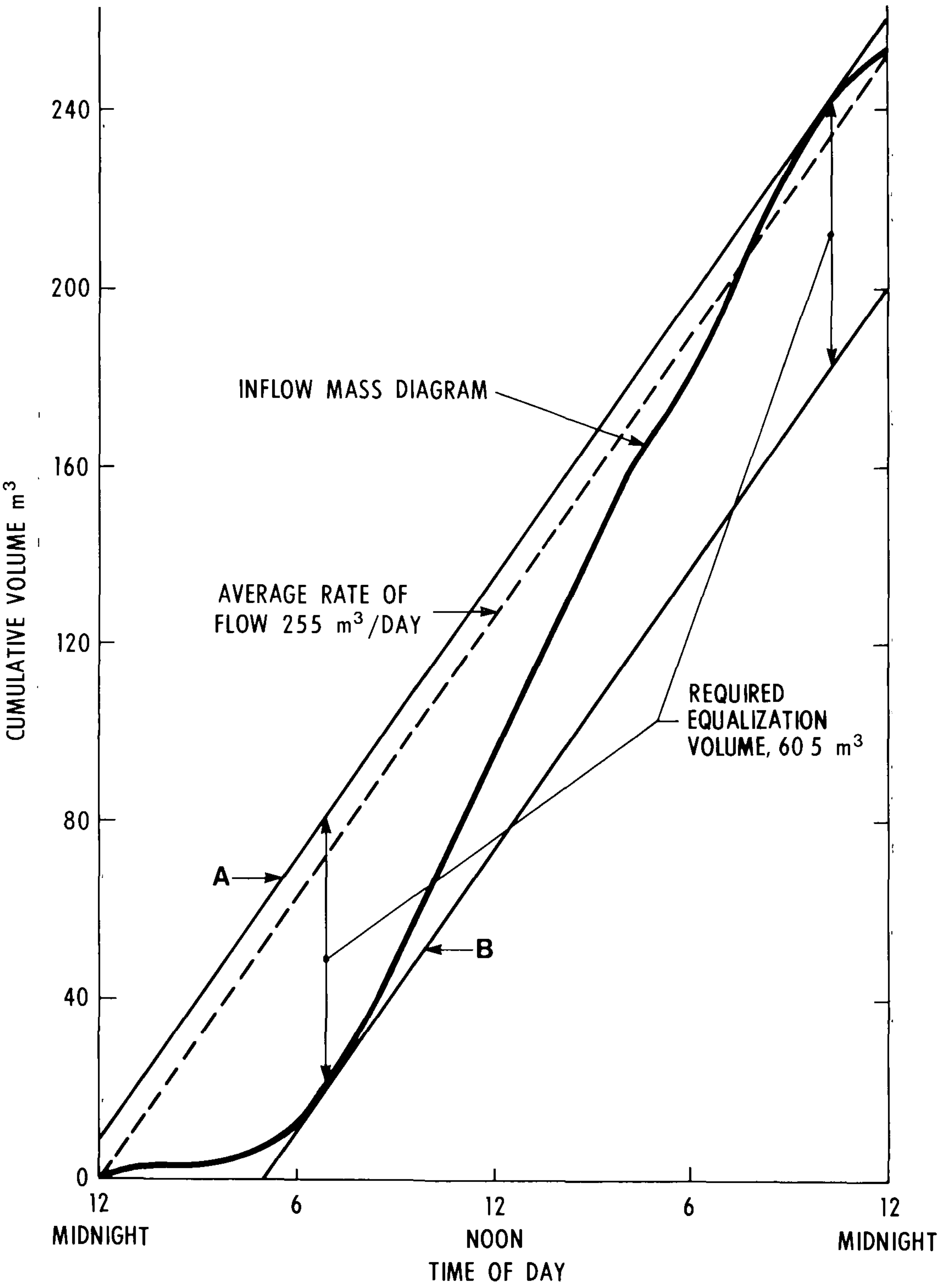


FIGURE 31 HYDROGRAPH OF A TYPICAL DIURNAL FLOW (37)

5) present and future flows;
6) type of construction;
7) aeration and mixing equipment requirements;
8) pumping and discharge flow rate control;
9) minimum operation and maintenance requirements under adverse weather and flow conditions;
10) feasible alternative treatment components sized for peak flows.

Treatment processes sized for peak flows at minimum water temperatures, to eliminate the need for equalization for consistent reliability, should be compared with sizes reduced to meet equalized needs of processes operated at equalized flows and characteristics, to ascertain the cost effectiveness for each. At some smaller treatment plants, some degree of equalization may be essential if consistently acceptable plant effluent is to be obtained, particularly if the quality of the effluent must meet very strict standards.

Mixing or aeration equipment must also be installed where it is necessary to prevent the wastewater from becoming septic or where the contents of the equalization facility must be blended. This equipment must be selected with careful consideration of the varying wastewater flows to the equalization facility and the power fluctuations caused by these flows. A municipal wastewater with an SS concentration of approximately 200 mg/L will require mixing at from 0.004 to 0.008 kW/m^3 of storage. To maintain aerobic conditions, air should be supplied at a rate of 0.009 to 0.0015 m^3/m^3 per minute of storage. Mechanical aerators are one method of providing both mixing and aeration (37,43).

Flow equalization imposes an additional head requirement at the treatment plant. As a minimum this head is equal to the sum of the dynamic losses and the normal surface level variation. Pumping facilities are usually required to accomodate the additional head requirement, either for the raw influent, the equalized flow, or both. Influent pumping will require larger capacity pumps to handle diurnal peaks.

The basin size, type of construction, availability of land, location, and mixing, aeration and pumping requirements all have considerable effect on the cost of flow equalization. This cost must considered against: 1) the savings in cost of modifying subsequent processes to accept diurnal variations, and 2) the improved performance that can be achieved by operating subsequent facilities under relatively constant loading conditions (37,43).

4.3 Primary Treatment

4.3.1 Sedimentation (37,38). Sedimentation or settling is an important process in sewage treatment. It is employed in a primary treatment process to remove settleable solids and scum from raw sewage.

Sedimentation may be accomplished in horizontal or vertical flow tanks. In a horizontal flow tank, the sewage enters at one end of the unit and leaves at the other end. In a vertical flow tank, which is usually circular, sewage enters at the centre and flows to the periphery of the tank. Most tanks are provided with hoppers and mechanical sludge collecting devices. Sludge settles to the tank floor and is removed by mechanical scrapers into hoppers. A rectangular settling tank is shown in Figure 32, and a circular tank in Figure 33.

Basically, a continuous flow settling tank can be divided into four zones: 1) an inlet zone where influent suspended solids disperse over the cross-section at right angles to the flow; 2) a settling zone where the suspended particles settle; 3) a sludge zone adjacent to the bottom, where the removed solids accumulate and from which they are withdrawn for disposal; 4) an outlet zone where the clarified flow is carried to the effluent conduit. These zones are illustrated in Figure 34 for a horizontal-flow tank. Similar zones exist in vertical-flow tanks.

The inlet of a settling tank should be designed to bring the sewage into the tank at low velocity (approximately 1 m/s) and distribute it evenly over the cross-section or periphery of the tank. In rectangular tanks, a series of submerged openings spaced across the inlet end of the tank can be used. A baffle is provided to reduce the sewage velocity and help spread the flow, as shown in Figure 32. For circular tanks, satisfactory results are generally obtained by bringing the incoming flow into a influent well at the centre through a pipe in the body of the tank (Figure 33), or by an upward flow through a central riser from a pipe entering under the tank. A circular baffle provides a satisfactory distribution of the flow.

The settling zone of the sedimentation tank is normally designed based on the surface loading (also called overflow rate), given by the flow rate divided by the surface area. Typical design parameters for primary sedimentation tanks are in the following ranges (37):

Surface loading:

Peak overflow rate	80-100 $m^3/m^2 \cdot d$ (2000-2500 gpd/ft^2)

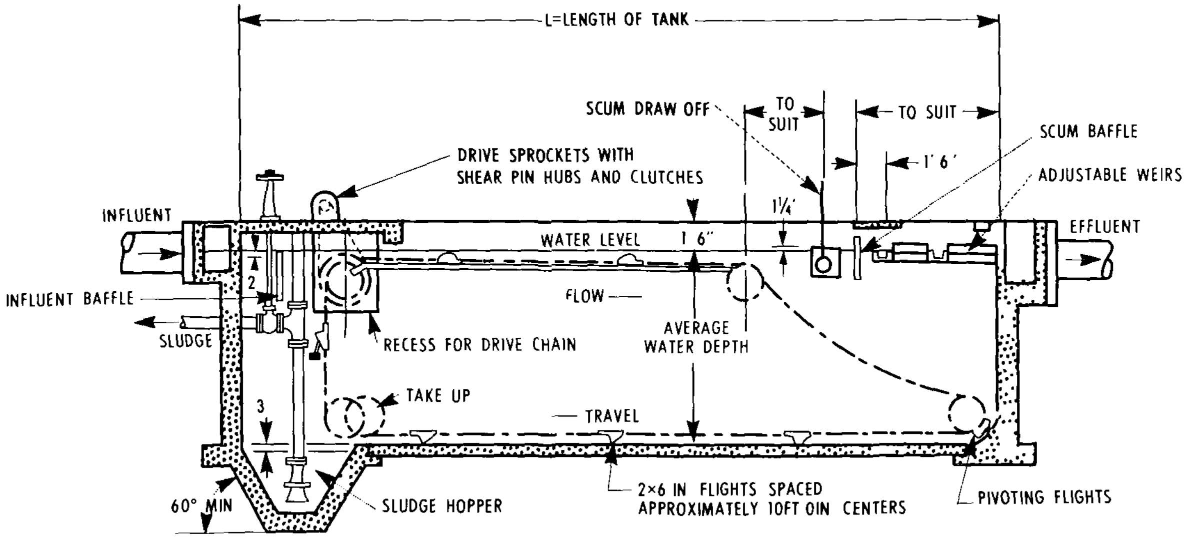


FIGURE 32 TYPICAL RECTANGULAR PRIMARY SEDIMENTATION TANK

Conversion factor: 1" = 2.54 cm

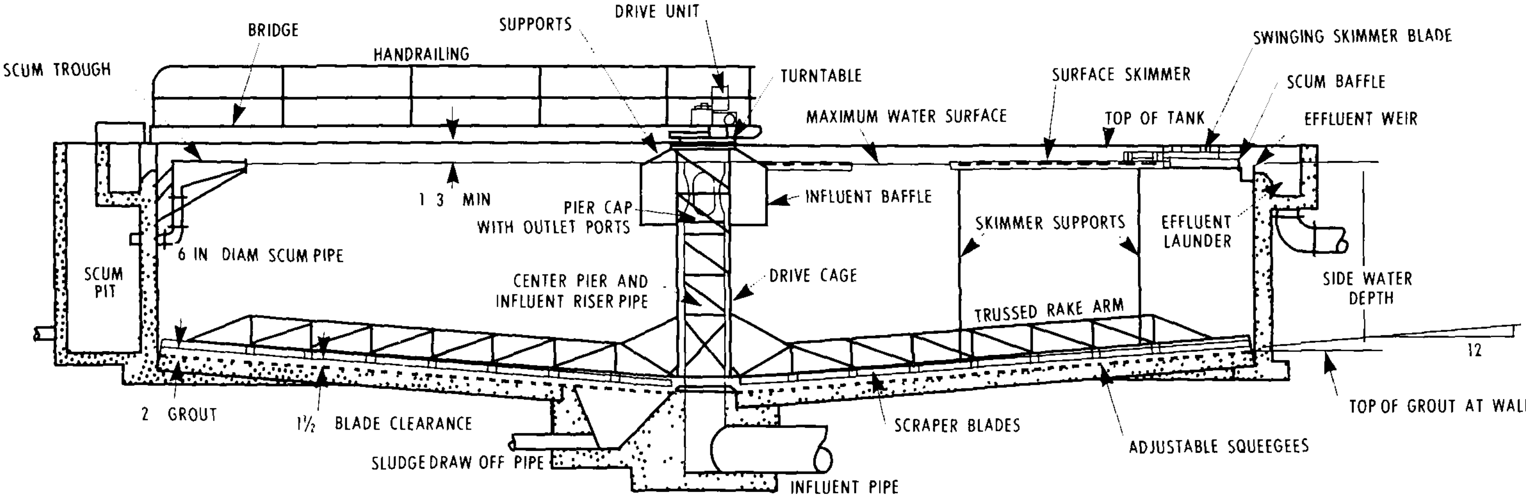


FIGURE 33 TYPICAL CIRCULAR PRIMARY SEDIMENTATION TANK

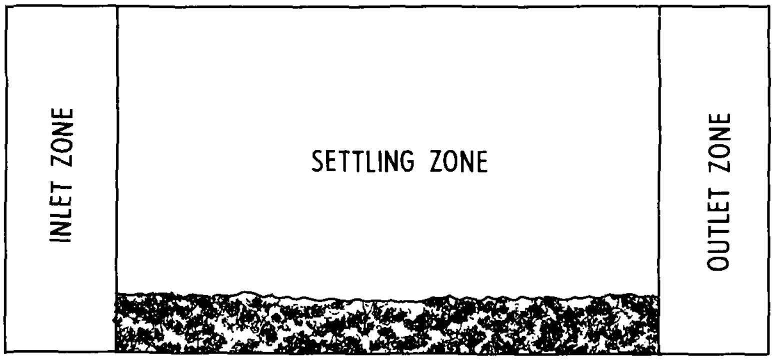


FIGURE 34 VERTICAL CROSS SECTION THROUGH A RECTANGULAR SETTLING TANK

Average overflow rate	25-33 $m^3/m^2 \cdot d$ (600-800 gpd/ft^2)
Detention time:	2-3 hours
Water depth:	3-5 m (7-12 ft)
Depth to length ratio:	1/10 - 1/30 (rectangualr tanks)

The settling tank outlet generally consists of serrated or smooth weirs. Weir loading should be approximately 150 m^3/d per metre of weir length (10,000 gpd/ft).

Mechanical sludge removal equipment is essential to good performance of the settling tank. Scrapers are used to collect the sludge into hoppers, as shown in Figures 32 and 33. The common mechanism in rectangular basins is a chain and flight collector which moves settled solids to a sump at the inlet end. Since sludge accumulation is heaviest near the inlet end of the tank, sludge hoppers should be located in that vicinity. The side slopes of a hopper should be greater than 60 degrees to the horizontal, and the bottom dimensions are usually 60 cm by 60 cm (2' x 2'). Standard equipment for circular basins consists of at least two radial arms with angled scrapers attached, which move settled solids to a central sump.

An accumulation of scum is to be expected on the surface of primary settling tanks. A scum baffle, extending at least 15 cm (6-12 inches) below the surface, is usually used to prevent the scum from discharging with the effluent (Figure 32). A pipe with a flared opening is commonly installed just below the water surface in small rectangular systems to collect scum. Circular tanks are equipped with surface skimmers (Figure 33) to remove scum from the liquid.

The performance of a settling tank depends greatly on the design and operation of the tank. In a well-designed and properly operated primary settling tank treating domestic wastewater, 25 to 40% of BOD, and 50 to 65% of suspended solids removals can be expected. Factors affecting the performance of settling tanks include overflow rate, detention time, tank configuration, wastewater characteristics and operational practices at the plant. Wind action and convection currents of thermal origin will contribute to short-circuiting of the flow and result in poor treatment efficiency.

Specific problems are likely to occur seasonally. In winter, sludge will be more difficult to pump, sludge lines will collect grease more quickly, and scum quantities will increase; however, septic conditions and odour will not normally occur. Summer operating conditions are accompanied by increased septicity and odour problems, and less difficulty with sludge pumping and scum (40).

Cleanliness is essential in the operation of settling tanks. All exposed parts of the tank and channel should be washed frequently, daily if necessary, and scraped or squeegeed to prevent the accumulation of exposed deposits (40).

Sludge is preferably removed from the hoppers by pumping. Sludge withdrawal pumps can be operated continuously or intermittently at a predetermined frequency. Intervals between pumping of settled sludge may range from once every 30 minutes to once every 12 hours, depending on the characteristics of the raw sewage, including: strength and freshness; the period of sedimentation and the degree of purification effected; and the condition of deposited solids, including specific gravity, water content and changes in volume influenced by tank depth or sludge-removal devices.

Approximately 470 manhours/year are required to operate and maintain a 450 m^3/d (0.1 mgd) primary sedimentation tank (39).

Settled sludge has an offensive fecal odour, and is quickly putrescible if not promptly removed. Rising bubbles and lumps of floating sludge are evidence of septic conditions and equipment or process breakdown. Overloads caused by fluctuations in sewage flow result in poor performance of the settling tank (40).

4.3.2 Fine Screens. Static fine screens used as a wastewater treatment process result in BOD and SS removals in the range of 10 to 30 percent. Consequently, installation of these devices could be a viable alternative to the use of primary sedimentation tanks. However, there is presently no long-term operational information on fine screening for domestic wastewater treatment.

Static fine screens have three sections with progressively flatter slopes on each lower section. The screen wires are triangular in cross-section and usually spaced 1.5 mm (0.06 inches) apart when used for raw wastewater screening. Above the screen and running across its width is a headbox. A light-weight hinged baffle at the top portion of the screen is used in some commercial units to reduce flow turbulence.

Inclined screening units are generally constructed entirely of stainless steel. Lighter units with fibreglass housings and frames may also be obtained.

Figure 35 illustrates a commercially manufactured static screen (hydrasieve). Hydrasieves have performed satisfactorily in test studies at loading capacities of 0.1 to 0.4 L/s per centimetre (4 to 16 U.S. gpm per inch) of screen width. The hydraulic capacity of a screen is a function of the fluid viscosity (a function of temperature), the solids loading, and the spacing of the individual slots. Slot width (normally 0.8-1.5 mm; 0.03 to 0.06 in) is selected by testing using sample screens. Once the slot opening has been chosen, the screen's capacity per centimetre of width can be determined.

Operating experience with static screens is quite limited in Canada, although they have been favourably received in some U.S. operations. Pilot studies report SS removal efficiencies in the 10 to 30 percent range and a solids generation rate of 0.2 m^3 per 1000 m^3 of wastewater. The average solids content of recovered matter ranges from 12 to 15 percent. Incidental to SS removal in this process is the aeration of the separated water. Static screens have been found to aerate wastewater to a level of 2 mg/L dissolved oxygen (38,40).

Separate grit removal facilities may be required in some cases and should be installed downstream of the inclined screens.

Maintenance of static screens normally involves daily cleaning and removal of solids. Washing is done with hot water to remove any grease accumulated on the screen which prevents passage of wastewater (38,40).

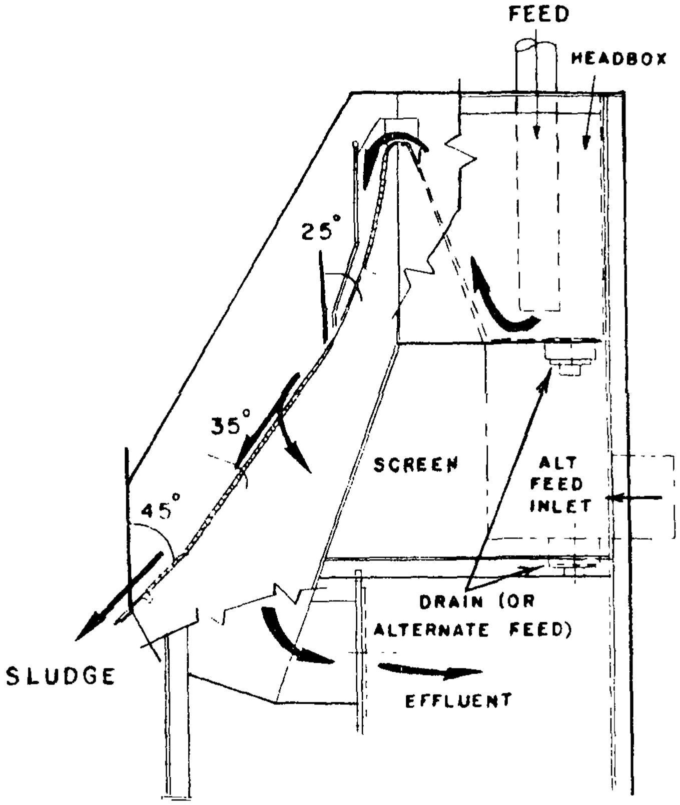


FIGURE 35 HYDRASIEVE SCHEMATIC (From C-E Bauer)

4.4 Biological Treatment

Various biological processes have been used successfully in the treatment of municipal and industrial wastewaters. The effective control of the biological treatment system is based on an understanding of the basic principles governing the metabolism of microorganisms. It is therefore essential to know the fundamentals of biological reactions in a sewage treatment system and the key factors affecting the biological activities in such a system.

4.4.1 Biological Reactions in a Treatment System. Aerobic biological waste treatment involves the utilization of a mixed population of microorganisms to convert soluble organic contaminants to new cellular material (sludge). At the same time, a portion of the organics is oxidized to carbon dioxide and water. Because of the complex nature of the wastewater, which contains a variety of organic and inorganic compounds, different types of organisms, including bacteria, fungi, protozoa, algae and other higher forms of life, can be found in the sewage treatment system.

When a microbial population is brought into contact with a wastewater containing adequate substrate for growth in a batch reactor, the change in substrate concentration and microbial population follows the pattern shown in Figure 36. The

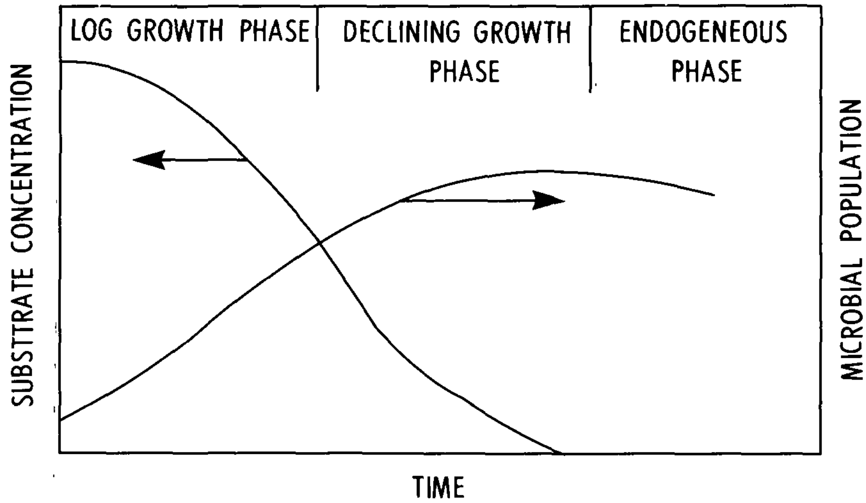


FIGURE 36 SUBSTRATE REMOVAL AND MICROBIAL GROWTH RELATIONSHIPS

breakdown of the substrate by the microorganisms results in a decrease in substrate concentration, accompanied by a corresponding increase in cellular material. The lower portion of the growth curve is called the logarithmic growth phase, during which maximum multiplication of microbial cells occurs because there is an abundant food supply. As growth progresses, the substrate gradually decreases. The growth rate is limited by the availability of substrate and a declining growth phase occurs. After depletion of the food source, a number of cells die, resulting in a decrease in the microbial mass. This is called the endogeneous or auto-oxidation phase. In some cases, a lag phase may exist before the log-growth phase. This is the period in which the microorganisms adjust to a new food source or environment (41,42,44).

In a continuous biological treatment system, the substrate removal and growth relationship can also be identified at the steady-state operating condition. This relationship is a function of the loading condition, which is discussed section 4.4.2.

A microbial population in the declining growth phase is most commonly used in wastewater treatment. The endogeneous phase of metabolism is the basis of treatment of organic waste in the extended aeration process, which has been used successfully for the removal of soluble organics as well as for the oxidation of the cellular material. Attempts to use microorganisms in the logarithmic phase for the treatment of wastewater have been unsuccessful due to the dispersion of the microbial population and the presence of a high organic residual in the effluent (41,42,44).

All microorganisms involved in wastewater treatment can be classified into three groups, according to their ability to utilize oxygen. Organisms which can only exist when there is a supply of molecular oxygen are identified as strict or obligate aerobes. Those which can only exist in an environment that is completely free of molecular oxygen are referred to as obligate anaerobes. Organisms having the ability to survive either with or without the presence of molecular oxygen are called facultative organisms. Accordingly, biological treatment processes using aeration to supply molecular oxygen to the microorganisms are identified as aerobic processes, while processes using anaerobic microorganisms to bring about biological reactions are called anaerobic processes.

The metabolic reactions of the aerobic and the anaerobic microorganisms are essentially the same. The only difference lies in the final products of metabolism. While low energy, stable compounds such as water, carbon dioxide, nitrate and sulphate are produced in the aerobic process, high energy and unstable products such as methane, ammonia and sulphides are formed in addition to carbon dioxide in the anaerobic process. The presence of sulphides, particularly hydrogen sulphide, produces objectionable odours in the anaerobic system.

Since the majority of biological processes used for the treatment of wastewater from small communities are aerobic systems, emphasis in the following sections will be given to the discussion of the activity of aerobic microorganisms.

4.4.2 **Factors Affecting Biological Activity.** It has been established that a number of factors affect the activity of the microbial population. To ensure efficient operation of a biological treatment system, it is essential that these factors be controlled to provide optimum growth conditions for the microorganisms involved. The four environmental factors of most importance are pH, temperature, wastewater characteristics and loading conditions.

pH. The optimum pH range in a biological system lies between 6.5 and 8.5. Extremely low or high pH will exert a toxic effect on microorganisms due to the high concentration of hydrogen (H^+) or hydroxyl (OH^-) ions. Because the pH of domestic sewage is generally within the range specified, there is normally no requirement for pH adjustment. However, neutralization may be necessary if the pH of the untreated wastewater lies beyond the optimum range.

Temperature. Microorganisms display a wide variety of responses to temperature and are classified into three groups according to the temperature range in which they function best. In general, bacteria that grow best at lower than 20°C are identified as psychrophiles. Microorganisms which prefer to grow at temperatures greater than 45°C are called thermophiles. Those growing best at a temperature between 20 and 40°C are referred to as mesophiles.

Most bacteria found in municipal sewage are mesophiles, with the optimum temperature being around 35°C. The relationship between growth rate and temperature for this group of microorganisms is illustrated in Figure 37. It is generally recognized that the rate of growth doubles with every 10°C increase in temperature until some limiting temperature is reached.

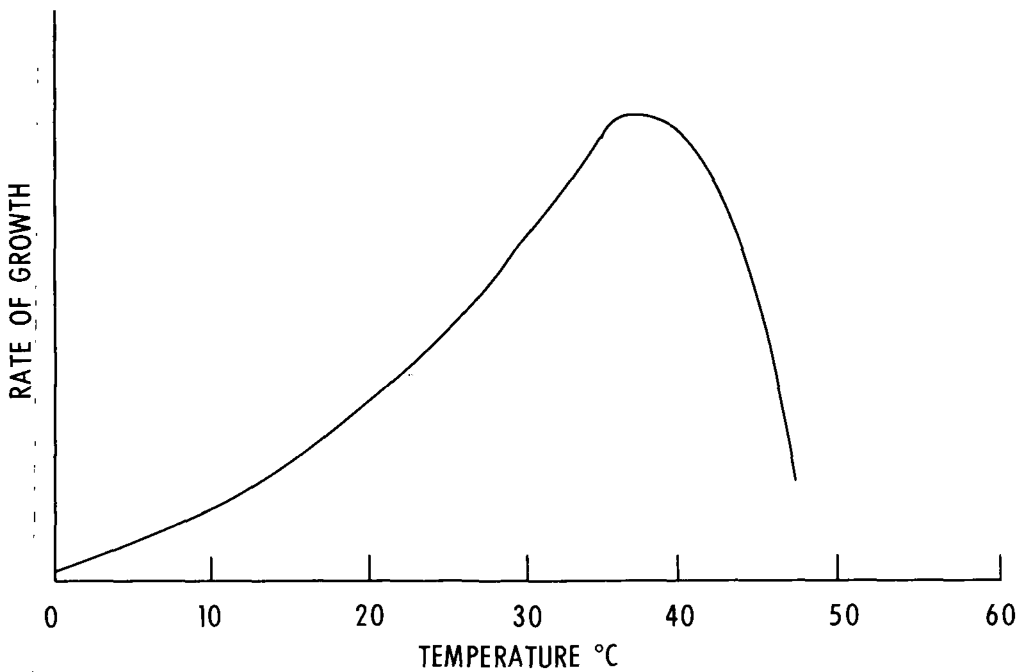


FIGURE 37 EFFECT OF TEMPERATURE ON MICROBIAL GROWTH RATE OF MESOPHILES

Microbial activity decreases with the decrease in liquid temperature. As temperatures approach freezing, the rate of growth and metabolic reactions become very slow. This temperature effect should be considered in the design of treatment systems. To be on the safe side, sewage treatment plants should be designed based on temperatures encountered in the winter months rather than on summer operating temperatures (36,37,38,41,42,44).

Wastewater characteristics. Optimum growth of the microbial population is dependent on readily available essential nutrients and trace elements. Nitrogen, phosphorous, sulphur, iron, calcium, magnesium, potassium, manganese, copper, zinc and molybdenum must be available, in addition to carbon, to satisfy the requirements for bacterial metabolism. If these elements are not present in the required concentration, they must be added to the wastewater to provide a balanced nutrient level for microbial growth. The two most critical elements which are frequently deficient in industrial wastewater are nitrogen and phosphorous. To encourage the growth of a desirable microbial population in a biological treatment system, it is advisable to maintain a BOD:N:P ratio of 100:5:1. Failure to maintain a balanced nutrient level could result in operational problems such as poor performance and profuse growth of filamentous microorganisms in the treatment plant. Fortunately, as most domestic sewages contain a sufficient quantity of organics and inorganics for microbial growth, there is generally no need to add nutrients to the wastewater (41,42).

Many industrial wastes contain substances which exert toxic effects on biological treatment processes. The discharge of these wastes into the public sewers may create serious operational problems in sewage treatment plants. Phenol, cyanide, ammonia, sulphide, heavy metals and many organic compounds may completely inhibit the microbial activity in a treatment system if their concentrations exceed the threshold limit which can be tolerated by the microorganisms. High-strength industrial waste, such as canning and dairy wastes, may cause excessive filamentous growth in the activated sludge system, rendering the system inoperative.

Loading Conditions (37,38). The amount of substrate available per unit weight of microbial mass in a biological system is called the organic loading and is identified as the food to microorganisms (F/M) ratio. The substrate concentration in the wastewater is generally measured by the BOD_5, and the microbial mass is approximated by the concentration of suspended solids. The F/M ratio can be calculated from the following equation:

$$F/M = \frac{S_o}{X_a \cdot t}$$

where: S_o = BOD_5 concentration of the wastewater (mg/L),
X_a = average concentration of MLSS in the aeration tank (mg/L),
t = detention time in the aeration tank (days).

For example, if a wastewater with a BOD_5 concentration of 200 mg/L is treated in an aeration tank with a detention time of one day and containing 4000 mg/L mixed liquor suspended solids (MLSS), then:

$$F/M = \frac{200}{4000 \times 1} = 0.05 \text{ mg } BOD_5/\text{mg MLSS} \cdot \text{d}$$

or 0.05 kg BOD_5/kg MLSS•d (0.05 lb BOD_5/lb MLSS•d).

The performance of a biological treatment system is closely related to the loading condition. At high F/M ratios, the bacteria are actively metabolizing the organic matter and usually do not form settleable flocs; i.e., in a highly-loaded treatment system, microorganisms are likely to remain dispersed throughout the system, producing an effluent with a high concentration of suspended solids. At extremely low loading conditions, endogeneous metabolism will cause disintegration of bacterial cells, giving an effluent of high solids concentration due to the presence of pin-point flocs. Biological treatment systems operated in the declining growth phase usually produce a microbial population exhibiting the optimum flocculant characteristics.

The effect of organic loading on sludge settleability is illustrated in Figure 38. The settleability of the sludge floc is measured as the sludge volume index (SVI), which is defined as the volume in millilitres occupied by one gram of MLSS after 30 minutes of settling. A good settling sludge has an index of 100 ml/g. This usually corresponds to an organic loading of 0.2 to 0.5 kg BOD/kg MLSS•d and is identified as the design range for conventional activated sludge systems.

Sludge retention time (SRT), or cell residence time, is a measure of the average retention time of solids in the activated sludge system. For a system with recycle and sludge wasting, the SRT is defined as the kilograms mixed liquor volatile suspended solids (MLVSS) in the aeration tank, divided by the kilograms MLVSS wasted per day. For a flow-through system without sludge return and wastage, the SRT is equal to the detention time for the aeration cell.

The SRT must be maintained at a level greater than the maximum generation time of the microorganisms in the activated sludge system. Otherwise, the bacteria are washed away from the system faster than they can reproduce themselves and the process fails. For microbial populations having long generation times, the operation of the

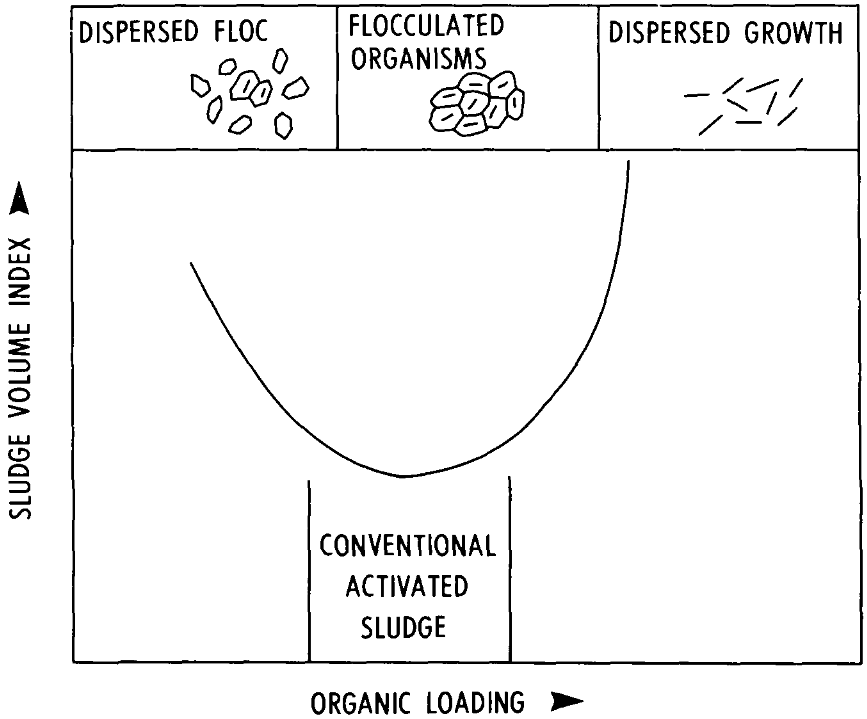


FIGURE 38 SLUDGE SETTLEABILITY AS A FUNCTION OF ORGANIC LOADING

activated sludge process must be related to the SRT rather than the F/M ratio. This condition applies for autotrophic nitrifiers and, thus, operation and performance of a biological nitrification system are related to the SRT (37,38).

Another parameter frequently related to organic loading is volumetric loading, normally expressed in terms of kg BOD_5/m^3•d. Volumetric loading is a measure of the waste load applied to a unit volume of reactor (e.g., aeration tank) and is calculated by multiplying the organic loading (kg BOD_5/kg MLSS•d) by the concentration of MLSS (mg/L).

4.4.3 Nitrification and Denitrification. In an aerobic biological treatment system, the organic nitrogen in the wastewater is converted to ammonia by heterotrophic microorganisms. Under favourable conditions, the ammonia can further be oxidized to nitrite and nitrate by autotrophic bacteria such as Nitrosomonas and Nitrobacter. The conversion of ammonia to nitrite and nitrate is identified as nitrification. This process usually takes place in biological treatment systems operated under low organic loading and high temperature conditions.

As 4.6 g of oxygen are required to oxidize 1 g of ammonia-nitrogen to nitrate nitrogen, additional oxygen should be provided in the treatment system where nitrification takes place.

Other groups of heterotrophic bacteria, such as Pseudomonas denitrificans, can reduce nitrate to nitrite and then to gaseous nitrogen under anoxic conditions. This process is called denitrification. The microorganisms use oxygen combined in the nitrate or nitrite for metabolism instead of molecular oxygen.

In many activated sludge systems, the problem of rising sludge in the clarifier is associated with denitrification. This occurs when nitrification is taking place in the aeration tank and denitrification in the clarifier. Small bubbles of nitrogen gas accumulate in the sludge and lift it to the surface of the clarifier. This, in turn, results in loss of sludge and production of effluents with high solids concentrations.

4.4.4 **Suspended Growth Biological Treatment Processes.** In the past years, a variety of biological processes have been developed for the treatment of wastewaters. They are generally classified as suspended growth and attached growth processes. In suspended growth processes, the microbial population is kept in suspension using compressed air or mechanical methods. Attached growth processes, which use microbial populations attached to a solid surface, are described in section 4.4.6. Treatment systems classified as suspended growth processes include activated sludge, contact stabilization, extended aeration, oxidation ditches, aerated lagoons, and waste stabilization ponds.

The most common process utilizing the suspended growth system is the activated sludge process. There are conventional and modified activated sludge processes which have been developed to meet specific requirements and to achieve economic advantages in operation and construction. In this section, discussion is restricted to the conventional process and the modifications which have become standardized and are considered suitable for small plant applications.

Conventional activated sludge process. The process consists of an aeration tank, a secondary clarifier and a sludge recycle line. Floating matter and settleable solids in the raw sewage are generally removed by pretreatment and primary treatment before aeration. A flow diagram of a conventional activated sludge plant is shown in Figure 39.

The process utilizes a mixed microbial population in the aeration tank to aerobically convert the organic matter into cellular material which can be subsequently separated from its suspending liquor in the secondary clarifier. The cellular material in the aeration tank is called the activated sludge or mixed liquor suspended solids (MLSS), which consists of an active mass of different species of microorganisms. A portion of the settled sludge in the clarifier is recycled to the aeration tank while the remainder is

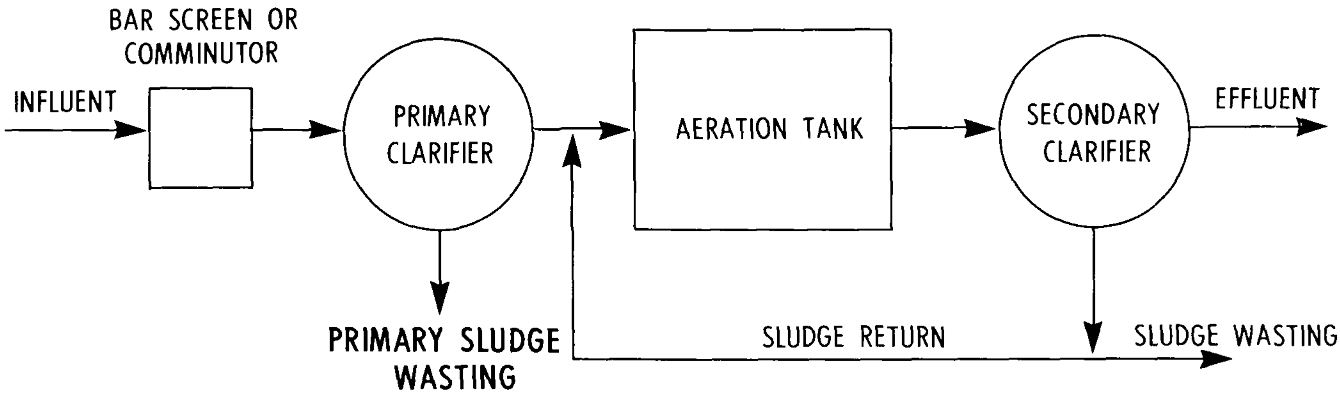


FIGURE 39 FLOW DIAGRAM OF A CONVENTIONAL ACTIVATED SLUDGE TREATMENT PLANT

wasted at a rate proportional to the rate of new cellular production. The treatment of the wasted sludge will be covered separately in section 4.5.

Design criteria for conventional activated sludge treatment facilities are (36,37,38,41):

Organic loading:	0.2-0.5 kg BOD_5/kg MLSS•d
Volumetric loading:	0.4-1.8 kg BOD_5/m^3•d
	(25-110 lb BOD_5/1000 ft^3•d)
Sludge Retention time:	5-15 days
Detention time:	3-8 hours
MLSS concentration:	2000-4000 mg/L
Sludge return ratio:	25 to 100% of process influent
Water depth:	3-5 m (10-15 ft)
Oxygen requirements:	1.2-1.5 kg O_2/kg BOD_5
Sludge production:	0.5-0.7 kg/kg BOD_5 removed.

In a properly designed and operated conventional activated sludge plant, 90 to 95% BOD reduction can be achieved for normal domestic sewage. The effluent is usually clear, odourless and high in dissolved oxygen, with effluent BOD and suspended solids concentrations in the range of 10 to 20 mg/L.

Activated sludge plants require continuous skillful maintenance. Two to four weeks are generally required for a new plant to build up the required amount of sludge. The following items should be checked regularly to ensure proper operation of the system:

a) The dissolved oxygen level should be checked at various points in the aeration tank at least twice a day; one check should be made during the period of peak loading. A minimum dissolved oxygen level of 1 mg/L, preferably 2 mg/L, should be maintained throughout the tank. Excess dissolved oxygen does not adversely affect the performance of the system, but will increase operating costs. If a dissolved oxygen deficit occurs, the aeration devices should be adjusted.

b) It is essential to check and adjust the concentration of MLSS to the required level at least once per day (see c, d and e).

c) Regular wasting of sludge should be carried out to maintain a constant concentration of MLSS in the aeration tank. This can be achieved by a continuous or batch wasting of sludge from the sludge return line.

d) Occasional adjustment of the return sludge rate may be necessary to maintain a desired concentration of MLSS in the aeration tank. Usually an increase in the return sludge rate will result in an increase in the concentration of MLSS in the aeration tank.

e) The 30-minute sludge settling test should be carried out daily. Any variation in settled sludge volume indicates a change in quantity and quality of MLSS in the aeration tank; for example, an increase in the settled volume would indicate that either the sludge concentration is increasing and some sludge wasting is required or the sludge quality is deteriorating. The sludge volume index (SVI) can also be determined to characterize the compactness and settleability of the sludge floc. A good settling sludge usually has an index of less than 100 ml/g. A rising index at an activated sludge plant is indicative of approaching trouble and prompt action should be taken to bring it under control.

f) Periodic inspection and replacement of worn mechanical parts, regular cleaning, oiling and greasing of aeration devices, compressors, pumps, etc., are essential to maintain stable, reliable and highly efficient operation of the treatment plant.

Approximately 3400 man-hours per year are required to operate and maintain a 450 m^3/d (0.1 mgd) conventional activated sludge plant comprised of the components shown in Figure 39, plus aerobic sludge digestion and sludge drying beds (45).

Operational problems associated with conventional activated sludge processes include (40):

1) Bulking sludge - The most serious problem encountered in the operation of activated sludge systems is sludge bulking. A bulked sludge has poor settling and compaction characteristics, normally caused by the presence of filamentous microorganisms (Figure 40). Bulking sludge causes two problems in the treatment system: i) continuous loss of sludge over the overflow weir of the secondary clarifier, resulting in poor effluent quality, and ii) a significant drop in the solids concentration in the sludge underflow, and therefore the MLSS concentration in the aeration tank, because of the loss of sludge over the clarifier and the lack of compaction. There is no simple and efficient method to control the different species of filamentous microorganisms associated with sludge bulking. However, the sludge bulking potential can be minimized by proper design of the treatment system, including precautions such as selection of an F/M ratio not conducive to filamentous growths; provision of adequate oxygen and nutrients; control of pH and waste load fluctuations in the aeration tank; and proper design of the secondary clarifier.

FIGURE 40 FILAMENTOUS MICROORGANISMS IN ACTIVATED SLUDGE

2) Oxygen deficiency - A minimum dissolved oxygen level of 1 mg/L, preferably 2 mg/L, should be maintained in the aeration tank at all times. Failure to maintain this level, particularly during the peak loads period, could create anaerobic conditions resulting in offensive odour, poor settling sludge and a deterioration in effluent quality. The problem can be corrected by operating the air blowers at full capacity, by installation of additional blowers or by providing equalization facilities to dampen peak loads.
3) Flow fluctuations - Fluctuation in the flow rate and characteristics of sewage could adversely affect the performance of an activated sludge system. This condition can be corrected by the installation of a flow equalization tank (Section 4.2.4).
4) Foaming problems - Large quantities of foam may be produced during start-up of the process, when the MLSS are low or whenever high concentrations of surfactants are present in the sewage. Foam usually entraps sludge solids and a large number of bacteria. The wind may lift the froth off the tank surface and create nuisance conditions. Methods for control include: spraying water on the surface of the aeration tank; increasing the solids concentration in the aeration tank; and, the addition of anti-foaming agent.
5) Clogging of air diffusers and return sludge lines is also a problem frequently encountered in small activated sludge systems, particularly in package plants. Regular inspection and cleaning are essential. Diffusers should be designed so that they can be lifted from the mixed liquor for inspection and cleaning without emptying the aeration tank. To eliminate return sludge interruptions, positive displacement pumps should be used.

Modified activated sludge processes. Among the various modified processes, extended aeration and contact stabilization have been used successfully for the treatment of wastewaters from small communities.

The extended aeration process operates in the endogenous phase of the growth curve and provides sufficient aeration time for oxidizing the biodegradable portion of the sludge synthesized from the organics removed in the process. Extended aeration is characterized by a long detention time and a high MLSS concentration. Although most of the synthesized sludge can be easily degraded in the system, a small portion of the cellular material is highly resistant to oxidation. This portion builds up and makes periodic wasting of the sludge necessary. However, the amount of sludge to be wasted is less than in the conventional activated sludge process. The sludge normally contains little putrescrible organic material and can be discharged for direct drying on sludge beds without offensive odours.

The extended aeration process has been used extensively to treat wastewater at 9.5 to 3500 m^3/d (2000 to 775,000 Igpd). Prefabricated package plants are frequently used at isolated institutions, schools, workcamps and small communities.

A flow diagram for a typical extended aeration plant is shown in Figure 41. The system is basically the same as a conventional activated sludge process except that a longer aeration period is required. Raw sewage is screened or comminuted before entering the aeration tank but primary sedimentation is generally omitted to simplify sludge treatment and disposal.

Design criteria for extended aeration are as follows (37,38,41):

Organic loading:	0.05-0.15 kg BOD_5/kg MLSS•d
Volumetric loading:	0.16-0.40 kg BOD_5/m^3•d (10-25 lb $BOD_5/1000\ ft^3$•d)
Sludge retention time:	20-30 days
Detention time:	18-36 hours
MLSS concentration:	3000-6000 mg/L
Sludge return ratio:	75 to 200% of process influent
Water depth:	1.5-3.0 m (5-10 ft)
Oxygen requirements:	2.0-2.3 kg O_2/kg BOD applied
Mixing requirements:	280 L/m•minute (length of aeration tank)
Sludge production:	0.2-0.6 kg/kg BOD_5 removed.

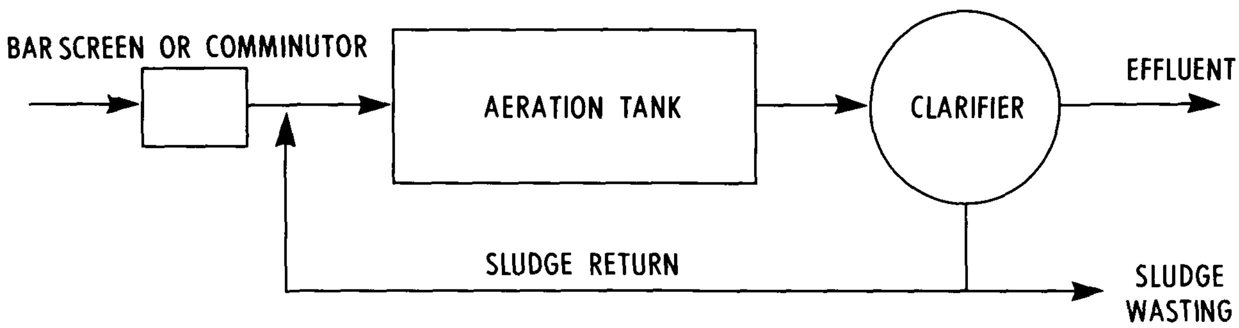


FIGURE 41 EXTENDED AERATION PROCESS FLOW DIAGRAM

BOD removals from extended aeration plants are approximately the same as for conventional activated sludge plants. Because of the extremely low loading used in the extended aeration system, disintegration of sludge flocs may occur. As a result, effluent suspended solids are usually higher than for the conventional activated sludge system. An effluent quality of 10 to 50 mg/L of BOD and 20 to 60 mg/L of suspended solids can be achieved in a properly designed and operated plant (40,41).

Monitoring requirements are basically the same as for a conventional activated sludge plant. It is erroneous to assume that an extended aeration plant will run satisfactorily without attention. Trained, skilled operators and several hours of conscientious monitoring per day are essential to efficient operation.

According to one report (45), an estimated 2100 man-hours per year are required to operate and maintain a 450 m^3/d extended aeration plant with the components shown in Figure 41 plus sludge drying beds. Another source (46) recommends 400 hours as the minimum annual requirement for successful operation of extended aeration plants in the 10 to 150 m^3/d range.

Because the principles involved in the design and operation of extended aeration and conventional activated sludge processes are approximately the same, similar operational problems can be anticipated. In addition, the following operational problems have been documented in extended aeration plants operating under Canadian conditions (46,47):

1) Insufficient biomass in the aeration tank - The maintenance of an adequate biomass in the biological reactor is essential for successful operation. However, the MLSS concentration can be affected by clogging of sludge return facilities, hydraulic overloading in the clarifier, and poor settling due to bulking sludge. Prompt action by trained operators is necessary to avoid problem situations.
2) Clogging of sludge return facilities - The greatest problems in extended aeration plants appear to be related to failure of equipment to scrape hopper-type clarifiers adequately and inoperative sludge return lines. These malfunctions cause low MLSS concentrations which, in turn, increase the F/M ratio beyond the optimum range. Air-lift pumps are used in many plants for sludge recycling because they are easy to operate and maintain. However, clogging is a common problem with these pumps, due to paper, rags, stones and sand in the sludge. Daily inspection and regular cleaning is required. Backflushing with air or water, or rodding to free the blockage are used to unclog air-lift pumps. Gravity sludge return systems have a history of

unsatisfactory operation due to lack of control of MLSS in the aeration tank and clogging of the slots which connect the settling zone to the aeration zone. Positive displacement pumps, although more expensive, eliminate these problems and improve system performance. Sludge scraping mechanisms may be incorporated into the clarifier to remove sludge adhering to the sloping sides of the clarifier and deposit it in the hopper. Manual scraping by the operator is otherwise necessary.

3) Improper aeration - In many cases, the mixing requirement for the extended aeration process governs the amount of air required (280 L/m•minute applied to the length of the aeration tank). DO concentrations are maintained at 1-3 mg/L in the mixed liquor. Over-aeration, in addition to wasting energy, increases scum formation and can create a high shearing force which disintegrates sludge floc and impairs settleability. Under-aeration causes poor treatment performance and offensive odours. Although some plants are equipped with timers to enable intermittent operation of the blowers, these do not provide enough flexibility to match aeration rates with changing organic loads. Solutions to the problem of improper aeration include adjusting the shims in the air blowers to control the air supply, providing "on/off" blower operation cycles, or installing equalization facilities to dampen the quantity and strength of wastewater entering the aeration chamber. The latter solution would also improve overall plant efficiency.

4) Long start-up periods - A long start-up period, usually greater than four weeks, is required before the treatment plant can operate at design efficiency. Seeding the treatment system with activated sludge from a mature plant treating a similar waste could significantly reduce the start-up period.

5) Offensive odours - This problem is usually caused by any one or a combination of the following factors: accumulation of solids in the comminutor pit; oxygen deficiency in the aeration tank; excessive scum accumulation on the clarifier surface; and/or prolonged storage of sludge in the clarifier. Good housekeeping, including daily cleaning of the comminutor pit, proper disposal of screenings, regular inspection and cleaning of air diffusers, skimming devices and sludge return facilities, is essential to control this problem.

6) Lack of sludge treatment facilities - Like other biological treatment processes, the extended aeration process produces excess sludge requiring further treatment and disposal. Unfortunately, facilities for sludge treatment and disposal are usually not included in the design of package plants. In plants not equipped with such facilities,

the accumulated sludge is left in the treatment system and eventually discharged over the weir of the clarifier. The solution to the problem is to establish a sludge wasting program to prevent excessive sludge from building up in the system, and to provide treatment and disposal facilities for the wasted sludge. Measurement of the MLSS concentration is essential to determine whether sludge wasting is required. A simple and reliable way to measure the sludge concentration in the aeration tank is to use a small hand-operated centrifuge. By measuring the sediment height in the centrifuge tube after centrifugation and comparing it with the calibration curve, the MLSS concentration can readily be determined in the field. The wasted sludge can be treated in an aerobic digester and dewatered in a sludge drying bed or hauled to a large sludge treatment system for processing if such facility is available in a nearby municipality. Sludge treatment and disposal options are discussed in greater detail later in this section.

The oxidation ditch is a modified extended aeration process (Figure 42). The ditch forms the aeration basin in which the raw sewage is mixed with the microbial population and converted to new cellular material. Aeration and mixing may be provided by a Kessener Brush, cage rotor or other similar device. The rotor entrains oxygen in the wastewater and provides sufficient horizontal velocity to keep all solids in suspension. The mixed liquor is continually drawn off to a clarifier where the sludge is settled and returned to the aeration basin.

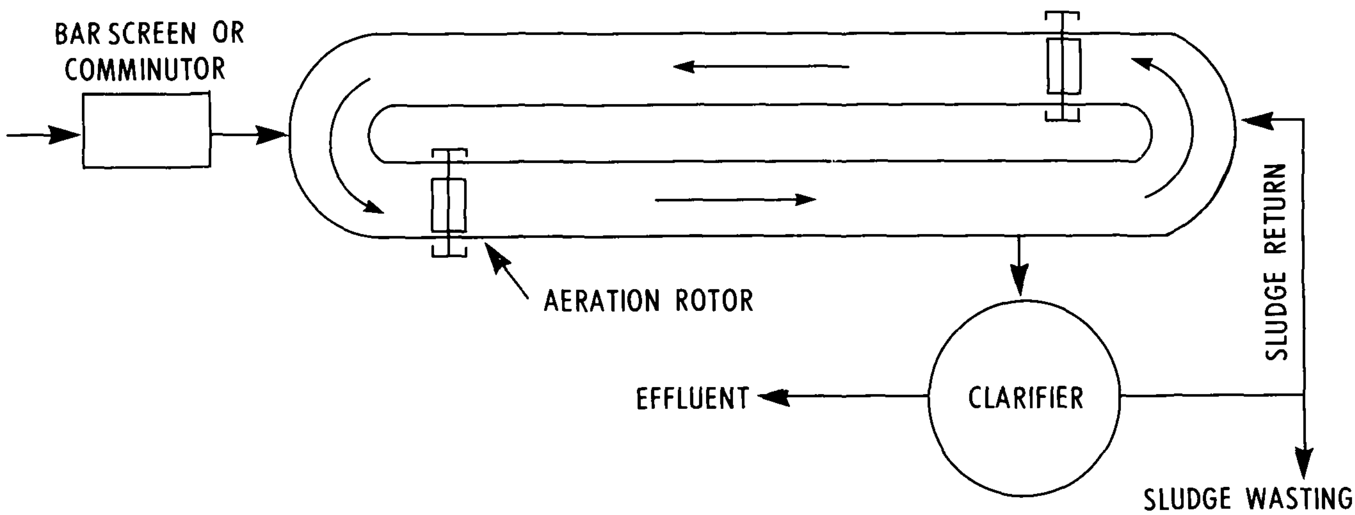


FIGURE 42 OXIDATION DITCH FLOW DIAGRAM

Design criteria for oxidation ditches are as follows:

Organic loading:	0.05-0.2 kg BOD_5/kg MLSS•d
Volumetric loading:	0.19-0.48 kg BOD_5/m^3•d
	(12-30 lb $BOD_5/1000\ ft^3$•d)
Sludge Retention time:	20-30 days
Detention time:	12-36 hours
MLSS concentration:	3000-5000 mg/L
Horizontal velocity in the ditch:	0.25-0.35 m/s (0.8-1.2 ft/sec)
Sludge return ratio:	25-75% of process influent
Shape of the aeration basin:	
Water depth	1-1.5 m (3-5 ft)
Oxygen requirements	2.0-2.3 kg O_2/kg BOD_5
Sludge production	0.2-0.4 kg/kg BOD_5 removed.

Process performance, monitoring requirements and operational problems are the same as encountered with the extended aeration process.

The contact stabilization process is particularly useful for the treatment of wastewater containing large quantities of colloidal or fine suspended organics. A typical flow diagram is shown in Figure 43. Raw sewage is combined with sludge from the stabilization tank and aerated in a contact tank for 20 to 60 minutes. In the contact unit, a large portion of the colloidal and suspended BOD is adsorbed by the cells of the floc particles for synthesis. The mixed liquor is settled in a clarifier and the majority returned to the stabilization tank where it is aerated for three to six hours. During the stabilization period, the organics adsorbed and entrapped in the sludge floc are further oxidized. This process differs from the conventional activated sludge process in that the two mechanisms, adsorption and absorption, are separated and occur in different tanks, while in the conventional sludge process, these two occur in a single tank. Furthermore, because a considerable amount of activated sludge is held in reserve in the contact stabilization process, the solids concentration can be adjusted more easily to meet different loading conditions, providing more flexible operation than the conventional activated sludge process. The total aeration volume requirements are approximately 50% of those of the conventional activated sludge plant. Thus, it is often possible to double the capacity of an existing conventional plant by modifying it to the contact stabilization process. The redesign may require only changes in plant piping or relatively minor changes in the aeration system.

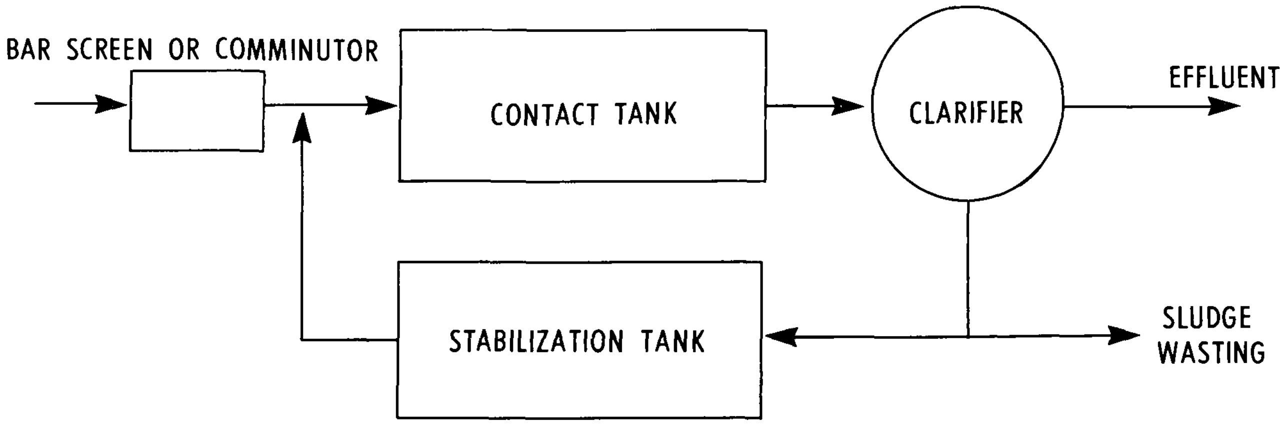


FIGURE 43 CONTACT STABILIZATION PROCESS FLOW DIAGRAM

Design criteria for contact stabilization are as follows (37,38,41):

Contact tank

Organic loading:	0.2-0.4 kg BOD_5/kg MLSS•d
Volumetric loading:	0.5-1.2 kg BOD_5/m^3•d
	(30-75 lb $BOD_5/1000\ ft^3$•d)
Sludge retention time:	6-12 days
Detention time:	20-40 minutes
MLSS concentration:	1000-3000 mg/L
Sludge return ratio:	25-100% of process influent
Water depth:	3-5 m (10-15 ft)
Oxygen requirements:	0.7-1.0 kg O_2/kg BOD_5 removed
Sludge production rate:	0.4-0.6 kg/kg BOD_5 removed.

Stabilization tank

Detention time:	3-6 hours
MLSS concentration:	4000-10 000 mg/L
Oxygen requirements:	0.3-0.5 kg O_2/kg BOD_5 removed

The contact stabilization process should be used only for wastes containing organics predominantly in colloidal or fine suspended form. It has been used successfully for the treatment of domestic sewage. The performance of the process depends greatly on the condition of the sludge in the stabilization tank. If the retention time in the stabilization tank is too short, unoxidized organics will be returned to the contact tank and the removal efficiency will be inadequate. If the stabilization period is too long, the sludge will undergo endogenous metabolism and lose its activity in the contact tank. The optimum stabilization period will produce a sludge which, when mixed with the influent waste, is ideal for the adsorption of suspended and colloidal organics. The treatment efficiency and effluent quality of a properly designed and operated contact stabilization plant are similar to those of a conventional activated sludge plant.

Monitoring requirements for contact stabilization are basically the same as for the conventional activated sludge process except that the determination and adjustment of the sludge concentration and dissolved oxygen level should be carried out both in the contact and stabilization tanks. Because two separate aeration tanks are used for different functions, a more competent operator is required for a contact stabilization plant than a conventional activated sludge plant. This limits the use of the process to large treatment plants which would have a qualified operator (40).

Operational problems encountered with the contact stabilization process are the same as those encountered in conventional activated sludge plants.

4.4.5 Aeration Equipment. Aeration equipment commonly used in the biological wastewater treatment systems previously described consist of: 1) air diffusion devices, in which air is forced under pressure through submerged porous plates, perforated pipes, or other devices so that air bubbles rise through wastewater; 2) submerged turbine aeration devices, in which compressed air is released below the rotating blades of an impeller; and, 3) mechanical surface aeration devices, in which oxygen transfer is accomplished by high surface turbulence and liquid sprays. Diffused air and submerged turbine systems accomplish oxygen transfer by bringing quantities of air into contact with the liquid, i.e., the air is the transported or principal phase. With surface aerators, the wastewater is the transported or principal phase brought into contact with air. There are also various submerged turbine devices available which incorporate both air and water transport.

In addition to providing the oxygen transfer needed for biological treatment, aerators also mix the wastewater. The mixing requirement is an important feature of design and may influence equipment selection. Good mixing is needed to keep biological life in suspension and maintain contact between dissolved oxygen and biodegradable organic material.

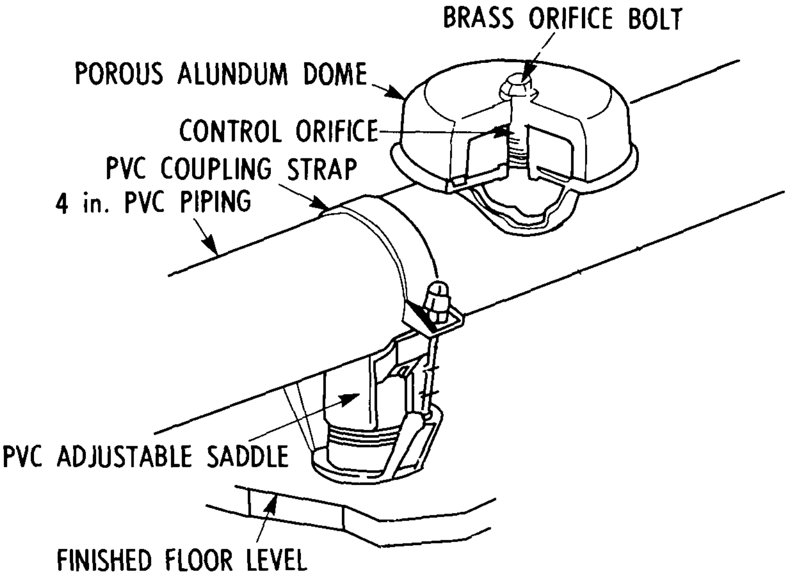
Diffused air systems (48,49). Diffused air equipment is the most commonly used in small wastewater treatment systems. Air diffusers or injection aerators bubble compressed air into the wastewater through porous or non-porous diffusers. Porous diffusers are constructed of ceramic material and produce medium and fine sized bubbles. Bubble caps, disc valves or valve orifices used in non-porous bubble diffusers generate medium to large bubbles. Large-bubble devices are easy to maintain, but have lower absorption and oxygen transfer efficiencies. Fine gas bubbles achieve greater oxygen transfer efficiencies because of the increased interfacial area of the small bubbles. However, the diffusion device can become plugged more easily than in large-bubblers. Filtering or cleaning devices on the air intake line of the compressor, and regular cleaning of the diffusion device itself can limit this problem. The time interval between cleanings will depend on the composition of the wastewater and size of the opening in the diffuser.

A somewhat different type of diffused air system is the tubular up-flow unit. Air is admitted through relatively open nozzles or slits in an air line and flows up through a tortuous pathway within a vertical tube (helical aerator). This device increases the exposure time by lengthening the route of air bubbles through the liquid. This unit is used primarily with aerated lagoons.

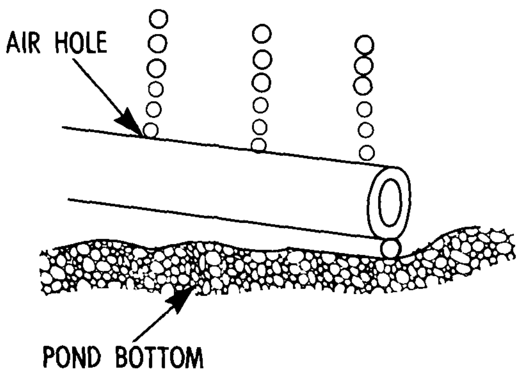
Air diffusers typically used in small wastewater treatment systems are illustrated in Figure 44.

Variables affecting the performance of diffused aeration units are the type and porosity of the diffuser, the size of the bubbles produced, diffuser air rate, the depth of submersion, velocity of surrounding medium and other factors such as pH, temperature, and wastewater characteristics and loadings. Diffusers are located at basin bottoms and spaced at intervals which are dependent upon the type of diffuser in use and the level of aeration required. Increased oxygen transfer can be achieved by locating air diffusers at greater depths below the liquid surface. However, optimum balance between oxygen transfer and mixing is usually achieved at a diffuser depth of 4 to 6 m (8 to 16 ft).

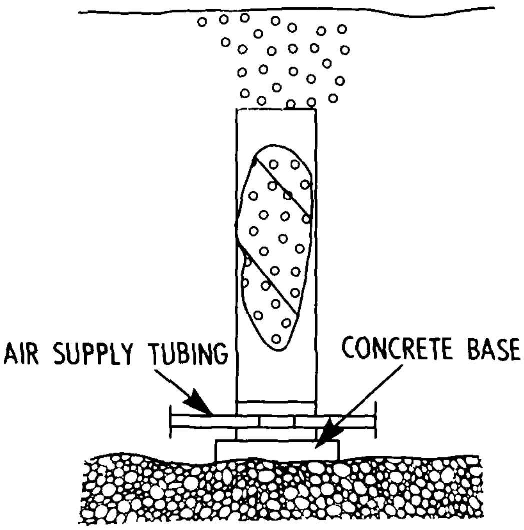
Transfer efficiencies indicated in manufacturers' literature show variations from 5 to 20 percent, with 8 percent probable from porous tube diffusers and 6 percent probable for coarse-bubble diffusers. Standard porous diffuser tubes are normally designed to deliver from 0.1 to 0.4 m^3/min (4-15 cfm) per unit. Air requirements



a) DOMED AIR DIFFUSER



b) PLASTIC TUBE AERATOR



c) HELICAL AERATOR

FIGURE 44 TYPICAL DIFFUSED AIR DEVICES

to ensure good mixing with diffused air systems will vary from 20 to 30 m^3 per minute per 1000 m^3 of tank volume.

Two types of air blowers or compressors are in common use: centrifugal, and rotary positive displacement. Centrifugal blowers are used almost universally where the unit capacity is equal to or greater than 425 m^3 per minute (15,000 cfm) of free air. Rated discharge pressures vary from 48 to 62 kPa (7 to 9 psi). For capacities smaller than 427 m^3 (15,000 cfm) per unit, rotary positive displacement blowers are generally used. The specified capacity of blowers, particularly centrifugal blowers, should take into account that air intake temperature may reach 40°C or higher and the pressure may be less than normal. The specified capacity of the motor-drive should also take into account that the intake air temperature may be -30°C or less, and may require oversizing of the motor or a means of reducing the rate of air delivery to prevent overheating or damage to the motor.

Air diffusion piping consists of the mains, valves, meters and fittings that transport compressed air from the blowers to air diffusers located in the aeration tank. Because the pressures are low (generally less than 69 kPa or 10 psi), lightweight piping may be used. The diffuser system should be capable of delivering 200 percent of the normal air requirement (50), and the spacing of diffusers should accomodate oxygen requirements through the length of the tank. The diffusers should also be installed so that they can be removed for inspection without draining the aeration tank or shutting off the air supply to other diffusers in the tank for extended periods.

Submerged turbine aerators (48,49). In turbine aeration systems, compressed air is discharged beneath a submerged rotating impeller and dispersed by the shearing and pumping action of impeller blades designed for maximum air retention in the system. (The helical aerator, Figure 44, operates on the same principle.) The unit functions mechanically to keep activated floc in mobile suspension, as well as injecting air for oxygen transfer.

One of the most common submerged turbine systems consists of a radial flow impeller located above an orifice sparge ring or an open air pipe. Air rising from the pipe is dispersed by the impeller and distributed throughout the liquid. Figure 45 illustrates a typical turbine aerator.

Submerged turbine aeration devices have an intermediate gas transfer efficiency. Oxygen transfer can be adjusted independent of mixing. This capability gives the device a decided advantage where wide loading fluctuations are experienced. The

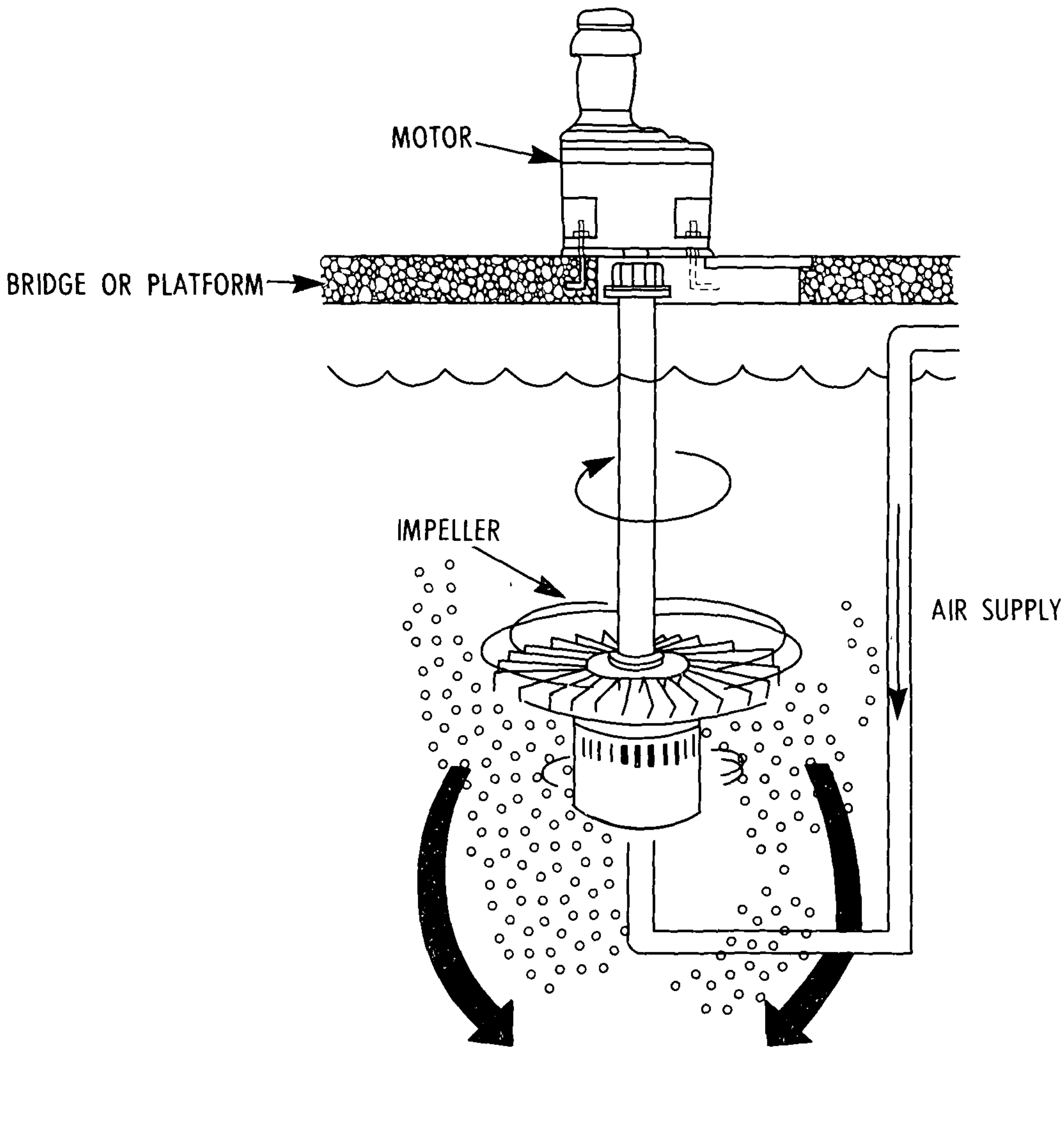


FIGURE 45 TYPICAL SUBMERGED TURBINE AERATOR

oxygen transfer efficiency of a single impeller submerged turbine is in the range of 1.1 to 1.6 kg oxygen/kWh (1.7 to 2.5 lb oxygen/hp-hr).

Mechanical surface aerators. Surface aerators can be classified into three types: radial-flow slow-speed; axial-flow high-speed; and horizontal brush rotors. The surface aerator brings wastewater to the surface for contact with air. Surface aeration equipment using a surface impeller pumps liquid from beneath the blades and sprays it across the surface of the tank. An alternate system (simplex cone aerator) uses a vertical upflow draft tube with an impeller at the top which discharges the mixed liquor over the tank surface. In this system, the contents of the tanks are continuously circulated through the draft tube. Another device, the brush rotor, uses a horizontal rotating vaned impeller to agitate the wastewater surface, transferring oxygen while moving the liquid in a horizontal direction. Oxygen transfer occurs as the waste is sprayed through the air and at the turbulent liquid surface of the aeration tank. Surface aerators generally provide higher efficiencies than other devices; oxygen transfer efficiencies for low-speed surface aerators range from 1.8 to 2.4 kg oxygen/kWh (3 to 4 lb oxygen/hp-hr). These units can be controlled for varying oxygen demand requirements through use of submergence adjustment, cycle timers and speed control with variable speed motors. The various surface aerators in use are illustrated in Figure 46.

Characteristics of different types of aeration equipment are summarized in Table 11.

4.4.6 Attached Growth Biological Treatment Processes. Attached growth or fixed film systems use microbial populations attached to a solid surface to remove the organic components from the wastewater. The fixed film systems, unlike the suspended growth systems, do not require aeration equipment to supply the oxygen and keep the biomass in suspension. The microbial population adheres to the surface of the media used and the oxygen required for the aerobic degradation of organics is transferred from the air to the microorganisms.

The most common process under this category is the trickling filter process. Recently, there has been growing interest in the rotating biological contractor (RBC) as an alternate treatment system for small communities. Although the basic metabolic reactions, microbial growth pattern and responses to environmental conditions for the fixed film system are the same as for the suspended growth systems, there is a significant difference in design, operation and cost between these two systems.

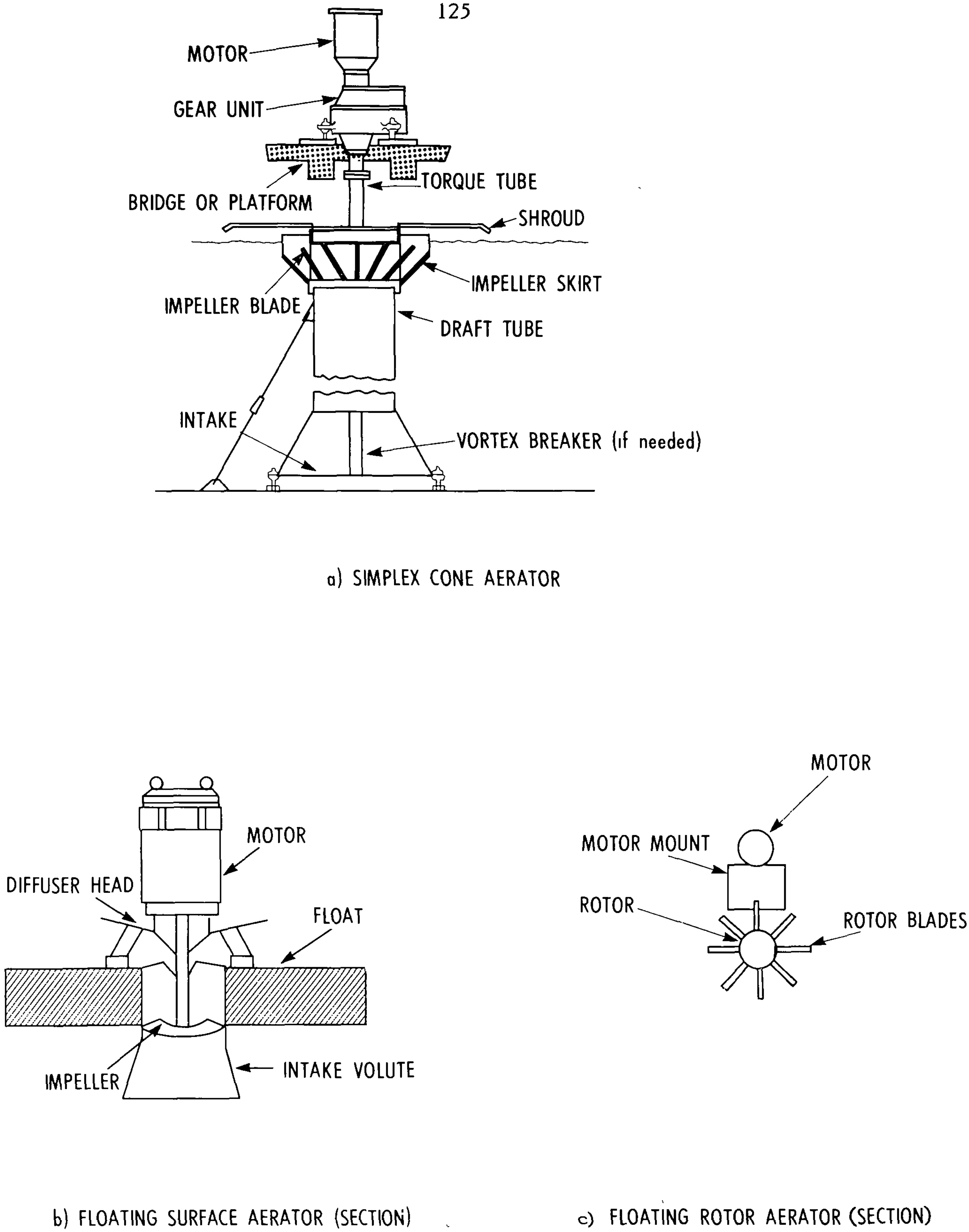


FIGURE 46 SURFACE AERATORS

TABLE 11 CHARACTERISTICS OF SOME AERATION EQUIPMENT (39)

Equipment Type	Equipment Characteristics	Processes Where Used	Advantages	Disadvantages	Reported Transfer Efficiency (kg O_2 /kWh) for conditions, O DO, 20°C, 101.4 kPa and clean water
Diffused Air:					
A. Bubbler					
Porous diffusers	Produces fine-to-medium bubbles. Made of ceramic domes, plates, tubes or plastic-cloth tube or bag.	High rate, conventional extended, step, modified, contact-stabilization activated sludge process.	Good mixing; maintains liquid temperature. Varying air flow provides good operational flexibility.	High initial and maintenance costs; air filters needed; spiral configuration limits tank geometry.	1.1-1.5
Nonporous diffusers	Made in bubble cap, nozzle, valve, orifice, or shear types, they produce coarse or large bubbles. Some made of plastic with check-valve design.	Same as for porous diffuser.	Non-clogging; maintains liquid temperature; low maintenance cost.	High initial; cost; low oxygen transfer efficiency; high power cost. Fouling may occur.	0.7-1.1
B. Tubular	Produces high shear and entrainment as water-air mixture is forced through vertical cylinder containing static mixing elements. Cylinder construction is metal or plastic.	Primarily aerated lagoons.	Economically attractive; low maintenance; high transfer efficiencies for diffused air systems. Well suited for aerated lagoon application.	Ability to adequately mix reactor basin content is questionable. Application for use in high rate biological systems unconfirmed.	1.1-1.6

TABLE 11 (Continued)

Equipment Type	Equipment Characteristics	Processes Where Used	Advantages	Disadvantages	Reported Transfer Efficiency (kg O_2 /kWh) for conditions, O DO, 20°C 101.4 kPa and clean water
Submerged Turbine	Units contain a low speed turbine and provide compressed air to diffuser rings or open pipe. Fixed-bridge application.	Same as for bubbler diffusers.	Good mixing; high capacity input per unit volume; deep tank application operational flexibility. No icing or splash.	Require both gear reducer and blower; high total power requirements; high cost.	1.0-1.5
Mechanical Surface:					
A. Radial flow, low speed, 20-60 rpm	Low output speed; large diameter turbine, floating, fixed-bridge or platform mounted. Used with gear reducer	Same as for bubbler diffuser.	Tank design flexibility; high pumping capacity.	Some icing in cold climates. Initial cost higher than axial flow aerators. Gear reducer may cause maintenance problems.	1.2-2.8
B. Axial flow, high speed, 300-1 200 rpm	High output speed Small diameter propeller. Direct, motor-driven units mounted on floating structure.	Aerated lagoons and reaeration.	Low initial cost; easy to adjust to varying water level. Flexible operation.	Some icing in cold climates; poor maintenance accessibility; mixing capacity may be inadequate.	1.2-1.5
C. Brush rotor	Low output speed; used with gear reducer.	Oxidation ditch, applied either as an aerated lagoon or as an activated sludge.	Moderate initial cost, good maintenance accessibility.	Subject to operational variables which may affect efficiency; tank geometry is limited.	1.5-2.1

Trickling filter. A trickling filter consists of a bed of inert media, such as plastic or other synthetic material, broken stone, gravel or slag, 5 to 10 cm (2.5-4 inches) in size, on which a biological slime is grown. Wastewater is distributed over the top of the bed by a rotary distributor and trickles down through the media. Organic material and oxygen are absorbed and utilized by the microorganisms attached to the filter media. The quantity of biological slime produced is controlled by available food, hydraulic dosage rate, type of media, type of organic matter, amount of essential nutrients present, temperature and the nature of the biological growth. The biological slime is sloughed off the media either periodically or continuously during filter operation.

Trickling filters are usually classified as low or high rate according to the applied organic or hydraulic loadings. The introduction of synthetic media to replace rock media allows substantially higher loadings to the trickling filter than possible with conventional rock-filled filters. Plastic media of large surface areas and void spaces have been built at depths of 6 to 12 m (25-40 ft). Most low-rate trickling filters are designed with depths ranging from 1 to 2.1 m (5-7 ft), while high-rate filters are designed with depths of 0.9 to 1.8 m (3-6 ft).

An important element in trickling filter design is the return of a portion of the filter effluent through the filter. This practice is called recirculation and the ratio of returned to incoming flow is called the recirculation ratio. Recirculation apparently increases BOD removal efficiencies in stone filter design, and prevents synthetic media from drying out.

Details of a trickling filter are shown in Figure 47, and flow diagrams of treatment plants using the trickling filter system are given in Figure 48 (37,38). The two-stage system illustrated in Figure 48 would provide a more consistent and better effluent than a single filter plant. The required level of treatment, local economics, and loading conditions will affect the final selection of the process, and combination and types of units.

The underdrain system of a trickling filter plant collects effluent and ventilates the filter, providing oxygen for the microorganisms on the filter media. Underdrain channels should have sufficient capacity to ensure that design flows occupy no more than 50 percent of the cross-sectional area of the channel. Devices such as vent stacks and manholes are sometimes designed into systems for the purpose of ventilation.

Humus sludge which sloughs off a trickling filter settles more readily and is more easily dewatered than other secondary sludges. An average of 20 to 30 percent of

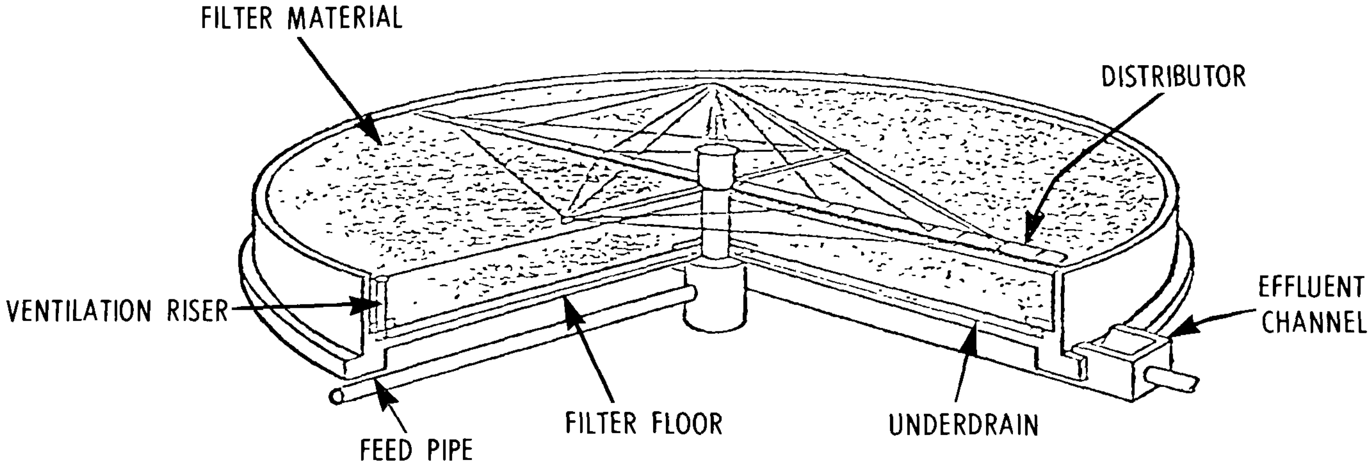


FIGURE 47 TRICKLING FILTER CROSS SECTION

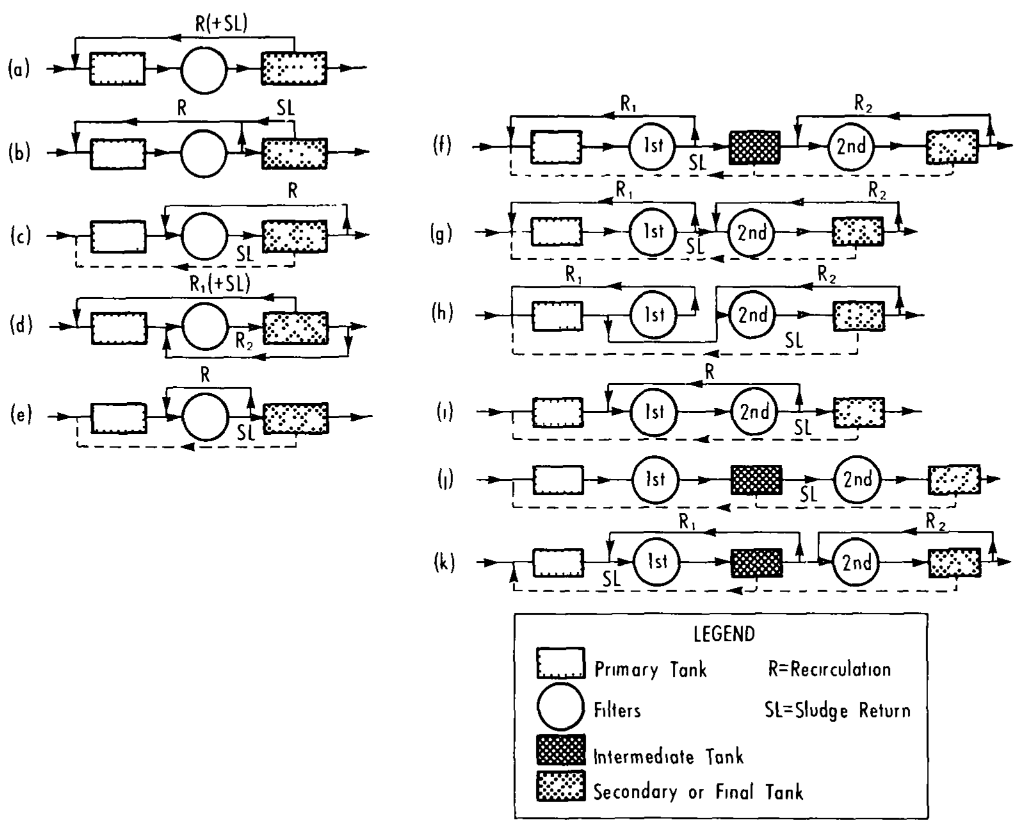


FIGURE 48 FLOW DIAGRAMS OF SINGLE AND TWO-STAGE TRICKLING FILTER PLANTS

the BOD_5 removed is converted to sludge, and thus the amount of sludge requiring treatment and disposal would be less for trickling filters than for activated sludge systems.

Trickling filter final sedimentation tanks should be designed for a hydraulic loading of 40 to 48 $m^3/m^2 \cdot d$ (1000 to 1200 gpd/ft^2) for peak flow conditions. The surface-loading rate is based on plant flow plus the recycle flow minus the underflow (37,38).

Design criteria for trickling filters are as follows:

	Conventional Media		
	Low Rate	High Rate	Synthetic Media
Organic Loading			
kg $BOD_5/m^3 \cdot d$	0.1 - 0.2	0.3 - 1.0	1.0 - 2.2
lb $BOD_5/1000\ ft^3 \cdot d$	6 - 12	20 - 60	60 - 140
Hydraulic Loading			
$m^3/m^2 \cdot d$	1.5 - 3.0	5 - 10	10 - 30
gpd/ft^2	30 - 60	100 - 200	200 - 600
Depth			
m	2 - 3	1.2 - 3	6 - 12
ft	6 - 10	4 - 10	20 - 40
Recirculation Ratio			
% of process effluent	0	100 - 400	100 - 400
Packing material	Rock, slag	Rock, slag	Plastic, redwood slats
Dosing interval	generally intermittent, not more than 5 minutes.	generally continuous	continuous
Sloughing	intermittent	continuous	continuous
Sludge production rate	0.2 - 0.3 kg/kg BOD_5 removed		

The trickling filter is a relatively simple and highly dependable device, producing a consistent effluent quality. Although effluent quality may deteriorate due to organic or toxic shock loads, the system will generally recover to good performance several hours after reversion to normal influent conditions (37,38).

Temperature is a factor of operation over which little control can be effected. Effects on the filter may be physical (freezing), biochemical (slowing reactions) or biological (lowering biological activity). The impact of temperature on efficiency of a filter can be estimated by the following relationship:

$$E_t = E_{20}\,\theta^{(T-20)}$$

where:
E_t = filter efficiency at temperature T,
E_{20} = filter efficiency at 20°C,
T = wastewater temperature, °C,
θ = constant varying from 1.035 to 1.041.

Units designed for use in cold regions should incorporate design features such as:

- enclosing filters or placing them next to a structure where heat is available;
- controlling recirculation to reduce heat loss (i.e., reduce or shut off during cold weather);
- providing covers to protect filters from wind and to control ventilation.

The expected performance for properly designed and operated trickling filters treating domestic wastewater is presented in Table 12.

TABLE 12 ESTIMATED EFFLUENT QUALITY FOR TRICKLING FILTERS

Effluent Quality	Low Rate Filters	High Rate Filters	Synthetic Media Filters
BOD (mg/L)	30 - 50	40 - 70	30 - 70
Suspended Solids (mg/L)	30 - 50	30 - 60	20 - 40

An outstanding advantage of the trickling filter process is the simplicity of operation. Unlike the activated sludge process, there is no sludge concentration and dissolved oxygen to be measured and adjusted. Provided the effluent quality is acceptable, the trickling filter process is particularly suitable for adoption in small communities where constant skilled attendance is not available.

Routine inspection for clogging of orifices or nozzles on fixed or rotating distributors should be carried out daily. The underdrain system should be inspected periodically to assure that drainage channels are neither clogged nor surcharged. Flushing or rodding of plugged nozzles or channels may be required on occasion (40).

Operational problems which can occur with trickling filter systems include (40):

a) Ponding occurs when the voids between the filter media are completely filled with biological growths. This condition may develop when the filter media are too small or the organic loading is excessive in comparison to the hydraulic loading. Ponding may be controlled by raking the filter surface, applying a high-pressure water jet, removing the filter from service for 24 hours, shutting off the distributor over the pond area to allow a continuous flow of wastewater to wash the growths out of the filter, and as a final alternative, replacing the filter medium.

b) The filter fly is a nuisance frequently associated with the operation of trickling filters, particularly in low rate filters. Dosing the filter continuously, removing excessive biological growths, flooding the filter regularly, and maintaining a clean filter and surrounding area will help prevent filter flies from becoming a nuisance.

c) Odours can be a problem because of high organic loadings, poor ventilation or poor housekeeping practices. High organic loadings (>500 mg/L) should be investigated with attention to how their impact on plant operation can be reduced. Auxiliary ventilation devices and improved housekeeping are obvious requirements when odour becomes a problem.

It is estimated that 2700 man-hours per year are required to operate and maintain a 450 m^3/d (0.1 mgd) treatment plant with the component equipment shown in Figure 48, and sludge digestion and drying facilities (45).

Rotating biological contactor. The rotating biological contactor (RBC) process is, in principle, similar to the trickling filter process and is frequently referred to as a rotating biological surface or rotating biological disc. Basically, it consists of a series of closely spaced plastic discs mounted on a horizontal shaft, supported in a semi-circular or trapezoidal concrete or steel tank. Each grouping of discs is identified as a stage and each stage operates in a separate compartment of the tank. The discs-shaft assembly is rotated slowly in the tank filled with the wastewater to be treated.

As the shaft rotates, the disc surfaces are alternately exposed to the wastewater and the atmosphere. Microorganisms naturally present in the wastewater adhere to and grow on the disc surfaces. Due to the rotating action, the discs carry a film of wastewater into the air. Oxygen is transferred from the air to the liquid film and ultimately to the slime layer. As the discs pass through the bulk of the wastewater, mixing at the disc surface is promoted and absorption of organics occurs. As the microbial growth proceeds, a biological film is formed on the disc surface. This growth eventually sloughs off under gravity or due to the shear force generated by the rotating action. The biological film that sloughs from the disc is removed by settling before the treated wastewater is discharged.

A schematic diagram of a four-stage RBC is shown in Figure 49, and Figure 50 is a flow diagram for a typical RBC plant. As noted in Figure 49, primary treatment is an integral part of the overall system (51). Primary treatment may consist of conventional primary clarification or fine screening followed by grit removal (when necessary). In small installations (19 to 115 m^3/d), primary treatment and sludge handling can be accomplished simply and effectively with a septic tank. Primary clarification or a septic tank is preferred over screening devices if large amounts of oil and grease are expected.

Final clarifiers for RBC systems have design requirements similar to those for clarifiers in trickling filter systems. Design criteria for RBC systems are:

Organic loading:	3-8 g BOD_5/m^2 disc surface•d
	(0.5-1.5 lb BOD_5/1000 ft^2•d)
Peripheral velocity:	10-25 m/minute
	(30-80 fpm)
Number of stages in series:	3-6
Sludge production rate:	0.5-0.8 kg/kg BOD_5 removed.

Because the RBC is an efficient heat transfer device, the effects of temperature must be considered in the design. The two most important effects of temperature are: i) reduced biological activity and treatment performance, and ii) ice

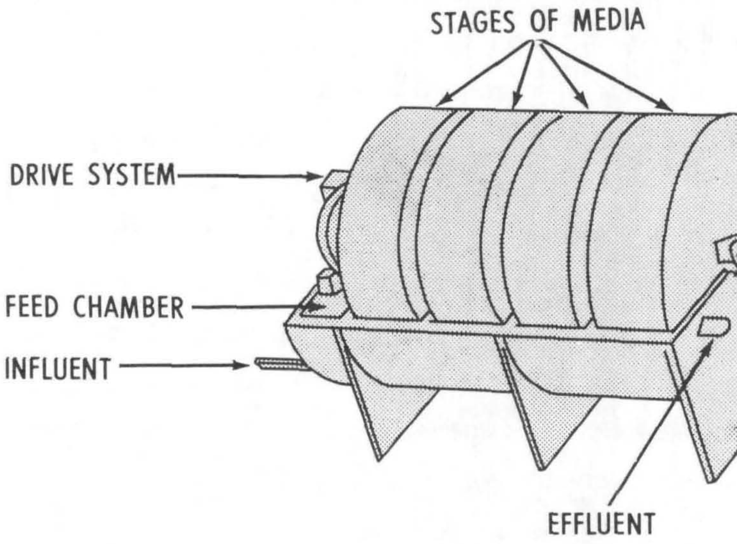


FIGURE 49 FOUR-STAGE ROTATING BIOLOGICAL CONTACTOR

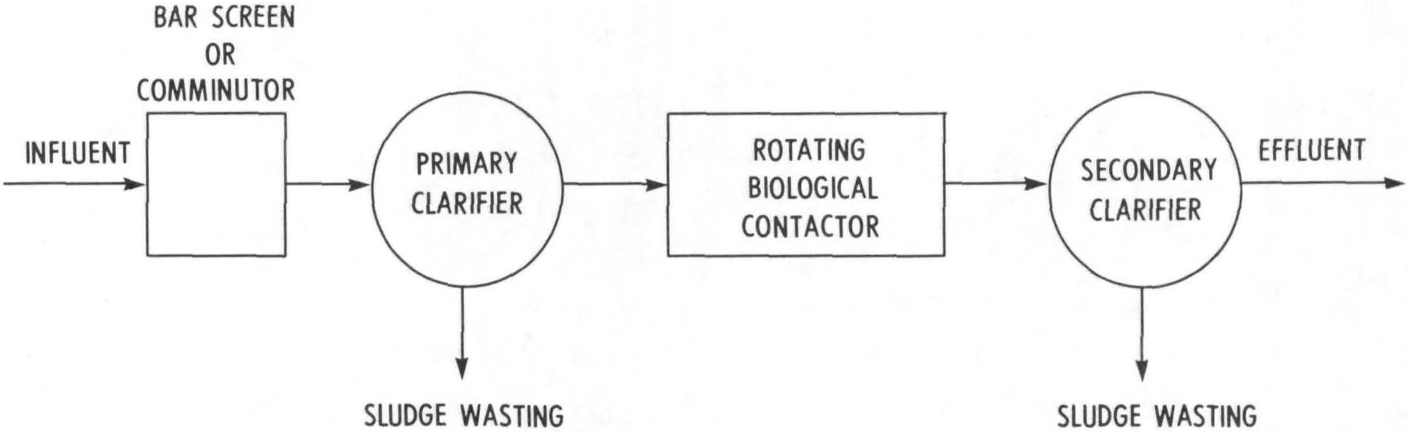


FIGURE 50 FLOW DIAGRAM OF AN RBC PLANT

formation. To achieve satisfactory operation under severe climatic conditions, it is essential to cover or provide an enclosure for the unit.

The RBC is a simple and reliable biological process which may be used successfully for the treatment of municipal wastewaters. Because of the limited residence time normally provided in these units, the RBC has little capacity to equalize or absorb varied or shock loadings. Additional equalization or a suitable design factor for areal requirements must be provided where diurnal variations in flow and BOD_5 is anticipated (51).

At the loading range specified, 90% BOD removal can be achieved in a properly designed and operated RBC. The effluent will contain 10 to 20 mg/L BOD, and 20 to 30 mg/L of suspended solids.

Like the trickling filter process, simplicity of operation is an outstanding feature of the RBC process. There are no MLSS and dissolved oxygen adjustments, no sludge volume index to measure and sludge bulking is never a problem. The mechanical simplicity of the process calls for only minimal maintenance consisting of regular wasting of sludge and periodic oiling and greasing of the drive mechanisms. This advantage renders the RBC system particularly suitable for small plant operations.

It has been estimated that 2700 man-hours per year are required to operate and maintain a 450 m^3/d (0.1 mgd) RBC plant consisting of the components illustrated in Figure 50, plus aerobic sludge digestion and sludge drying beds (45).

4.4.7 **Secondary Clarification.** The secondary clarifier or secondary settling tank separates the solids produced in the biological treatment units. This is the final step in the production of a clarified effluent in the sewage treatment plant. Although the design principles for a secondary clarifier are similar to those for a primary settling tank, special consideration must be given to the high concentration of biological solids in the mixed liquor. Clarifiers in an activated sludge plant serve a dual purpose. In addition to providing a clarified effluent, they must also provide a concentrated source of return sludge for process control. Adequate area and depth must be provided for sludge compaction to occur, while avoiding rejection of solids into the tank effluent. When the MLSS concentration is less than about 3000 mg/L, the clarifier size will normally be governed by hydraulic overflow rates (average and peak). At higher MLSS concentrations, solids loading rates become more important in determining tank size.

The depth of clarifiers in activated sludge systems is extremely important. It must be sufficient to permit the development of a sludge blanket, especially under sludge bulking conditions, and to ensure that the interface of the sludge blanket and the clarified wastewater is well below the effluent weirs. The horizontal velocity in a secondary clarifier should be limited to less than 0.8 cm/s (100 fph).

Criteria for rectangular secondary clarifiers are similar to those for primary tanks. However, it is common practice in long tanks to locate the sludge withdrawal hopper about 1/3 to 1/2 the distance to the end of the tank to reduce the effects of density currents.

Clarifiers following trickling filters and RBC processes are designed based on hydraulic overflow rates, similar to the method used in the design of primary clarifiers. Design overflow rates must include recirculated flow where clarified secondary effluent is used for recirculation. Because the influent SS concentrations are low, tank solids loadings need not be considered (37,38).

The following factors are considered important in the design of secondary clarifiers (37,38).

	Overflow Rate $m^3/d \cdot m^2$ (gpd/ft^2)		Solids Loading* $kg/d \cdot m^2$ (lb/day/ft^2)		Depth m (ft)
	Average	Peak	Average	Peak	
Settling following Trickling Filtration	16-25 (400-600)	40-50 (1,000-1,200)	- -	- -	3-37 (10-12)
Settling following Air-Activated Sludge (excluding Extended Aeration	16-32 (400-800)	40-50 (1,000-2,000)	98-147 (20-30)	<244 (<50)	3.7-4.6 (12-15)
Settling following Extended Aeration	8-16 (200-400)	32 (800)	98-147 (20-30)	<244 (<50)	3.7-4.6 (12-15)

* Allowable solids loadings are generally governed by sludge settling characteristics associated with cold weather operations.

A properly designed and operated secondary settling tank should produce a stable, well-clarified effluent low in BOD and suspended solids. Good housekeeping is essential in the operation of secondary clarifiers. It is good practice to operate the secondary clarifier with as little sludge on the bottom as possible.

Scum removal facilities are an essential component of secondary clarifiers in small wastewater systems. Scum can be drawn off continuously through pipes with flared openings placed just below the water surface, and delivered to the aeration tank by an air-lift pump or wasted.

A sludge scraping mechanism should be provided in a secondary clarifier to deliver settled sludge to the hopper. Excess sludge should be removed from the hopper or the return sludge line for disposal or supplementary treatment, preferably by a positive displacement sludge pump.

Prolonged storage of settled sludge in the secondary clarifier may result in denitrification, eventually causing sludge to rise to the surface of the clarifier, and producing effluents of poor quality. This problem is prevented if sludge-wasting and return equipment are provided with sufficient capacity (at least 200% of design flow) to ensure prompt removal of settled sludge. Mechanical scrapers should be inspected regularly and properly maintained. Sludge clinging to inclined surfaces should be removed manually by scraping or jetting.

Scum not promptly removed from secondary clarifiers may become putrescible and odourous, and deterioration of effluent quality can result.

Hydraulic shock loads should be avoided in secondary clarifiers; equalization facilities will enhance performance.

Other problems that may be encountered include plugging of sludge ports, fouling of weirs and short circuiting of flows. Regular maintenance and adjustment of equipment can alleviate many of these unexpected upsets.

4.4.8 **Lagoons.** Lagoons, or stabilization ponds, are simply basins, usually built entirely of earth, which are open to the sun and air. They may be excavated into natural soil or built above the natural grade by enclosing an area with earthen dikes after stripping off the original topsoil. A large area is usually required for lagoons, and the shape and layout frequently must be chosen to accommodate the space available. Functionally, the most important feature of a wastewater treatment lagoon is the biological life that is encouraged to grow in it. Each type of lagoon, whether it be facultative, aerated, aerobic or anaerobic, will have a particular type of predominant biological life.

Physical design parameters for lagoons have been enumerated in many texts and other publications, including those of the various provincial regulatory agencies. Therefore, only the most important parameters and recommendations will be covered herein (52,53,54).

a) Pond shape. The shape of all cells should be such that there are no narrow or elongated portions. Square or rectangular ponds with length to width ratios of 2:1 to 4:1 are considered most desirable. However, if mixing and circulation are not impaired, a lagoon of another shape may suit the topography. Dikes should be rounded at the corners to minimize accumulations of floating materials.

b) Location of pond site. When locating a lagoon system, such factors as distance from nearest existing or planned habitation, prevailing winds, surface runoff and groundwater pollution must be considered. Provincial agencies normally specify these requirements.

c) Embankments and dikes. The vertical to horizontal interior side slopes for lagoons are normally maintained at 3:1 ratio. Shallower slopes may be conducive to emergent vegetation, while steeper slopes may require rip-rap or other stabilization measures to control erosion. Outer side slopes are normally limited to a maximum grade of 3:1, with allowable minimum slopes to ensure that surface runoff does not enter lagoon cell(s). A freeboard of 0.6 to 0.9 m (2 to 3 ft) is suggested for most lagoon systems. The top width of dikes is recommended as 3 m (10 ft) to permit access by maintenance vehicles. Material used in the construction of berms should be impervious and compacted sufficiently to form a stable structure. If the native soil is porous, an impervious layer of clay (0.3 m or 1 ft thick) or a membrane liner may be considered, depending on the economics involved. Embankments should be seeded from the outside toe to one foot above the high-water line on the dikes, measured on the slope. Local agricultural or environmental agencies can usually advise the most suitable grasses to use.

d) Influent lines. The influent line to a primary cell should discharge along the centreline of the structure and terminate at approximately the third point farthest from the outlet structure. The inlet should be located over the deepest part of the sludge storage area, at a point 0.2 to 0.3 m above the expected final sludge storage level. Horizontal inlets are preferable for gravity flow because of head requirements. When the wastewater is pumped and sufficient head is available, the inlet may be directed vertically upwards.

e) Overflow structure. Overflow structures should consist of a manhole or box equipped with multiple-valved pond drawoff lines, or an adjustable overflow device (stop-logs), so that the liquid level of the pond can be adjusted to permit operation at specified operating depths. The lowest drawoff lines should be at least 30 cm

(1 ft) off the bottom to control eroding velocities (0.02 to 0.025 m/s), and to avoid pickup of bottom deposits. To ensure maximum removal of microbial cells by settling, the quiescent area near an outlet must be designed to maintain a surface overflow rate during peak flows of less than 3.0 $m^3/m^2 \cdot d$ (800 gpd/ft^2). Drain outlets may be desirable for maintenance.

f) Miscellaneous. Normally, lagoon systems must be fenced to discourage trespassing, with a locked gate provided for access by maintenance vehicles. Provisions for flow measurement should be made at both the inlet and outlet structures.

Facultative lagoons. Waste stabilization in a facultative lagoon is accomplished by a combination of anaerobic, aerobic and facultative microorganisms. The facultative lagoon is designed to permit accumulation of settleable solids on the basin bottom, where they are broken down anaerobically. The liquid and gaseous intermediate products from the accumulated solids, together with the dissolved solids from the original wastewater, supply the food for the aerobic and facultative bacteria in the surface and intermediate layers of the pond liquid. Facultative pond configurations vary. In general, configurations with three or more cells in series have been most efficient. The initial or primary cell is designed to retain the most easily settleable solids. Most suspended solids settle out rapidly near the inlet of a primary cell, reducing the actual BOD loading on the pond. However, some of the settled BOD is reimposed on the pond by the oxygen demand in the gases rising from the anaerobically digesting settled solids. Additional depth should be provided in the primary cell(s) for anaerobic digestion and storage of settled solids.

In hot weather, facultative pond water depths (exclusive of sludge storage) should be maintained between 0.9 and 1.5 m (3 and 5 ft) to control weed growth and improve odour control. In Canada, where icing conditions occur, additional depth must be provided for wastewater storage when ice cover, ice breakup, or thermal conditions may prevent the effluent from meeting stipulated quality standards (52). Operating depths generally vary with local conditions and requirements.

Because of the low organic loadings and the partially aerobic conditions during the summer, BOD removal ranging from 75 to 90 percent can be accomplished in facultative waste stabilization ponds having controlled, intermittent discharge. Effluents may be high in suspended solids, and this is normally attributable to the presence of high algae populations in the ponds. The algal growth rate is slow in late spring, and in the fall pond contents become stable and distinct thermal layers prevent mixing. These times of year are usually optimum for pond drawdown. Provincial authorities may specify a two to four-week period during April to May, and October to November as being satisfactory for discharge of lagoon contents.

Design criteria for facultative lagoons are as follows:

Parameter	Average Winter Temperature 0°C to 15°C	Below 0°C
Total system organic loading		
kg BOD_5/ha•d	22 - 45	11 - 22
(lb BOD_5/acre•d)	(20 - 40)	(10 - 20)
Detention time* (d)	40 - 60	80 - 180
Depth		
m	1.2 - 2	1.5 - 2.25
(ft)	(4 - 6)	(5 - 7)
First cell of multi-cell system:		
Extra depth for sludge storage		
m	1 - 1.5	1.2 - 2
(ft)	(3 - 5)	(4 - 6)
Depth above sludge storage		
m	1.2 - 2	1.5 - 2.25
(ft)	(4 - 6)	(5 - 7)
Retention time (d)	15 - 30	30 - 80
Organic loading		
kg BOD_5/ha•d	67 - 135	34 - 67
(lb BOD_5/acre•d)	(60 - 120)	(30 - 60)

* In locations where ice forms, consideration should be given to making detention time in the ponds 150 to 240 days, or sufficient for the period of ice cover plus 60 days, unless other means are provided to prevent odour and to polish the pond effluent. If strong winds (which prevent good sedimentation) frequently occur, the orientation of the long dimensions of the pond should be about 90° to the prevailing strong wind direction, wind breaks should be provided, and/or retention times increased (53, 54).

Estimated effluent characteristics for a facultative waste stabilization pond with intermittent controlled discharge are approximately 20 to 60 mg/L BOD_5 and 20 to

100 mg/L SS. Single-cell ponds are not as efficient as multi-cell series ponds in reducing algal and bacteria concentrations, colour and turbidity.

Pond systems should be designed to allow any one cell to be taken out of operation for cleaning. It should also be possible to allow parallel flow through primary cells, followed by series flow through the remaining cells, when sludge deposits are being removed, or when the parallel configuration reduces organic loadings more effectively and is less likely to produce odours in the primary cells. Figure 51 illustrates four process configurations.

The operation and maintenance requirements of waste stabilization ponds are minimal. Elimination of emergent vegetation, care of embankments and control of odours and mosquitoes are all that is required. Because part of the settled solids in the ponds will undergo anaerobic decomposition, the net accumulation of sludges is generally very small compared to the capacity of the ponds. For this reason, desludging may be required only at intervals of several years to prevent the ponds from filling up with solids. For small installations, a single-pond reactor may be adequate to produce an effluent of primary quality. For larger communities, or where there is a need to produce an effluent of secondary quality, two or more ponds may be required.

While parallel units provide better distribution of settled solids, series operation is optimum for high level BOD removal. In series operation, aerobic conditions to prevent odours can be promoted by recycling the final effluent to the first cell receiving raw sewage.

It has been estimated that 400 man-hours per year are required to operate a waste stabilization pond (55).

As with any biological process, temperature has a very significant effect on the performance of waste stabilization ponds. At lower temperatures, microbial activity is significantly reduced, resulting in effluent of inferior quality. To protect the receiving water from a poorly treated effluent during the winter months, the pond should be designed with sufficient capacity to accommodate all winter flows. Canadian winter conditions will usually prevent aeration and reduce sunlight penetration. Therefore, only anaerobic microorganisms will continue to function and the products of anaerobic decomposition will accumulate under the ice cover. When the ice melts, these products, particularly hydrogen sulphide, may cause odour problems in the immediate and downwind vicinity. It is therefore recommended that ponds be located at least 305 m (1000 ft) from residences and, if possible, to the lee side.

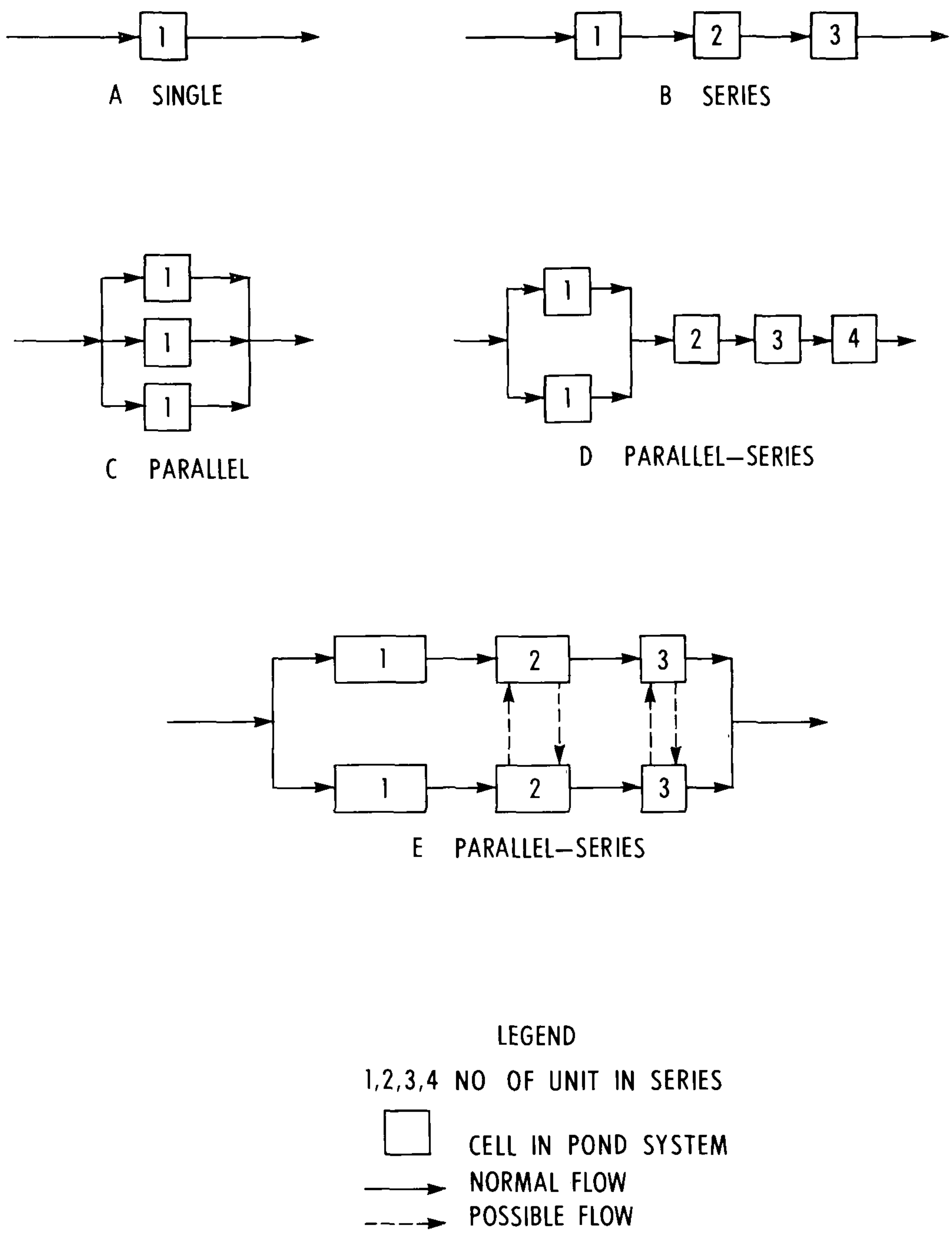


FIGURE 51 FACULTATIVE POND SYSTEM CONFIGURATIONS

The waste stablization pond is a potential breeding ground for mosquitoes and other insects. Insect generation occurs in sheltered or quiescent portions of the lagoon where there may be vegetation or layers of scum. Removal of vegetation and scum is one basic measure for insect control. Application of insecticide or periodic agitation of the pond may also limit the insect population.

Erosion of dikes by wave action or storm water can be controlled by seeding the dikes, applying rip rap, or placing stop logs around the edges of lagoons (40,53).

Burrowing animals, especially muskrats, will dig partially submerged tunnels into the dikes. The tunnel locations depend on the water level. When the water level rises, the animal extends the interior portion of the tunnel to keep it above the new water level. When the water level falls so that the entrance is no longer submerged, the burrow is likely to be abandoned. Thus, one method of controlling muskrats is to alter the lagoon level several times in rapid succession. Sometimes this will discourage the animals to such an extent that they will seek a more hospitable location.

If the slopes of the dike are sufficiently flat, a layer of sand or fine gravel may be placed on the inner slope. Because these materials will collapse when the animal attempts to tunnel into them, burrowing is likely to be discouraged. Coarse gravel should not be used because mosquitoes will tend to breed in the water in the interstices.

If the above techniques do not discourage burrowing animals, trapping may be the only feasible solution. Advice on techniques for the particular area should be sought from game and fish authorities (54).

Aerated lagoons. An aerated lagoon is a stabilization pond that does not depend on algae and sunlight to furnish dissolved oxygen for bacterial respiration, but instead depends on mechanical or diffused aeration equipment. Two types of aerated lagoons may be considered for use in small systems: completely-mixed and partially-mixed.

Completely-mixed aerated lagoons (aerobic) keep all of the incoming solids and biological solids produced from waste conversion in suspension. The essential function of this type of aerated lagoon is waste conversion. Depending on the detention time, the effluent will contain about one-third to one-half the value of the incoming BOD in the form of all tissue. Before the effluent can be discharged to a receiving stream, however, the solids must be removed by settling.

Partially-mixed aerated lagoons (aerobic-anaerobic) employ aeration devices to maintain aerobic conditions in the upper zone of the pond. A large portion of the incoming solids and the biological solids from waste conversion settle to the bottom of the lagoon where they eventually undergo anaerobic decomposition. The partially-mixed aerated pond is particularly useful in Canada because aerobic oxidation can be continued under ice cover. Partially-mixed ponds are designed to maintain a minimum of 2 to 3 mg/L dissolved oxygen in the upper zone of liquid. An important consideration in the design of aerated lagoons is the fluid pumping capacity of the aeration equipment, i.e., the energy input for oxygen dispersion. This common denominator for comparing equipment mixing characteristics in a lagoon has been established in the range of 5 to 20 minutes. Equipment manufacturers publish performance specifications in terms of pumping circulation and mixing capabilities, and these capabilities (direct pumpage and induced circulation) combine to yield a fluid pumpage factor (m^3/min or gal/min).

Aeration devices may have to be adjusted to maintain the dissolved oxygen level at greater than 2 mg/L, and turbulence may have to be controlled to meet varying operating conditions. Increasing the air supply may correct persistent odour problems in lagoons. Regular inspection, cleaning, oiling and greasing of aeration equipment is essential to reliable, efficient operation. It has been estimated that 1000 man-hours per year are required to operate and maintain a 450 m^3/d (0.1 mgd) aerated lagoon system (55).

Aerated lagoons normally discharge continuously. The biological solids produced in aerated ponds do not settle readily, thus inhibiting the production of a consistently high quality effluent. Poor solids removal or effluent quality may be caused by any of the following (40):

a) overloading,
b) low ambient temperatures,
c) ice formation,
d) toxic material in the influent,
e) short circuiting,
f) loss of liquid volume because of sludge accumulation, leakage, or evaporation,
g) aeration equipment malfunction,
h) mixing or agitation equipment malfunction,
i) operating level too deep,
j) excess turbity from storm flows or by algal mats and scum,
k) excess plant growth on dikes,
l) interference from industrial wastes.

Careful design can control many of these problem causing situations. Otherwise, routine maintenance and care of the facility by the operator is the essential element in the operation of a nuisance-free and environmentally acceptable aerated lagoon system. Operational practices described for facultative lagoon systems should also be followed for aerated systems.

Depending on the operating conditions aerated lagoons treating domestic sewage should produce effluents containing 20 to 60 mg/L BOD_5 and 30 to 170 mg/L SS.

Anaerobic lagoons. Anaerobic lagoons are normally used to treat high strength organic wastes as "roughing" ponds designed to reduce organic loadings to downstream aerobic or facultative processes. Anaerobic ponds can also be used to treat wastewater in regions which have severe winters. The low surface to volume ratio (increased depth) used in designing these ponds minimizes heat loss during winter operation. Influent wastewater is stabilized in anaerobic ponds by a combination of solids precipitation and anaerobic conversion of organic wastes to simple organic acids, methane gas, carbon dioxide, other gaseous products and cell tissues.

Design criteria for anaerobic lagoons include (37,38):

Organic loading:	
kg BOD_5/ha•d	220 - 4500
(lb BOD_5/acre•d)	200 - 4000
Volumetric loading:	
kg BOD_5/1000 m^3•d	10 - 280
(lb BOD_5/acre-ft•d)	30 - 760
Depth:	
m	2.5 - 5
(ft)	8 - 15
Detention time (days):	2 - 10

Anaerobic lagoons are employed in the pretreatment of domestic wastewaters at several locations in western Canada. In Saskatchewan and Alberta anaerobic pretreatment ponds with detention times of two to four days, followed by facultative ponds with detention times of three to six months, achieve BOD removals in the range of 45 to 90 percent, and SS removals of 64 to 91 percent. The lowest BOD removals occur during the winter. Loadings for these ponds range from 0.04 to 0.28 kg BOD_5/m^3•d, and pond depths are 2.5 to 3.5 m. Anaerobic pretreatment ponds in British Columbia are loaded at a rate of 220 kg BOD_5/ha and achieve BOD removals of 75 to 90 percent (52).

Odour problems from these anaerobic ponds appear to be less than one would assume, based on reports from British Columbia and Alberta (52). Anaerobic ponds tend to have less spring odour problems than facultative ponds, presumably because of a thinner ice cover and shorter periods of coverage. However, low-intensity odours are produced year-round, and are often accompanied by floating sludge, scum and grease (54).

In Alberta and Saskatchewan, sludge accumulation has been estimated at approximately 0.34 L per person per day. As a rule, summer accumulation is about one-third of that in the winter (52).

Regular inspection and maintenance of the lagoon system to ensure dike stability and control of insects and vegetation is required.

4.4.9 Summary. A summary of operational characteristics, performance, manhour requirements, capital and operating costs of all the biological processes discussed in the previous sections is presented in Tables 13 to 18. Operational problems and corrective measures are summarized in Table 19. As cost information for small treatment systems is not readily available, a detailed cost analysis for each unit process cannot be provided. Tables 18 and 19 are basically derived from figures reported by Tchobanoglous (45), with modifications to reflect current Canadian conditions. It should be noted that these data are intended to serve as a guide and should be revised according to local operating conditions.

The selection of the process for sewage treatment is based on considerations such as:

a) the quantity and quality of the wastewater to be treated;
b) the degree of treatment required;
c) the assimilation capacity of the receiving water;
d) the available space and the topographical conditions at the site;
e) the conditions for the handling and disposal of sludges;
f) availability of operating personnel; and
g) capital and operating costs of the process.

Because these conditions cannot be considered independently, and because the treatment process selected is dependent on the way in which these conditions are combined, fixed rules cannot be given for the selection of a particular process. In

TABLE 13 OPERATIONAL CHARACTERISTICS OF VARIOUS TREATMENT PROCESSES

Process	Operational Complexity	Power Requirement	Ease of Operation & Maintenance	Potential Environmental Impacts	Space Requirement	Temperature Sensitivity	Shock Loads Effect	Application
Conventional Activated Sludge	complex	high	extremely complex	-	small	moderately sensitive	susceptible	Small or large communities
Extended Aeration	moderate	high	complex	-	moderate	sensitive	resistant	Small communities, package plants
Oxidation Ditch	moderate	high	complex	-	moderate	sensitive	resistant	Small communities
Contact Stabilization	complex	high	extremely complex	-	small	moderately sensitive	susceptible	Package plants, applicable for the treatment of wastewater containing organics in colloidal or fine suspended form
Trickling Filter	moderate	moderate	moderate	odours & filter fly	small	sensitive	moderate resistance	Small communities particularly suitable where filamentous growth may be a problem
Rotating Biological Contactor	simple	moderate	moderate	odours	very small	sensitive	moderate resistance	Small communities particularly applicable where filamentous growth may be a problem
Aerated Lagoon	simple	low	moderate	odours	large	very sensitive	resistant	Suitable for use where very large land areas are available
Waste Stabilization	simple	no requirement	simple	odours & mosquitoes	very large	very sensitive	resistant	Suitable for use where very large land areas are available & effluent quality need not be constant

TABLE 14 ESTIMATED EFFLUENT QUALITY FOR VARIOUS TREATMENT PROCESSES*

Process	BOD_5 (mg/L)**	Suspended Solids (mg/L)
Conventional Activated Sludge	10 - 20	10 - 20
Extended Aeration	10 - 20	10 - 50
Oxidation Ditch	10 - 20	10 - 50
Contact Stabilization	10 - 20	10 - 20
Trickling Filter		
Low Rate	30 - 50	30 - 50
High Rate	40 - 70	30 - 70
Tower Filter	30 - 70	20 - 40
Rotating Biological Contactor	10 - 20	20 - 30
Aerated Lagoon	20 - 60	30 - 170
Waste Stabilization Pond		
Facultative	20 - 60 (filtered)	20 - 100
Anaerobic	40 - 120 (filtered)	80 - 160

* Based on a domestic waste of 200 mg/L BOD_5 and 200 mg/L suspended solids.
** Unless specified otherwise refers to non-filtered BOD_5.

TABLE 15 ESTIMATED ANNUAL MANHOUR REQUIREMENTS FOR VARIOUS TREATMENT PROCESSES WITH A DESIGN FLOW OF 450 m^3/d (0.1 mgd) (45, 63)

Process	Annual Manhours
Conventional Activated Sludge*	3400
Extended Aeration**	2100
Oxidation Ditch**	2100
Contact Stabilization*	3200
Trickling Filter*	2700
Rotating Biological Contactor*	2700
Aerated Lagoon	1000
Waste Stabilization Pond	400

* With aerobic sludge digestion and sludge drying bed.
** Without separate sludge digestion unit but sludge drying bed is included.

TABLE 16 ESTIMATED CAPITAL COSTS* FOR VARIOUS TREATMENT PROCESSES WITH A DESIGN FLOW OF 450 m^3/d (0.1 mgd) (45, 63)

Process	Capital Cost	Remarks
Conventional Activated Sludge (a)	\$ 250 000-300 000	(a) Aerobic sludge digestion & sludge drying bed included
Extended Aeration (b)	\$100 000-150 000	(b) without separate sludge digestion unit but sludge drying bed is included in the process.
Oxidation Ditch (b)	\$100 000-150 000	
Contact Stabilization (a)	\$250 000-300 000	
Trickling Filter (a)	\$180 000-280 000	
Rotating Biological Contactor (a)	\$120 000-180 000	
Aerated Lagoon	\$100 000-150 000	
Waste Stabilization Pond	\$ 60 000-90 000	

* Based on 1975 Engineering News Record Construction Cost (ENRCC) Index of 2200. Components comprising the processes are shown in flow diagrams for the individual processes illustrated previously.

TABLE 17 ESTIMATED OPERATION AND MAINTENANCE COSTS FOR VARIOUS TREATMENT PROCESSES WITH DESIGN FLOWS OF 450 m^3/d (0.1 mgd)

Process	Labour[a]	Power[b]	Supplies & Services[c]	Total
Conventional Activated Sludge[d]	\$23 000	\$4 400	\$1 400	\$28 800
Extended Aeration[e]	\$14 000	\$3 200	\$1 300	\$18 500
Oxidation Ditch[e]	\$14 000	\$3 200	\$1 300	\$18 500
Contact Stabilization[d]	\$22 000	\$3 800	\$1 400	\$27 200
Trickling Filter[d]	\$19 000	\$1 300	\$1 400\$	\$21 700
Rotating Biological Contactor[d]	\$19 000	\$ 700	\$1 400	\$21 100
Aerated Lagoon	\$ 6 900	\$3 200	\$1 300	\$11 400
Waste Stabilization Pond	\$ 2 700	0	\$1 300	\$ 4 000

a computed from data in Table 15 assuming 1750 productive hours per man-year and a cost of \$12 000 per man-year.

b based on figures given by Tchobanoglous (45, 63) and assuming 2.5 cents per kWh.

c based on figures given by Tchobanoglous and the 1975 ENRCC Index of 2200.

d with aerobic sludge digestion and sludge drying bed.

e without separate sludge digestion unit but sludge drying bed is included.

TABLE 18 ESTIMATED TOTAL ANNUAL AND UNIT COSTS FOR VARIOUS TREATMENT PROCESSES WITH A DESIGN FLOW OF 450 m^3/d (0.1 mgd)

	Annual Cost (dollars)				
Process	Initial Capital Cost[a]	Capital[b]	Operation & Maintenance	Total	Unit Cost (dollars/1000 gal)
Conventional Activated Sludge[c]	275 000	32 000	28 800	61 000	1.67
Extended Aeration[d]	125 000	14 700	18 500	33 200	0.91
Oxidation Ditch[d]	125 000	14 700	18 500	33 200	0.91
Contact Stabilization[c]	275 000	32 300	27 200	59 500	1.63
Trickling Filter[c]	230 000	27 000	21 700	48 700	1.33
Rotating Biological Contractor[c]	150 000	17 600	21 000	38 700	1.06
Aerated Lagoon	125 000	14 700	11 400	26 100	0.72
Waste Stabilization Pond[e]	75 000	8 800	4 000	12 800	0.35

a average from Table 16.
b based on the capital recovery factor of 0.11746 (20 years at 10% interest).
c with aerobic sludge digestion and sludge drying bed.
d without separate sludge digestion unit but sludge drying bed is included.

TABLE 19 OPERATIONAL PROBLEMS AND CORRECTIVE MEASURES FOR VARIOUS TREATMENT PROCESSES

Process	Operational Problems	Corrective Measures
Conventional Activated Sludge	1. Bulking Sludge	a) Providing an adequate nutrient supply. b) Adjust F/M ratio. c) Increase the air supply to the system. d) Chlorinate return sludge. e) Add copper sulphate or ferric chloride to the aeration tank.
	2. Oxygen deficiency	Increasing air supply to the aeration tank.
	3. Foaming	a) Water spraying. b) Increase the solids concentration in the aeration tank. c) Add antifoaming agent.
Extended Aeration or Oxidation Ditch	In addition to items 1, 2 and 3 mentioned above, the following problems could be encountered:	
	1. Long start-up period	Seed with sludge from other plants.
	2. Deposition of sands or sludges in the aeration tank	Increase the agitation.
Contact Stabilization	Same as conventional activated sludge process.	
Trickling Filter	1. Ponding	a) Reduce organic loading. b) Increase hydraulic loading. c) Raking or forking filter surface. d) Wash filter surface with pressure water stream.

TABLE 19 OPERATIONAL PROBLEMS AND CORRECTIVE MEASURES FOR VARIOUS TREATMENT PROCESSES (Continued)

Process	Operational Problems	Corrective Measures
Trickling Filter (cont'd)	2. Filter fly	a) Flooding the filter for 24 hours or more. b) Add chlorine or insecticide. c) Increase rate of recirculation to wash fly larvae out of filter. d) Maintain grounds to eliminate sanctuaries for flies.
	3. Odours	a) Practice good house-keeping throughout the plant. b) Reduce organic loading
	4. Icing on the filter ice	a) Break up and remove the ice. b) Provide a cover or enclosure for reactor.
Rotating Biological Contactor	Odours	a) Practice good housekeeping. b) Reduce the organic loading.
Aerated Lagoon	1. Odours	a) Increase the air supply to the lagoon. b) Reduce the organic loading.
	2. Algal growth	Add copper sulphate or chlorine.
Waste Stabilization	1. Odours	Add lime.
	2. Mosquitoes	a) Periodic agitation of the pond. b) Apply insecticide.

comparing the performance of sewage treatment processes, it must be realized that the operation of these systems is as important, or more so, than the design. Satisfactory performance of a well-designed plant can only be achieved by proper operation of the system. Therefore, the operating skill and the attention required should be taken into consideration during the design of a particular system.

4.5 Sludge Treatment and Disposal

One of the objectives of wastewater treatment is the removal of solids that otherwise would affect the water quality in receiving water bodies. These solids are referred to as sludge and are produced in many forms and various quantities. The amount and quality of sludge depends on the origin of the waste, the type of treatment plant and the method of plant operation.

Two principle sludge characteristics must be known to assess the type of processing required before disposal: sludge volume and solids concentration. Table 20 presents data for typical sludges produced in several conventional treatment processes, although for particular systems analysis of wastewater characteristics and process efficiencies is necessary to accurately determine sludge quantities (56,57).

TABLE 20 TYPICAL SLUDGE QUANTITIES AND CONCENTRATIONS

Treatment Process	Sludge Volume (m^3/1000 m^3 wastewater)	Solids Concentration in Sludge (kg/1000 m^3 wastewater)	(%)
Primary Sedimentation	2 - 3	120	4 - 6
Trickling Filter			
high rate	2 - 3	90	2 - 5
low rate	1 - 3	54	4 - 6
Activated Sludge			
primary & conventional	10.5 - 16	200 - 300	1.5 - 3.0
conventional	8.5 - 13	80 - 190	1.0 - 1.5
extended aeration	3.3 - 7	50 - 150	1.5 - 2.0
high rate	14 - 19	110 - 230	0.8 - 1.2

Several sludge treatment and disposal processes are used at wastewater treatment facilities. Many of these are not economical or are too complex for small treatment plants. Reduction and stabilization of organic compounds in sludges at small plants is normally accomplished by aerobic digestion or sludge lagooning. If acceptable land is available, the discharge of waste sludges to sludge ponds minimizes many operational problems.

Sludge dewatering is usually accomplished at small plants using sludge drying beds. Dried sludges or slurried stabilized sludges may ultimately be disposed of by application onto land or sanitary land fills.

4.5.1 Aerobic Sludge Digestion. The primary purpose of aerobic sludge digestion is to reduce the complex organic matter present in sludges to a simple, non-objectionable state. The process is based on the fact that microorganisms, in the absence of an external substrate, enter the endogeneous phase of the life cycle and are forced to metabolize their own cell tissue. The result is a decrease in the microbial population or sludge mass. In the process, approximately 70 to 80% of the cell mass can be oxidized to carbon dioxide and water, with the remainder being non-biodegradable. Sludge drawn from the aerobic sludge digestion process is low in organics and contains only inert materials that can be disposed of with little difficulty.

Aerobic sludge digestion can be used to treat sludges generated from plants employing conventional activated sludge, contact stabilization, trickling filter and rotating biological contactor processes. Since the mechanisms involved in the extended aeration and oxidation ditch processes are similar to aerobic digestion, sludges from these two processes are usually already highly stabilized. Further treatment of these sludges is generally not required and they can be discharged for direct drying on sludge drying beds.

Reactors used for aerobic digestion of sludges are basically the same as the aeration tanks used in the suspended growth systems. They are either circular or rectangular and are equipped with aeration equipment to provide oxygen and agitation in the process.

Aerobic digestion tanks are open to the atmosphere and do not generally have any special heat transfer equipment. The tank design in small systems should be flexible enough that the digester tank can also act as a sludge thickening unit. Design criteria for aerobic digestion are (41,56):

Solids loading:	
kg VSS/m^3•d	1.6 - 3.2

(lb $VSS/ft^3 \cdot d$)	(0.1 - 0.2)
Solids retention time (days):	
waste activated sludge	45 - 90
primary or primary +	
waste activated sludge	50 - 100
Dissolved oxygen level in	
liquid (mg/L):	1 - 2
Air requirements:	
waste activated sludge	
$L/min/m^3$	20 - 35
($cfm/1000\ ft^3$)	(20 - 35)
primary or primary +	
waste activated sludge	
$L/min/m^3$	>60
($cfm/1000\ ft^3$)	(>60)
Temperature (°C):	10 - 25

To extend the SRT in a digester beyond the relatively fixed hydraulic residence time, the system should include equipment and the operational flexibility to permit thickening of digesting sludge, decantation of supernatant, and withdrawal of the concentrated sludge. Most aerobic digesters are operated in such a way that decantation and thickening are achieved on a batch rather than continuous basis. After the aeration equipment is shut off and solids are settled, the supernatant is decanted by swing air lift or telescopic valve decanting devices, gates, weirs or selector pipes. The liquid level in the batch process fluctuates as the supernatant is decanted; therefore, weirs for continuous overflow are seldom necessary on the tanks. Air lift pumps and nonclog sludge pumps are used to draw off the thickened, digested sludge for secondary digestion or ultimate disposal.

Volatile solids in a waste sludge may be reduced by 20% to 50%, depending on such critical variables as VSS loading rates, temperature and solids retention times. In 1972, a study of seven Ontario sewage treatment plants using aerobic digesters concluded that a sludge age of 120 days would be necessary to achieve a completely stable sludge under Ontario climatic conditions and existing loading limits. The digester capacity required to achieve this level of stability was economically impractical, however, and trade-offs between required stability and final disposal methods were suggested (58). Disposal on land, for example, could require sludge ages as low as 45 days, depending on the location and method of application. Where local conditions make a more stable sludge necessary, a sludge age of 90 days was considered acceptable (58).

The operating temperature range of aerobic digesters is affected by factors such as the hydraulic retention time, and heat sources and losses within the system. While long retention times cannot be avoided if a stable sludge is required, temperature sources and sinks can be controlled. For example, air supply can be used as a heat source in diffused air systems. The simplest method of regulating the temperature of digester contents is to have a common steel wall between the digester and the activated sludge tank. The contents of the digester will then be kept at a temperature similar to that of the raw wastewater. If the digester is an isolated tank, earth embankments or covers on the tank should be used to minimize heat loss during the winter, or the tank should be placed below grade. An isolated steel tank completely above grade presents the worst heat loss problems during winter.

Compared to the anaerobic sludge digestion process, which is generally used in large sewage treatment plants, the operation and maintenance of the aerobic digester is much simpler. Routine monitoring requirements include regular testing of the dissolved oxygen level in the aeration tank, and daily shutdown of the aeration devices to permit settling of sludge and withdrawal of the clear supernatant. Depending on the degree of stabilization, periodic wasting of the digested sludge for disposal should be carried out to allow space for subsequent sludge addition. Regular inspection and replacement of worn mechanical parts, constant cleaning, oiling and greasing of aeration equipment, pumps, etc., are essential to maintain stable and highly efficient aerobic sludge digesters.

The most common operational problem associated with aerobic digestion is clogging of air diffusers. Diffuser clogging is caused when aeration/mixing equipment is turned off to concentrate digested sludge before the supernatant is decanted and sludge is drawn off. Over a period of time, fine particulates within the digester tend to lodge on and inside the diffuser mechanisms, eventually choking the air discharge. Replacement of existing diffusers with sock-type devices or with coarse bubble diffusers can prevent this problem. The use of swing-type diffusers that can be withdrawn from service without emptying the aeration tank is also recommended (40).

Low dissolved oxygen may also occur, because of loss of efficiency in the aeration system or increased VS loading to the digester.

Loss of aeration efficiency is the result of blower, mechanical aerator, or aeration appurtenance deterioration. Air delivery rate and pipeline pressures should be checked. A survey of the position of all air valves should be made. Clogging, leaks, and pressure drop should be considered. The horsepower delivered to mechanical aeration shafts should be checked. Mechanical aerator efficiency decreases if the liquid level exceeds certain upper and lower bounds. The manufacturer's engineering data should be examined for liquid level limits.

An increased VS loading can cause troubles whenever the rate of oxygen required for the aerobic digestion reaction is greater than the rate at which oxygen can be transferred into the sludge liquid by the plant's aeration system. Usually, the problem may be solved by reducing the organic loading rate and/or increasing the SRT. This can be done by decreasing influent sludge solids concentration, the total volume added, and the amount of sludge withdrawn from the digester (40).

Incomplete digestion can cause odour problems in an aerobic digester. This is counteracted by increasing the solids residence time by supernatant decantation, and then increasing the DO concentration by additional aeration (1 mg/L minimum).

Excessive foaming may be caused by factors as simple as organic overloading or as complex as filamentous bacterial proliferation. Reduction of foaming may be accomplished through such measures as decreasing organic loading, installing foam breaking water sprays, reducing excess aeration rates, and using defoaming chemicals. Water sprays should be operated sparingly to avoid dilution of the digester contents (40).

Curing filamentous bacterial growth can be a difficult task. Oxidants such as chlorine and hydrogen peroxide have been used with varying success. Shocking the system with several hours of anaerobic conditions also has been attempted. In most cases, the foaming must be controlled by limited sprays and defoaming agents until natural forces cause a shift to a nonfilamentous type of bacteria (40).

Deposition of solids may occur when gritty materials enter the digester and/or when the aeration/mixing devices do not create enough turbulence to resuspend the solids following a supernatant decantation sequence. Prevention of solids deposition may be implemented by installing or improving the operation of a grit chamber at the head of the plant. Other methods include installation of a grit separator on the sludge feed stream and the use of more powerful aeration/mixing equipment (40).

Extended subfreezing weather may lead to ice formation on the liquid surface and on mechanical aeration equipment. To prevent malfunction and possible breakdown, the operator should examine open digesters for ice block formation during the winter. Ice should be broken and removed before it damages digester appurtenances by wind action or expansion forces. Mechanical aerators should be thawed by warm air if troubled by ice formation (40).

4.5.2 Sludge Lagoons. Lagooning is a popular method of stabilizing or disposing of sludge when suitable, inexpensive land is available. Sludge lagoons are simply large holding basins with earth embankments. They are designed either as temporary holding basins or as a means for ultimate sludge disposal. Sludge lagoons may be designed as digesters. About three years detention time is generally required for thorough digestion in lagoons, one year being required for resting without sludge addition. The lagoon should be constructed with a depth of less than 2 m (6 ft) and at least two cells should be provided. Obviously, these lagoons are only functional where cheap land is available and where neighbours will not be upset by malodours.

Sludge drying lagoons are similar to sludge drying beds (section 4.5.3) in that sludge is periodically removed and the lagoon refilled. Sludge is stablized prior to discharge to drying lagoons to reduce odour problems. Solids loading rates for drying lagoons range from 35 to 38 $kg/m^3 \cdot yr$ (2.2 to 2.4 lb/yr/cu ft) of lagoon capacity. If sludge is filled to depths of 380 mm (15 inches) or less, it may be dewatered sufficiently in three to five months to remove with a front end loader.

Permanent lagoons are holding basins which receive sludges until the capacity of the system is reached. These lagoons are a form of ultimate disposal of wastewater sludges.

Among the factors that should be considered in the design of a sludge lagoon are: a) isolated location; b) site soils; c) groundwater levels; and, d) supernatant withdrawal facilities. The potential for groundwater contamination should be carefully assessed prior to construction of a sludge lagoon.

4.5.3 Sludge Drying Beds. Sludge drying beds are commonly used in small wastewater treatment plants to dewater the sludge prior to final disposal. Two mechanisms are involved in the process: filtration of water through the sand, and evaporation of water from the sludge surface. The leachate from the sludge drying bed is returned to the plant for treatment. The process is well-suited to sludges which have undergone proper aerobic or anaerobic digestion. Sludges from the conventional activated sludge, contact stabilization, trickling filter, and rotating biological contactor processes usually contain a large amount of volatile solids which tend to putrefy, creating unpleasant odour problems. Therefore, this method is generally not suitable for handling these sludges without prior stabilization.

A typical sludge drying bed (Figure 52) consists of 15 to 30 cm (6-12 inches) of coarse sand, underlain by approximately 20 to 45 cm (8-18 inches) of graded gravel ranging in size from 0.6 to 4 cm (1 1/2 inches). Open-jointed tiles of 10 to 15 cm (4-6 inches) diameter spaced at 2.5 to 6 m (8-20 ft) are laid in the gravel to provide drainage for liquid passing through the bed. Sludge is applied to the drying bed in a layer of 20 to 30 cm (8-12 inches), depending upon local climatic conditions, and allowed to dry for two to four weeks. After drying, the sludge is removed and disposed of in a landfill or used as a fertilizer.

Enclosing drying beds with glass can improve the performance of the dewatering process, particularly in cold or wet climates. In some cases, only 67 percent of the area required for an open bed is needed for an enclosed bed (41,56).

Design criteria for sludge drying beds are (41,56):

Solids loading:	50-125 $kg/m^2 \cdot$ annum (10-25 lb/ft^2/year)
Area requirements:	0.09-0.23 m^2/person (1.0-2.5 ft^2/capita)
Sludge drying time:	2 to 4 weeks
Size of beds:	6-10 m wide x 6-30 m long (20-30 ft wide x 20-100 ft long)
Number of beds:	At least 2

A reduction of more than 60% in the sludge volume will generally be achieved in the sludge drying bed. Dried sludge has a coarse cracked surface and is black or dark brown. The moisture content is usually less than 70%.

Periodic testing of the water content of the sludge should be carried out to determine whether the sludge is sufficiently dry to be removed from the drying beds for final disposal. Sludge can be removed by manual shovelling or mechanical scraping. After the dried sludge is removed, levelling of the sand bed is necessary before the next batch of sludge is applied. Because a portion of the sand is likely to be removed with the sludge, the sand should be replenished periodically.

Well-digested sludge discharged to drying beds usually presents no odour problem. Poorly-digested sludge is offensive in odour and dewaters slowly. Oil, greases and other fatty, floating materials clog the sand and should not be deposited on the beds. Rainy, snowy or extremely cold weather will adversely affect the performance of the sludge drying bed; therefore, the beds are sometimes covered with glass or other light

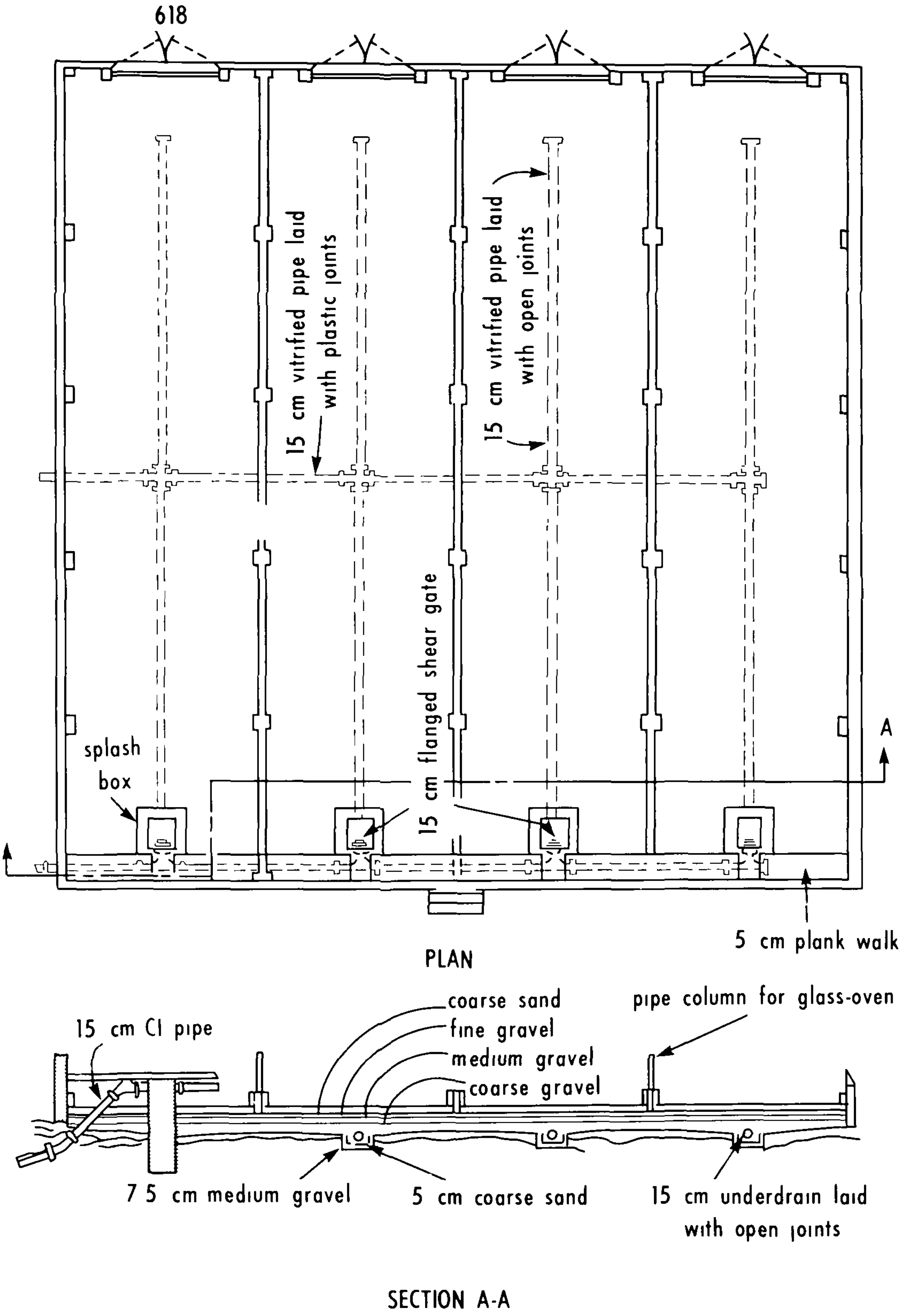


FIGURE 52 PLAN AND SECTION OF A TYPICAL SLUDGE DRYING BED (37)

transmitting materials in a structure similar to a green house to give protection against rain and snow.

4.5.4 Application of Sludges to Agricultural Land. The method of ultimate disposal of waste sludges should be selected in accordance with local, provincial and federal requirements. While no sludge residues, grit, ash or other solids should be discharged with treatment plant effluent, care must also be taken that the final procedure does not result in the indirect degradation of surface waters, groundwater, air or land surfaces.

Wastewater authorities are limited to two methods of sludge disposal:

- conversion processes (incineration, pyrolysis, composting), and
- land disposal (landspreading and landfilling).

Conversion processes are normally too expensive for small communities; incineration and pyrolysis due to energy costs, and composting because it is labour-intensive. Land-spreading and landfilling are generally recognized as the choices of disposal for small systems.

One other alternative for the disposal of sludges from small plants is to haul the sludge to a larger sludge treatment facility in a nearby municipality. The feasibility of this method depends on the hauling distance, quantity of sludge, and capacity available in the larger plant for processing the sludge. Dewatering by gravity thickening to reduce the sludge volume may be necessary to reduce the hauling cost.

Distribution on land is the most common method of disposal of wastewater sludges. The rate of application considered acceptable will depend on the system objectives. An agricultural application rate, where the objective is to recycle nutrients, will be limited by the nitrogen or heavy metal loadings. Higher loadings may be used when: 1) detailed environmental impact monitoring is being conducted; 2) reclamation of strip-mined land, or application to forests or sod farms is being considered; or, 3) non-food chain crops are grown on the site.

Planning a land application project begins with the collection of basic data on sludge characteristics, soils, climate and pertinent regulations.

Sludge data should include:

a) Current and future sludge quantities - Cost estimates, land area requirements, site life and application rates are all based in part on sludge production quantities.

b) Percent total and volatile solids - Total solids content will influence transportation and application method. Volatile solids content is an important indicator of potential odour problems.

c) Nitrogen, phosphorous, and potassium - These provide information on the fertilizer value of the sludge.

d) Heavy metals and specific organic compounds content - These provide information needed to determine the maximum annual or total application quantities.

e) Pathogens, parasites and viruses - These data are useful in assessing the degree of stabilization.

Precipitation, evapotranspiration, temperature and wind data are important parameters for determining: 1) the length of the growing season; 2) the number of days when sludge cannot be applied; and, 3) the storage requirements. Long periods of storage will be necessary if the growing season is short. Storage capacity must also include periods when inclement weather and frozen ground prevent sludge application.

Information on regulations governing sludge generation and disposal can be obtained from provincial regulatory agencies. In some cases, provinces may not have set policies regarding use of sludge on land, but information on accepted practices may be available. Throughout the planning stage, local regulatory officials and landowners should be aware of the sludge disposal strategies being considered.

Site selection procedures begin after it has been confirmed that sufficient land is available for a sludge application program. A rough estimate of the land area required can be obtained by dividing the total known or estimated quantity of sludge by an assumed application rate.

Potential application sites are evaluated based on the land use, topography and soil properties. Once an initial screening has identified the most suitable sites, more detailed evaluation can be made, including consideration of:

a) proximity of sites to homes, commercial centres, towns, etc.;
b) ready access from all-weather roads;
c) prevailing wind direction;
d) soil depth to groundwater and distance to nearest surface water;
e) total acreage available on farms being considered as disposal sites;
f) crop history;
g) prevailing soil types present and their suitability for sludge addition;
h) field slopes and general topography.

The established crop growing patterns in the community have usually developed because of favourable soil, climate and economic conditions, and should normally be maintained. One possible exception could occur if the cropping pattern is restricted to a single crop. In this case, additional crops could increase the opportunity for applying sludge over a longer period, or during different seasons.

Sludge application rates are calculated in the same manner as commercial fertilizer application rates. Annual application rates normally recommended by regulatory agencies for agricultural soils are based on the nitrogen and metal content (cadmium, lead, zinc, copper and nickel) of the sludge and the crop being grown. The "plant available nitrogen" from sludge is important in determining the application rate. From the composition of a sludge, available nitrogen can be calculated (56):

Available N = NH_4^+ + NO_3^- + 20 percent of organic N.

The life of a sludge application site is based on the cumulative amounts of lead, copper, nickel, zinc and cadmium applied to the soil. These limits are designed to preserve the soil capacity for growing useful future crops.

In Ontario, to preclude ground and surface water contamination and to control the rate of accumulation of phosphorous and metals in soil, it is suggested that the rate of application of sewage sludge should not exceed the equivalent of 134.5 kg/ha (120 lb/acre) ammonium plus nitrate nitrogen over a five-year period (59). Sludge application rates have been based on ammonium plus nitrate nitrogen because these forms of nitrogen are readily available for crop use. If sludge is to be used in commercial sod production, the frequency of application can be increased to every four years or the equivalent over a four-year period. The restriction of 134.5 kg/ha every four years for commercial sod and every five years for other crops is not set to restrict nitrogen application. This restriction is set to control the rate of accumulation and provide a wider distribution of the phosphorous and metals in soil, and facilitate their utilization by crops. It also allows some lead time to find out more about the impact of metals before soils are contaminated significantly.

When sludge is applied to agricultural land, the metals present may have detrimental effects on ground and surface water systems, soil, crop and higher food chain elements because of:

- metal build-up in soil,
- metal effects on plant growth,

- metal uptake by plants and subsequent transfer to animals and humans,
- surface water contamination via erosion and surface runoff,
- groundwater contamination via leaching following long-term metal build-up in the soil.

A two-fold approach involving maximum allowable metal concentration in soil and acceptable sludge quality is currently being considered in Ontario to control the level of heavy metal accumulation in soils and crops, and to ensure that soil metal levels do not build-up to excessive proportions. This concept is summarized in Table 21 (59).

Metals that are likely to approach critical levels under Ontario conditions are arsenic, cadmium, cobalt, chromium, copper, mercury, molybdenum, nickel, lead, selenium and zinc (Table 21, column 1). Mean metals concentrations in uncontaminated Ontario soil are given in Table 21, column 2, based on a survey of metals levels in agricultural soils conducted by the Ontario Ministry of Agriculture and Food (59). About 300 samples taken from the plough depth were collected from soils cropped with vegetables, fruits, cash crops, field crops and pastures to establish background levels.

Once a sludge amended soil has reached the maximum recommended metal content (Table 21, column 3), no more sludge should be allowed on that particular soil. Therefore, maximum recommended metal loadings in kilograms per hectare (assuming that 1 hectare of soil to the plough depth weighs 1844.4 tonnes) are given in Table 21, column 4.

In addition to setting recommendations for metal loading rates, it is also necessary to determine the acceptability of sewage sludges for agricultural use. The ammonium plus nitrate nitrogen to metals concentration ratios in the sewage sludge provide a relatively simple approach to determining which sludges are suitable for agricultural application. Sewage sludges with ammonium plus nitrate nitrogen to metal ratios larger than or equal to specified values (Table 21, column 5) are acceptable for agricultural utilization, whereas those with lower ratios are unacceptable and therefore should not be spread on agricultural land.

Also important in any sludge utilization plan is consideration of surviving pathogens, parasites and viruses. Organisms which survive sludge treatment processes can become potential hazards by:

- contaminating ground and surface waters;
- transmitting diseases to man or animals having access to sludged land;

TABLE 21 METAL CRITERIA FOR SEWAGE SLUDGE APPLICATION (59)

1	2	3	4	5	6	7
				Phase I		Phase II
Metal	Mean Metal Content of Uncontaminated Ontario soils (ppm)	Maximum Recommended Metal Content in Soil (ppm)	Maximum Recommended Metal Addition to Soil (kg/ha)	Minimum Ammonium Plus Nitrate Nitrogen (NH_4-N) plus NO_3-N) To Metal Ratios Required in Sewage Sludge	Number of Sewage Sludge Applications to Give Maximum Recommeded Metal Content in soil (Column 3)*	Ammonium Plus Nitrate Nitrogen (NH_4 -N plus NO_3 -N) to Metal Ratios Required to give Maximum Recommended Metal Content in Soil (Column 3) In 50 Applications
Arsenic	6.5	13	14.6	100	10	500
Cadmium	0.7	1.4	1.6	500	5	5000
Cobalt	4.5	18.0	30.3	50	10	250
Chromium	14.0	112	220	6	10	30
Copper	25.0	100	168	10	10	50
Mercury	0.08	0.5	0.9	1500	10	7500
Molybdenum	0.4	1.6	2.7	250	5	2500
Nickel	16.0	32	35.9	40	10	200
Lead	14.0	56	94.2	15	10	75
Selenium	0.4	1.6	2.7	500	10	2500
Zinc	54.0	216	363	4	10	20

* Based on 54.4 kg (120 lb) ammonium plus nitrate nitrogen per application and sewage sludge having minimum ratios. Number of applications for cobalt and copper are rounded off to 10.

- infecting man or animals through fruits, vegetables or other crops grown on sludged land.

Both viruses and parasitic ova and cysts can survive sewage treatment and sludge digestion; once deposited in the environment they are capable of surviving for relatively long periods in an infective state if the conditions are suitable. However, with safeguards such as minimum distances from wells and water courses, restriction of immediate access and some limitation of fruit and vegetable crops, there appears to be little danger to public health associated with spreading digested sludge on agricultural land. Past experience in Canada has indicated that there is, in fact, very little risk to either human or animal health, since no cases of infection have ever been associated with agricultural use of sewage sludge (60).

The transportation of sludge from the treatment plant to the application site is most easily accomplished in small communities by the use of tank trucks. This provides flexibility in locating land application sites and scheduling hauling, and permits direct application. Commercial tank trucks are available from companies handling equipment for sewage and sludge. If sludge disposal requirements are small, and the application site is relatively close to the treatment plant a farm tank wagon and a tractor may be sufficient to provide the necessary service.

An effective monitoring program, and its associated costs, must also be considered during the planning stages of a land application program. Monitoring is necessary to evaluate the success of a project. Factors of prime consideration are:

- public health impact through disease transmission,
- toxic materials and their impact,
- nitrogen compounds and their impact on ground and surface waters.

The objectives of system monitoring can be fulfilled by developing a program which evaluates:

a) applied sludge characteristics,
b) soil characteristics,
c) groundwater and surface water characteristics,
d) quality of vegetation produced.

Specialized testing methods will normally be specified by the local regulatory agency.

4.5.5 Use of Sludge for Landfill. Wastewater sludges, both stabilized and unstabilized, may be used satisfactorily for landfilling. Generally, only stabilized sludges are recommended for landfilling. However, stabilization may not be required in all provinces and special procedures for landfilling such sludges should be followed closely.

Several alternative methods and sub-methods are used in sludge landfilling, including (61,62):

1) Sludge-only trench fill
   a. narrow trench
   b. wide trench
2) Sludge-only area fill
   a. area fill mound
   b. area fill layer
   c. diked containment
3) Co-disposal
   a. sludge/refuse mixture
   b. sludge/soil mixture

For sludge-only trenches, subsurface excavation is required so that sludge can be placed entirely below the original ground surface. Trench applications require that groundwater and bedrock be sufficiently deep to allow excavation and still maintain sufficient buffer zones between the bottom of sludge deposits and the top of groundwater or bedrock. At sludge-only area fills, sludge is usually placed above the original ground surface. Because excavation is not required and sludge is not placed below the surface, area fill applications are particularly useful in areas with shallow groundwater or bedrock. In area fills, the landfilling usually consists of several consecutive lifts or applications of sludge/soil mixtures and cover soil. Stabilized sludges are better suited for area filling than unstabilized sludges. A co-disposal operation is defined as the deposit of sludge at a refuse landfill. In a sludge/refuse mixture operation, sludge is deposited at the working face of the landfill and applied on top of the refuse. The sludge and refuse are then mixed as thoroughly as possible and covered with soil. In a sludge/soil mixture operation, sludge is mixed with soil and applied as interim or final cover over completed areas of the landfill. One advantage of the sludge/soil mixture operation is that it removes sludge from the working face of the landfill where it may cause operational problems, while at the same time the mixture can be used to promote vegetation over completed fill areas. Only well-stablized sludges are recommended for use in sludge/soil mixture operations.

The potential nuisance or health hazards that may occur must be considered when selecting sites for landfill operations. The landfilling method used is dependent on the site characteristics, and the acceptability of a given combination of landfill method and site are, in turn, dependent on the characteristics of the sludge. Therefore, the first step in planning a landfill operation is a thorough investigation of sludge characteristics, followed by concurrent evaluation of available sites, and the landfilling method to be used.

Table 22 is a compilation of sludge characteristics and site conditions, and suggested landfilling methods (61). It is important to note that there may be no one best method for a given sludge or site. Rather, amenable landfilling methods for given sludges and sites are suggested.

Monitoring at a sludge landfill usually addresses potential groundwater and/or surface water contamination, and occasionally gas migration. The type and nature of a monitoring program is highly site specific and a hydrogeologist should be consulted.

4.6 Physical-Chemical Treatment

Physical-chemical processes, such as chemical coagulation, sedimentation, filtration and carbon adsorption have been successfully used in the treatment of domestic sewage and a number of industrial wastes. They can be used independently to provide a complete treatment of wastewaters, thus eliminating the need for biological treatment, or combined with a biological treatment system to improve the plant performance. Chemical coagulation and filtration are used to remove suspended solids, whereas activated carbon is employed to adsorb soluble organics in the wastewater. In many cases, sedimentation is incorporated into the process to remove settleable solids. Chemical coagulation is used to remove phosphorus.

Compared to biological treatment systems, physical-chemical processes have the following advantages:

a) They can be brought into operation or restarted quickly and easily. Unlike the biological process, seeding and acclimatization of sludges is not required.

TABLE 22 SLUDGE AND LANDFILL SITE CONDITIONS (61)

Method	Sludge Solids Content	Sludge characteristics	Hydrogeology	Ground Slope
Narrow trench	15-28%	Unstabilized or stabilized	Deep groundwater and bedrock	<20%
Wide trench	$\geq$20%	Unstabilized or stabilized	Deep groundwater and bedrock	<10%
Area fill mound	$\geq$20%	Stabilized	Shallow groundwater or bedrock	Suitable for steep terrain as long as level area is prepared for mounding
Area fill layer	$\geq$15%	Unstabilized or stabilized	Shallow groundwater or bedrock	Suitable for medium slopes but level ground preferred
Diked containments	$\geq$20%	Stabilized	Shallow groundwater or bedrock	Suitable for steep terrain as long as level area is prepared inside dikes
Sludge/refuse mixture	$\geq$3%	Unstabilized or stabilized	Deep or shallow groundwater or bedrock	<30%
Sludge/soil mixture	$\geq$20%	Stabilized	Deep or shallow groundwater or bedrock	<5%

b) Physical-chemical processes are less subject to upset from temperature changes. Their performance is not affected by the presence of toxic components in the wastewater.

c) Very high effluent quality can be obtained in a well-designed and operated physical-chemical treatment system. Significant phosphorus removal can usually be achieved simultaneously in the system.

The disadvantages of the physical-chemical processes are:

a) The total costs of physical-chemical plants are generally greater than those of comparable biological systems. The capital costs for the two systems are approximately the same; physical-chemical processes require less land area and tank volume but these savings are offset by the costs of automatic control systems, and sludge and chemical handling systems. The operating costs of physical-chemical plants are relatively high, primarily because of chemical consumption, although this cost differential is substantially reduced where chemical phosphorous removal is required in biological treatment processes.

b) Physical-chemical processes produce greater volumes of sludge than biological processes. The characteristics of the chemical sludges make handling and disposal more difficult.

c) Although the operation of physical-chemical processes is relatively simple, maintenance of the required control equipment may require more than a biological plant.

No attempt has been made to identify the number of physical-chemical treatment systems presently treating waste flows of less than 450 m^3/d. There are therefore no capital or maintenance cost data for small systems.

The most common unit processes employed in a physical-chemical plant treating wastewaters from small communities are chemical coagulation, sedimentation, filtration and carbon adsorption. Details of the design, performance, and applications of these processes, together with the disposal of chemically precipitated sludge are covered in a U.S. Environmental Protection Agency report entitled "Physical-Chemical Wastewater Treatment Plant Design" (64).

Independent physical-chemical treatment of municipal wastewater is relatively new. Several manufacturers are developing or have developed package physical-chemical wastewater treatment plants. Presumably, the application of independent physical-chemical processes has been limited by the costs and lack of experience with the systems. However, as more stringent effluent regulations come into effect the use of independent physical-chemical treatment systems may be expected to increase.

4.7 Disinfection

The elimination of bacteria and viruses in treatment plant effluent before discharge into receiving waters used for water supplies or recreation by man is the main function of disinfection. The most common method of disinfection of water and wastewater is chlorination. Alternative processes include ozonation, ultra-violet radiation, gamma irradiation, high lime treatment and heat treatment, to mention a few. These various alternatives, as yet, are unproven and/or uneconomical for use in small wastewater treatment systems and will not be discussed in this report.

4.7.1 Chlorine Reactions. Chlorine does not kill bacteria and viruses directly, but rather by forming hypochlorous acid, as shown in the following reaction:

$$Cl_2 + H_2O \rightleftarrows HOCl + H^+ + Cl^- \qquad \text{(hydrolysis)}$$
$$HOCl \longrightarrow H^+ + OCl^- \qquad \text{(ionization)}$$

When chlorine is added to water for disinfection, it also reacts with any organic and inorganic materials that might be present. Such reactions complicate the disinfection process because the chlorine demand of these materials must be satisfied in addition to demand associated with the disinfection reactions. Factors which affect chlorine reactions include (65):

a) pH. Chlorine compounds are most effective in bacteria and virus destruction at low pH. The pH of an effluent may be lowered, but at low pH values, the liquid effluent is corrosive and toxic, and must be raised above 7.5 before effluent discharge.

b) Temperature. Reaction rates in the chlorine contact chamber fluctuate with temperature (increase with increased temperature and decrease at low temperatures). Longer detention times may be required in cold weather to achieve the desired level of bacterial kill.

c) Turbidity. The presence of solids has a negative influence on the efficiency of chlorination of wastewater. Bacteria can be concealed within turbidity particles and are then immune to the chlorine. For this reason, chlorine should be added after solids removal (e.g., after secondary clarifiers) in the treatment system.

d) Ammonia. Chlorine, when added to treatment plant effluent, will react with ammonia according to the following equations to produce monochloramine and dichloramine, respectively:

$$NH_3 + HOCl \longrightarrow NH_2Cl + H_2O$$
$$NH_2Cl + HOCl \longrightarrow NHCl_2 + H_2O$$

Chemical combinations with ammonia or other nitrogenous compounds decrease the bactericidal effect of chlorine. The amount of chlorine applied must be increased to satisfy the chlorine demand of nitrogenous compounds if the required disinfection is to occur during a specified contact time in effluent containing ammonia.

e) Organics. Chlorine will also combine with compounds other than ammonia in a treatment plant effluent. For example, phenolic compounds, protein, amino acids and other compounds will react with chlorine, exerting a chlorine demand and making the addition of higher levels of chlorine necessary for disinfection.

f) Reducing substances. Inorganic substances such as sulphides, sulphites, ferrous and manganese ions react with chlorine and exert a chlorine demand. Again, higher levels of chlorine must be added to achieve the desired level of disinfection when these substances are present in treatment plant effluent.

4.7.2 **Chlorine Demand Measurement.** The difference between the amount of chlorine added to wastewater and the amount of chlorine residual (combined and/or free available chlorine) remaining at the end of a specified contact period is the chlorine demand. The chlorine demand for any given water varies with the amount of chlorine applied, the desired residual, the time of contact, the temperature, pH and the amount of chemical and organic contaminants in the wastewater. Test measurements of chlorine demand should be conducted with a chlorine solution, or with hypochlorites, depending upon the form that will be used in practice. Chlorine demand can be readily measured by treating a series of samples of the effluent in question with varying dosages of chlorine or hypochlorite. The effluent samples should be at a temperature within the range of interest and, after a specified contact period, determination of residual chlorine in the samples will demonstrate which dosage has satisfied the requirements of the chlorine demand in terms of the desired residual. Both the contact time and required chlorine residual will be specified by regulatory agencies.

Chlorine feeder capacities are normally rated in terms of kg/d (lb/day) of chlorine required, and may be calculated as follows:

Chlorine required (kg/d) = Dosage (mg/L) x Maximum Plant Flow (mgd) x 4.54

or

Chlorine required (lb/day) = Dosage (mg/L) x Maximum Plant Flow (mgd) x 10

Typical chlorine requirements for treated wastewaters are shown in Table 23. The chlorine requirement will vary according to the quality of discharged effluent and the required contact times and chlorine residuals, as specified by regulatory agencies.

TABLE 23 TYPICAL CHLORINE DOSAGES FOR DISINFECTION (65)

Type of Wastewater	Chlorine Dosage Range (mg/L)
Primary effluent	5 - 20
Secondary effluent	2 - 8
Chemical precipitation effluent	2 - 6
Trickling filter effluent	3 - 15
Multi-media filter effluent, following activated sludge plant	1 - 5

4.7.3 Chlorine Compounds. The chlorine compounds in most common use at wastewater treatment plants are calcium and sodium hypochlorite, and chlorine gas. Calcium and sodium hypochlorite are usually used in small treatment plants because of the relative safety in handling (65).

Calcium hypochlorite is available commercially in either a dry or wet form with high-test calcium hypochlorite containing at least 70 percent available chlorine. In dry form, calcium hypochlorite is available as a powder, granules, compressed tablets, or pellets (granular and pellet form preferred) which can be mixed with water to a concentration 5 kg of chlorine per 50 L of solution. Because of its oxidizing potential, calcium hypochlorite should be stored in a cool, dry location away from other chemicals, in corrosion-resistant containers.

Sodium hypochlorite solution is available in strengths from 1.5 to 15 percent available chlorine. The solution decomposes readily at high concentrations and is affected by exposure to light and heat. It must, therefore, be stored in a cool place in a corrosion-resistant tank. Where sodium hypochlorite is available at a reasonable cost, its use should certainly be investigated by the design engineer.

Chlorine may also be supplied to treatment plants as a liquified gas in high pressure containers varying in size from 68-kg (150-lb) cylinders to tonne containers. In general, plants using 136 kg (300 lb) of chlorine per day need 68-kg containers; plants using up to 900 kg per day would use tonne containers. Local availability of chlorine and the policy of the supplier may also govern the choice of container size.

4.7.4 Chlorination Apparatus

Pellet feed chlorinator. The pellet feed chlorinator consists basically of a rectangular box fitted with an intake pipe at one end (up to 30 cm or 12 inch diameter) and an adjustable weir plate at the outlet. The entire plant flow of treated wastewater passes into the chlorinator through the inlet adaptor, or pipe. As the stream of water flows past the feed tubes containing the chlorine tablets, active chlorine is released into the wastewater by the dissolving action of the water in contact with the tablets. A weir at the outlet end of the device (selected to match plant capacity) controls the water level in the chlorinator, which actually controls the chlorine concentration in the water. As the incoming water flow rate increases, the water level in the chlorinator rises, immersing a greater number of tablets. When the incoming flow rate decreases, the water level in the chlorinator drops, exposing fewer tablets to the water. From the chlorinator, the chlorinated wastewater flows into the chlorine contact tank where it is held for the specified time to permit effective bacteria killing action. Although each chlorinator is furnished complete with four feed tubes and a selection of weirs, the actual number of tubes to be filled with tablets, and the weir to be used in the process are determined by the average daily flow rate through the plant and the required residual chlorine content (66).

The tablets, which are sold under the brand name Sanuril 115, are a combination of calcium hypochlorite and 1,3,4,6-tetrachloroglycoluril, a patented compound. The chlorinator itself comes in three basic models which are capable treating up to 227 m^3/d (50,000 gpd). Figure 53 illustrates the largest model available. Little long-term operating information is available on the unit at this time. Perhaps the major concern about the pellet feed chlorinator is the ability, or inability, to control chlorine residual. Evaluation tests conducted by Ontario Ministry of the Environment revealed that the Sanuril 155 tablets had a tendency to dissolve very rapidly and release excess amounts of chlorine (67). Because of this, the number of days of service yielded by the chlorinator between the refillings of the feed tubes was only a small fraction of the time

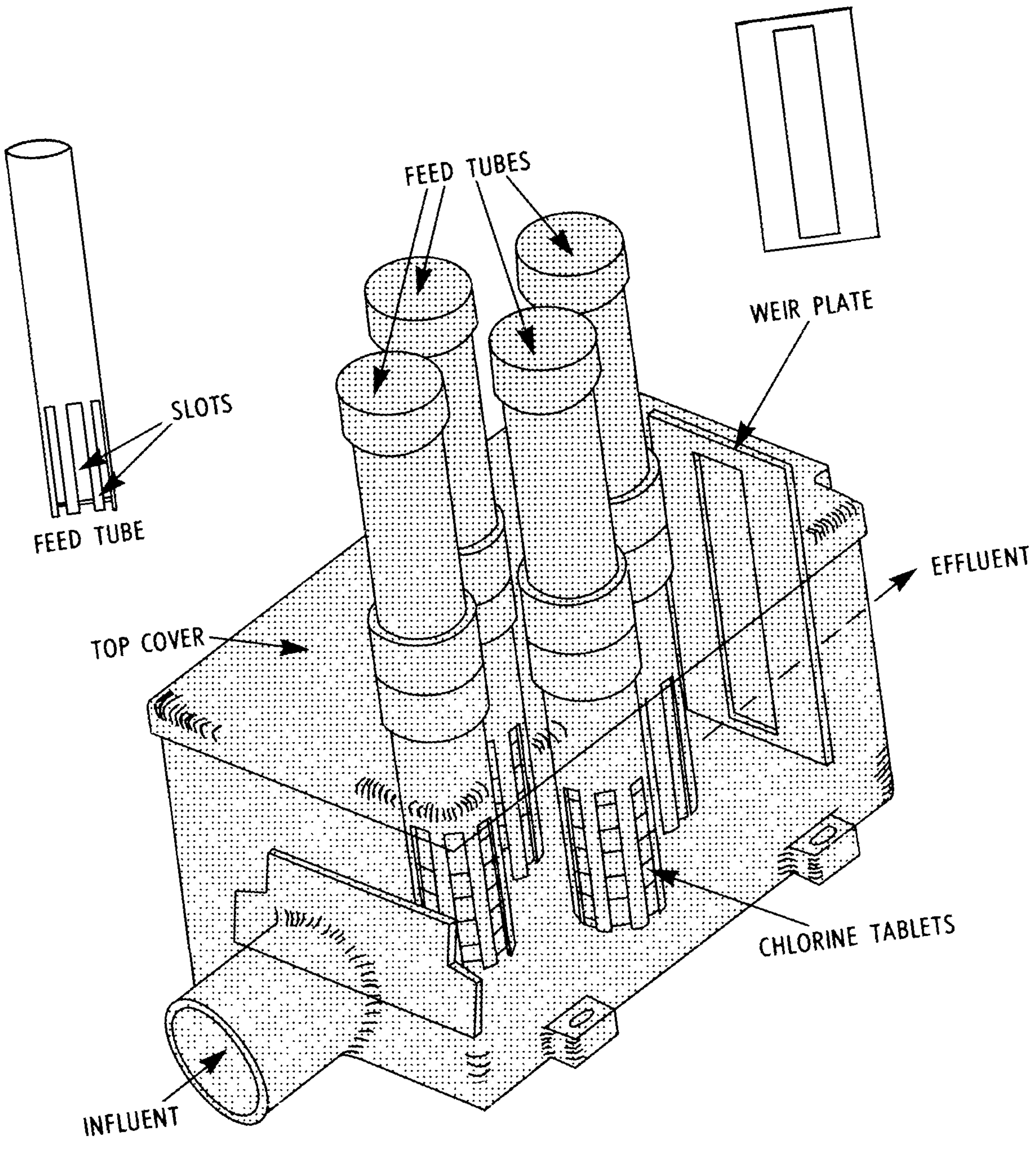


FIGURE 53 PELLET FEED CHLORINATOR (From Diamond Shamrock Corp.)

suggested by the manufacturer. Thus, the system would require more frequent maintenance than expected (i.e., the unit should be checked once a day and tablets replenished as necessary).

Hypochlorite solution feeders. The most satisfactory means of feeding calcium or sodium hypochlorite solutions is through chemical metering pumps. These units are available in capacities up to 22.7 m^3/d (5000 gpd), with adjustable stroke, variable speed drive and/or adjustable V-belts for varying feed rates. Control of feed rates to provide a chlorinated effluent with the required residual under varying flow conditions may be accomplished manually or automatically by a control device, as discussed in section 4.7.5.

The disinfection process, utilizing hypochlorite feeders, involves preparing a dilute aqueous solution in a storage tank and injecting the appropriate dosages into plant effluent. This technique requires daily preparation of chlorine solution and regular maintenance of the feeder to ensure proper disinfection of the sewage effluent. Mechanical mixing devices, plastic or ceramic storage tanks, and piping and valving are required in addition to the metering pumps.

Chlorine gas feeders. Gas chlorination should only be used as a method of disinfection in facilities operated by full-time trained operators. The operator using chlorine gas must be adequately informed on safety measures and operational procedures. This is best achieved through training courses such as the Ontario Ministry of the Environment's Gas Chlorination Workshop (68).

There are two methods of chlorine gas feeding: direct and solution feed. The direct feed chlorinator can supply up to 136 kg (300 lb) of chlorine gas per day directly to the treated wastewater. The chlorine cylinder pressure alone operates the chlorinator, as illustrated in Figure 54. This type of equipment is not recommended except under conditions which prevent the use of solution-feed chlorinators. A solution-feed apparatus meters chlorine gas under vacuum and dissolves it in a small amount of water, or treated effluent, to form a concentrated solution which is then applied to the treated effluent. At 20°C, one volume of water will dissolve two to three volumes of chlorine gas, or about 7000 mg/L. Figure 55 illustrates a solution-feed chlorinating device.

The range of chlorine feed rates available in gas chlorinators depends on the type of metering elements used, namely orifice and rotor control meters (65). The maximum 24-hour capacity of these meters is usually specified by the manufacturer and, in general, the range available varies from 0.7 to 3600 kg per 24 hour-period. Gas

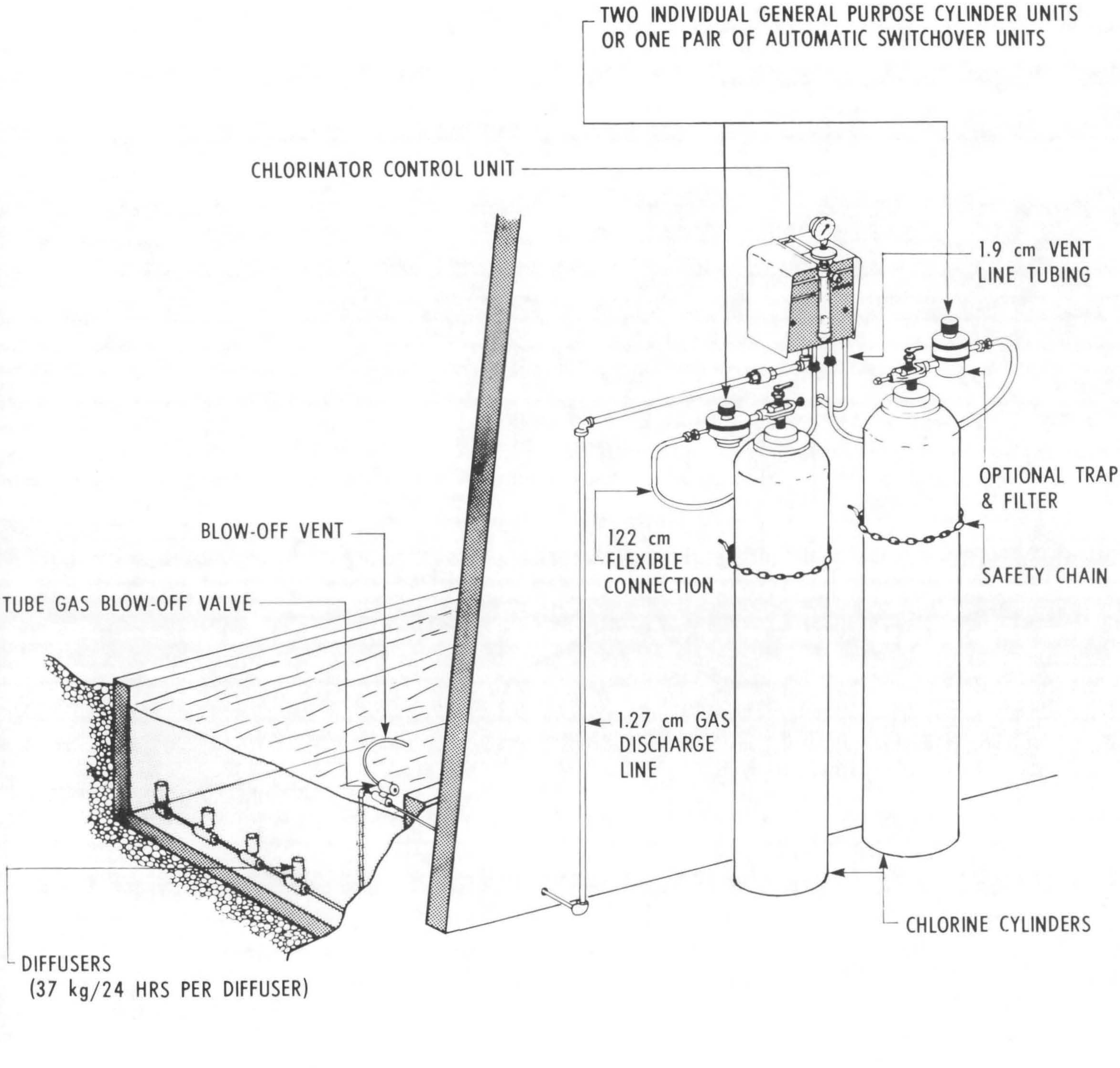


FIGURE 54 DIRECT FEED CHLORINATION SYSTEM - LOW CAPACITY 1-45 kg/d

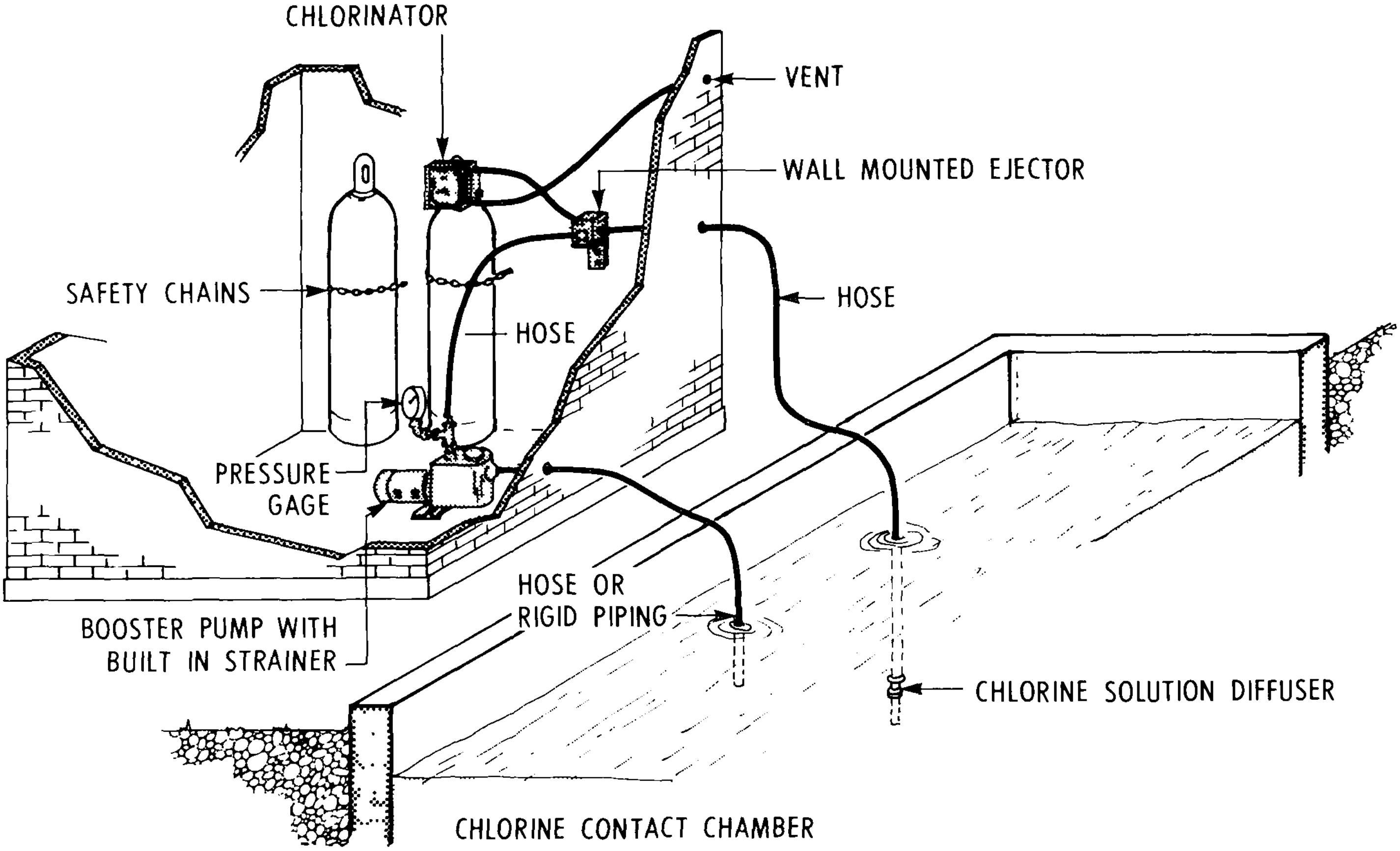


FIGURE 55 SOLUTION FEED CHLORINATION SYSTEM

chlorinators may be applied where the minimum wastewater flow is approximately 22.7 m^3/d.

Economic and safety considerations should be investigated in all instances where gas chlorination is being contemplated for small wastewater treatment systems. The many safety devices and handling precautions that must be designed into chlorine handling facilities are too numerous to mention in this document; designers are referred to local regulatory agencies and equipment suppliers for complete details. However, some fundamental considerations include:

a) Safety requires that the chlorine equipment and storage room be sealed off from the rest of the control building and, when possible, a fixed glass viewing window be installed on an inside wall for observation of the chlorine equipment. Separation of the chlorinator room and the chlorine container room is considered good practice. Access to the chlorine room should be through an exterior door opening out and equipped with panic hardware.
b) Ventilation is required for all chlorine equipment rooms. The ventilation intake or the exhaust fan must be located at the floor level. An exhaust fan with guard and shutters is ordinarily mounted at floor level on an exterior wall and operated intermittently as required. An alternate method consists of an exterior fan with intake duct run to the chlorine room floor. Good practice requires an air change every three minutes. Vent fan control should be located outside the chlorine room.
c) A self-contained breathing apparatus should be mounted outside the chlorine room for protection against chlorine gas leaks.
d) The chlorine equipment room must be heated to maintain a minimum temperature of 21°C. The chlorinator and auxiliary water supply should be maintained at a temperature above 10°C.
e) In small installations feeding less than 91 kg (200 lb) of chlorine per day, a minimum of 6 m^2 of floor area is considered adequate.
f) Gas cylinders in use should be set on platform scales, flush with the floor, and the loss of weight used as a positive record of chlorine dosage.

4.7.5 Control Systems. Four basic chlorine control systems are available (68):

- manual control,
- flow proportional or open loop control,
- direct residual or closed loop control,

- compound loop control.

These systems are used mainly in the operation of hypochlorinators and gas chlorinators. As noted in Section 4.7.4, chlorine feed rates with pellet feed chlorination are established through operational experience. Flow proportional automatic control is usually used at small installations for reasons of cost and simplicity of operation.

Manual control consists of stopping and starting the chlorinator by hand and/or adjusting the chlorine feed rate manually as required to maintain the specified chlorine residual in the contact tank. This method is perhaps most commonly used with hypochlorinators, and less often with gas chlorination systems.

In the flow proportional or open loop control system, the chlorine feed rate is adjusted in accordance with a command signal from a flow meter or pump starter. The chlorinator may be automatic start and stop, or manual start and stop. The dosage rate is manually set and the control device varies the rate in relation to volume of flow. This method of control permits maintenance of the desired chlorine residual under conditions of varying flow; however, variations in chlorine demand due to varying sewage strength are not taken into account. The feed rate of a hypochlorinator is controlled automatically by a variable-speed drive, or electric or pneumatic stroke-length positioner. A vacuum or water supply controller is used to vary the rate of chlorine feed from gas chlorination devices.

The closed-loop control system operates on the principle of feedback of chlorine residual information to the chlorinator control for comparison with a control set point. The operation consists of the following steps:

a) Continuous samples are withdrawn downstream from the point of chlorination and analyzed for chlorine residual.
b) A recorder compares the measured residual with the desired residual and determines whether it is necessary to increase or decrease the chlorine feed rate.

The compound loop control system is a combination of an open loop and a closed loop system. When flow increases, the chlorinator adds the mathematically correct amount of chlorine to maintain the dosage rate. Downstream a sample is withdrawn and analyzed to determine whether chlorine demand has changed. If so, information is relayed back to the chlorinator and the dosage rate is corrected. When flow remains constant, but chlorine demand does not, closed loop control is applied. Conversely, when flow varies and chlorine demand remains constant, flow proportional or open loop chlorination control is used. When both flow and chlorine requirements vary, compound loop chlorination control is used to maintain the desired residual of chlorine.

4.7.6 **Mixing.** The objective of the chlorine mixing process is to provide rapid intimate mixing of the chlorine solution with the wastewater stream. Ideally, the mixing mechanism should be able to homogenize the chlorine solution and the treated effluent in a fraction of a second. The mixing process practiced is usually a function of the treatment system layout. As illustrated in Figure 56 mixing systems include in-line, mechanical, hydraulic jump and baffling mixing. Each of these alternatives provides a turbulent treated effluent into which the chlorine solution is injected. In-line and the hydraulic jump methods of mixing are most commonly employed in small systems. An interesting and relatively new mixing unit under observation by pollution control agencies in Canada and United States is the jet disinfection system. The basic components of the system are a pump, a jet nozzle assembly, and a fibreglass reactor tube mounted on a baffle. The system is manufactured commercially and is available as a "package" installation.

4.7.7 **Chlorine Contact Chambers.** Chlorine contact chambers are used in wastewater treatment to ensure adequate contact between the chlorine solution and the wastewater prior to discharge. They are located immediately after the mixing chamber in the treatment plant flow sequence and, in the past, have consisted of anything from circular or rectangular tanks to long outfall pipelines. There is currently no commonly accepted criteria for the design of chlorine contact tanks but guidelines are beginning to emerge. A suggested approach to designing a chlorine contact tank includes the following steps:

a) Determine the contact time necessary to obtain the required level of disinfection. This contact period is normally specified by regulatory agencies. A value of 60 minutes at average flow rate, or 30 minutes minimum contact period at maximum flow rate is suggested.

b) Determine the required volume of tank using the formula

$$T = V/Q$$

where: T = the theoretical detention time (minutes),
V = volume of tank (gallons),
Q = average flow rate (gpm).

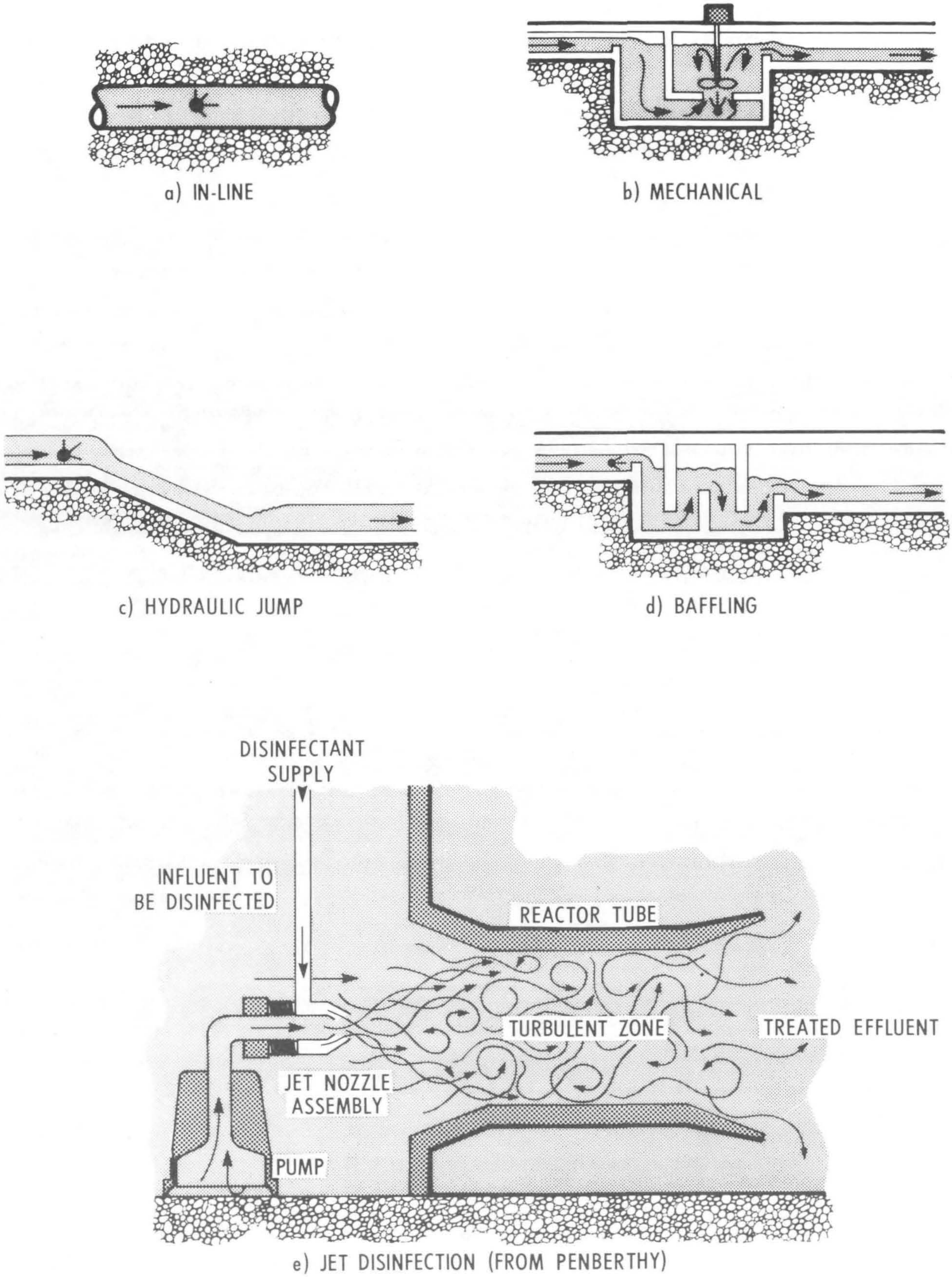


FIGURE 56 MIXING DEVICES

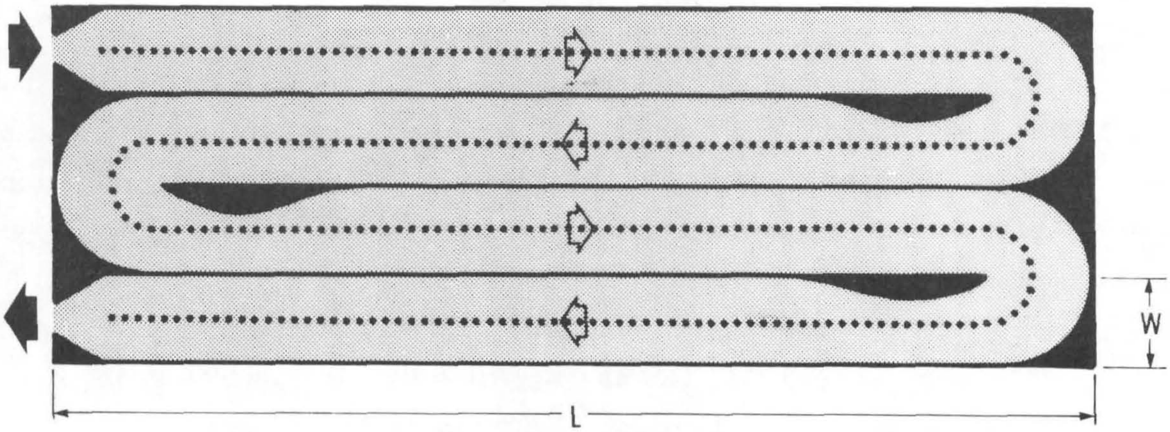
c) A chlorine contact tank should simulate plug flow conditions. This is best accomplished in a rectangular basin, complete with longitudinal baffling. The most significant factor in the configuration of the chamber is the length-to-width ratio, where length is defined as the total length of passes of the tank, and width as the width of a single channel of the tank. The minimum length-to-width ratio of 40:1 is necessary to obtain optimum plug flow conditions. Longitudinal baffling has an advantage over other forms of baffling in that the number of 180° turns is reduced, thus reducing non-uniform flow and stagnation, while optimizing length-to-width ratios.

d) Baffle spacing is determined by the average velocity at design flow 2.44 m/min or 8 ft/min average velocity.

e) The width of the turn path should be equal to one-half to one-third the spacing between the baffles.

Figure 57 shows the layout of a rectangular chlorine contact chamber.

FLOW LENGTH:WIDTH RATIO 40:1



L:W RATIO = 10:1

FIGURE 57 CHLORINE CONTACT CHAMBER WITH LONGITUDINAL BAFFLING

4.7.8 Monitoring Requirements. In-plant control of the disinfection process entails the determination and fulfillment of the chlorine requirement of the treated effluent. Chlorine requirement may be defined as the amount of chlorine which must be added per unit volume of treated effluent to produce a desired result under stated conditions. The result may be based on any number of criteria, such as a stipulated coliform density, a specified residual chlorine concentration, or others. In each instance a definite chlorine dosage will be necessary.

In most instances, the desired result is a specified chlorine residual, as required by regulatory agencies. Such residual may be determined manually by either the iodometric method as described in Standard Methods for the Examination of Water and Wastewater, APHA, AWWA, WPCF, 14th edition, 1975, or for more accurate measurement at low residual levels, by an amperometric titrator. Accurate manual control of chlorine residual may be impossible due to the great variations in flow and strength of sewage.

In small plants, chlorine residual should be checked and recorded at least each day when the maximum flow enters the plant. This normally ensures a sufficient chlorine dosage during the rest of the day. It may also be advisable to readjust chlorine feed rates during the night when the flow and chlorine requirement is much lighter. Automatic dosage control facilities, using open-loop, closed-loop, or compound-loop control systems, will have to be calibrated occasionally to ensure fulfillment of chlorine requirements throughout varying flow conditions. This requires little time.

It should be understood that the chlorine residual is not an absolute value, but rather a practical and realistic approach to the control of sewage disinfection (and to estimating the chlorine required). Bacteriological tests, usually conducted by regulatory agencies as part of a surveillance program, are undertaken to ensure that proper disinfection of the plant effluent is being performed.

The operator of a plant should record all chlorine residual measurements and the amount of chlorine used each day. Not only does this give the operator a comparison of the dosages and residuals, but also provides evidence to regulatory agencies that proper disinfection is being performed continuously by the operating staff.

4.7.9 Summary. The most popular method of disinfecting treated effluents from small sewage treatment systems is chlorination. Chlorination is achieved through the use of calcium or sodium hypochlorite or gaseous chlorine.

Two methods of disinfection using a hypochlorite compound are available. A relatively new type of chlorination system is designed to dispense chlorine from specially formulated sanitizing tablets. Some of the attractive features of the system are that it is simple, adjustable and operates continuously with minimal attention. It has no moving parts, and needs no power, mixing devices, solution tanks or pumps. Tests conducted reveal that the unit is capable of attaining levels of disinfection comparable to disinfection with calcium and sodium hypochlorite solutions (67). Major disadvantages of the unit are the apparent lack of operational control over chlorine residual and the relatively high cost of the chlorine tablets. Hypochlorite solution disinfection systems are perhaps the most common method of chlorination in small wastewater treatment systems today. Whereas the initial capital cost of the chemical metering pumps and ancillary equipment is higher than the pellet feed chlorination system, greater operational control and lower cost of available chlorine are advantages of hypochlorite solution chlorination systems.

Gaseous chlorine is rarely used in the disinfection of effluents from small wastewater treatment systems because of the high capital costs involved in providing the feeding equipment and other auxiliary facilities needed to ensure safe handling of this toxic gas. Gas chlorinators must be installed in a separate room equipped with an exhaust fan.

Four basic control systems are available for chlorine feed systems. Of these, manual and open-loop control systems appear to be the most viable considerations for small wastewater treatment systems.

Table 24 summarizes the costs of various alternatives in chlorine feed equipment.

4.8 Dechlorination

Concern currently exists regarding the effects of chlorine and chloramines upon the aquatic environment. In some instances, as defined by applicable regulatory agencies, it may be necessary to design a dechlorination process into a wastewater treatment system. Dechlorination processes applicable in small wastewater treatment systems include holding ponds and the addition of sulphur compounds.

4.8.1 Holding Ponds. The rate of chlorine destruction in holding ponds is not well-documented. It has been observed that free chlorine in a secondary effluent exposed to

TABLE 24 COST SUMMARY FOR CHLORINATION EQUIPMENT*

Chlorine Feed Method	Plant Flow (m^3/d)	Capital Cost**	Chlorine Cost***
Hypochlorination			
i) Pellet feed	6.8 45 227	$ 120 190 340	$5.38/kg
ii) Solution feed (including one meter pump, two solution tanks, one mechanical mixer)	22.7 454	1400 3000	3.31/kg
Gas Chlorination (chlorinator only)	up to 454	1800	0.88/kg

* Manufacturers' costs, 1976.
** Chlorination feed equipment only, unless specified otherwise.
*** Electrical and manpower costs to be considered as well in overall O&M.

bright sunlight decayed from 2.0 to 0.2 ppm in approximately 0.5 hours. Monochloramine, however, decays far more slowly.

A holding basin for the purpose of dechlorination should be shallow, about 1.2 to 1.8 m deep, to allow sunlight to penetrate the entire liquid, and sized for 24-hour retention to obtain a chlorine residual of about 0.1 mg/L. Short-circuiting of the chlorinated liquid should be prevented by baffles at the influent and effluent outlets to the cell (69).

A major disadvantage of this method of dechlorination is the space required for the construction of the shallow pond.

4.8.2 **Sulphur Compounds.** The addition of sulphur dioxide is probably the method most used for dechlorination in a larger treatment systems. Its common use is at least partially a result of its similarity to gaseous chlorine in its supply, dissolving, and feeding characteristics. It is able to remove both free chlorine and chloramines in a nearly instantaneous reaction (69).

Equipment used to meter sulphur dioxide addition is identical in all respects to that for metering chlorine. Sodium sulphite, sodium bisulphite or sodium metasulphite solution addition is controlled by a metering pump (similar to hypochlorite feed). Sulphur dioxide may be added to chlorinated effluents using gaseous flow controllers as discussed for chlorine gas. Contact time in this process is not important, but good turbulent mixing between the dechlorinating solution and the chlorinated effluent must be ensured (70). For small plants, a mechanically-mixed tank with a residence time of two to five minutes should be adequate. Dosage control of the dechlorinating agent can be effected automatically by a chlorine residual analyzer which relays the information to a control panel, activating the dechlorination process, or manually by determining the chlorine residual and injecting proportionate amounts of sulphur dioxide until the desired residual level is achieved. Table 25 summarizes the chemical requirements for dechlorination.

TABLE 25 CHEMICAL REQUIREMENTS FOR DECHLORINATION (70)

Sulphur Compound	Dosage (Part/Part Cl_2)
Sodium bisulphite $NaHSO_3$	1.46
Sodium sulphite Na_2SO_3	1.77
SO_2 (gas)	1 (in practice)

Sulphur dioxide addition for dechlorination reduces alkalinity and will reduce the pH of low-alkalinity wastewater. Table 26 shows the loss of alkalinity with the various additives and also identifies chemical additions necessary for alkalinity restoration. Another major consideration in the use of sulphur dioxide and other sulphur compounds is that they also react with dissolved oxygen. Control of the quantity of chemical added is very important because excess dosage will remove oxygen from the effluent and possibly the receiving water.

TABLE 26 ALKALINITY LOSS AND RESTORATION BY CHEMICAL ADDITION (70)

Chemical Added		Alkalinity Loss or Restoration (as $CaCo_3$)
1)	Alkalinity Loss (as $CaCO_3$)	
	Chlorine	1.41 mg/mg of chlorine
	Sulphur dioxide	2.8 mg/mg of chlorine removed
	Sodium bisulphite	1.38 mg/mg of chlorine removed
	Sodium sulfite	1.38 mg/mg of chlorine removed
2)	Alkalinity Restoration (as $CaCO_3$)	
	Lime	0.74 mg/mg of $Ca(OH)_2$
	Lime	0.54 mg/mg of CaO
	Caustic	0.8 mg/mg of NaOH
	Soda Ash	1.06 mg/mg of Na_2CO_3

5 EFFLUENT DISPOSAL ALTERNATIVES

The disposal of an effluent from a small wastewater treatment system will normally involve one of three disposal methods: i) subsurface land disposal; ii) discharge to a surface body of water; iii) surface land disposal. The first of these three items, subsurface land disposal, has been discussed extensively in Section 3. The following section outlines pertinent information pertaining to the disposal of wastewater effluent via surface water or land discharges. Because of the many different design criteria and regulatory agency requirements associated with this topic, it is suggested that more detailed information be obtained from local agencies.

5.1 Effluent Discharge to Surface Water Bodies

Effluent quality requirements for a wastewater treatment installation discharging to a surface water body will be given to the design engineer by the regulatory authority having jurisdiction in the area concerned. A combination of effluent standards, guidelines and/or codes of good practice are used by these regulatory agencies to clean up and prevent pollution to a receiving environment, with emphasis normally placed on practical solutions based on available pollution control technology. General information on outfall design and location is contained herein as background material.

Shoreline release of treatment plant effluents is rarely a satisfactory method of disposal and, in most instances, is not permitted by regulatory agencies because of poor dispersal and mixing characteristics in receiving waters. To avoid situations where the zone of influence of a discharge could extend miles downstream along a shoreline, many regulatory agencies require the installation of an engineered outfall.

The design, location and operation of an outfall should be established on a case-by-case basis, in consultation with local regulations offices, and should be based on (71):

a) the physical, chemical and biological characteristics of the body of water;
b) the present and future uses of the body of water;
c) the quality of the wastewater effluent being discharged.

Outfalls in freshwater streams should extend below the lowest anticipated water level, and designed to make use of the available dilution. Outfall orientation affects the dispersal of wastewater in streams. Because sewage is normally warmer than receiving waters, it tends to rise to the surface layers of a stream. Obviously, countercurrent discharge will produce better dispersion than cocurrent discharge, provided the energy of waste discharge is great enough to force the plume far enough upstream (71).

The release of wastewater into lakes creates special problems. Wind and thermal currents establish complex flow patterns in lakes, and water masses tend to acquire a vertical and horizontal balance which may be upset by external forces. Onshore and offshore winds, prevailing currents, seasonal stratification, ice cover, and stagnation are all variables that can affect the behaviour of a wastewater effluent discharging into a lake. Detailed hydraulic and hydrographic studies are required to avoid potential problems (71).

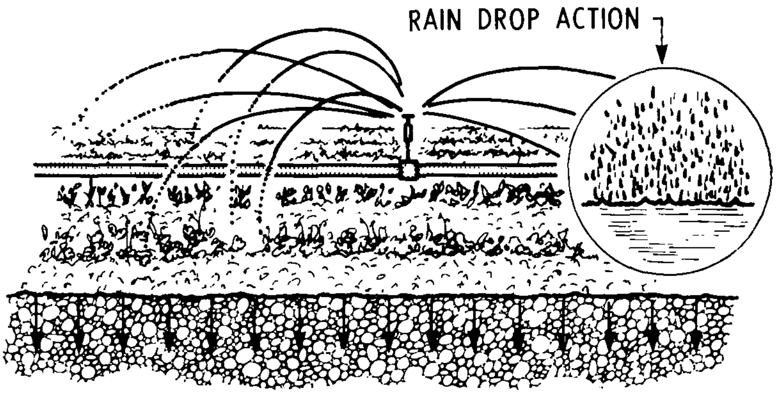
Marine outfalls are designed to provide optimum protection of the beneficial uses of the receiving waters (e.g., recreation, shellfish harvesting, etc.). Like lake waters, coastal waters are colder and heavier than wastewater effluents. Discharged at some depth below the surface, effluents rise in columns and, on reaching the surface, fan out radially. Mixing of the two waters is mainly a function of wind, currents and tides. Dispersion and dilution are increased by subsurface horizontal discharge and multiple outlet ports. Simple circular ports appear to be adequate for small wastewater discharges (71).

Outfalls should be constructed so as to be protected against the effects of floodwater, tides or other hazards. Navigational hazards must also be considered in designing outfall sewers. A manhole should be provided at the shore end of all gravity sewers extending into receiving waters.

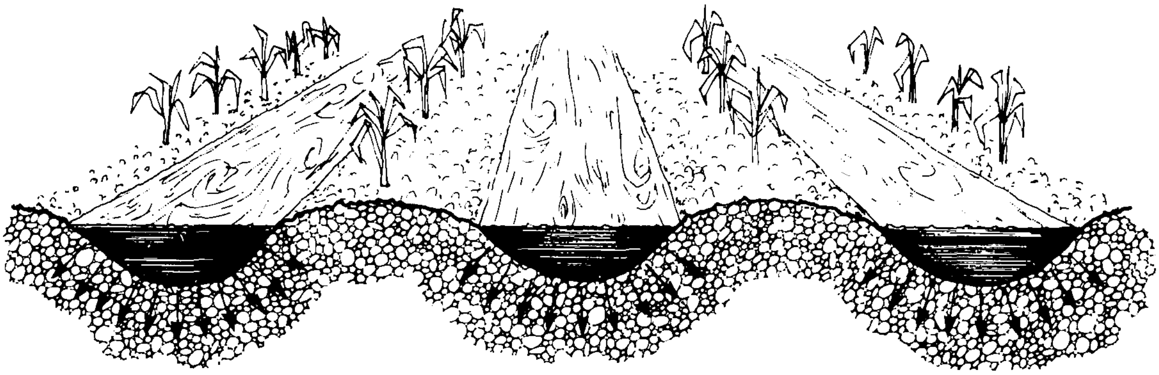
5.2 Surface Land Disposal of Effluents

Surface land disposal may be a feasible alternative for disposing of secondary effluent from domestic waste sources where direct release to a receiving body of water is not possible or desirable. This method of disposal provides additional effluent polishing through contact with soil and cover crop. Land disposal for domestic wastewater most commonly follows facultative or aerated lagoon treatment, but can be used for disposal of other secondary effluents. Proper design and operation of a land disposal system is important to avoid problems such as surface ponding and groundwater pollution. The following section provides information on acceptable design and operational criteria. Local regulatory agencies will provide detailed information.

Figure 58 illustrates two basic methods of applying treated effluent onto land, i.e., spray irrigation, and ridge and furrow irrigation. Figure 59 further illustrates two

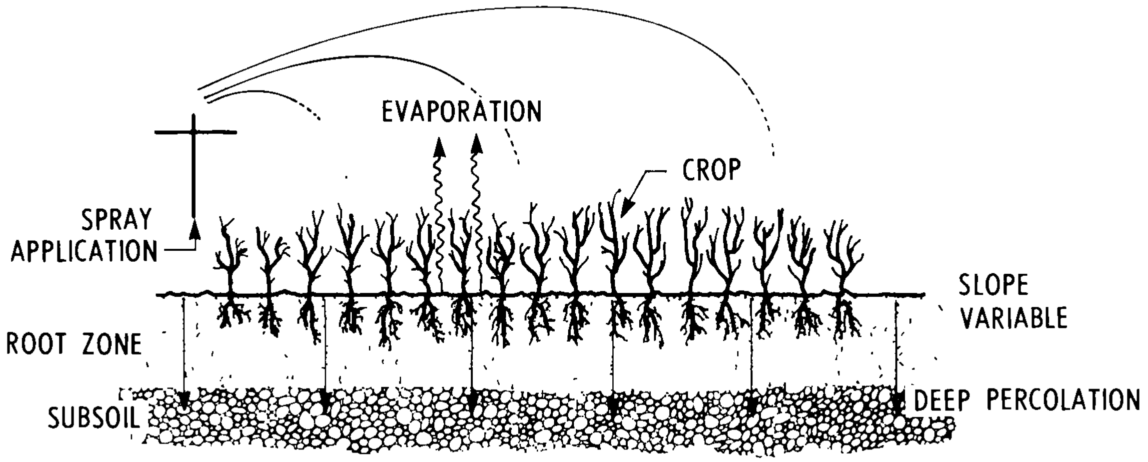


SPRAY IRRIGATION

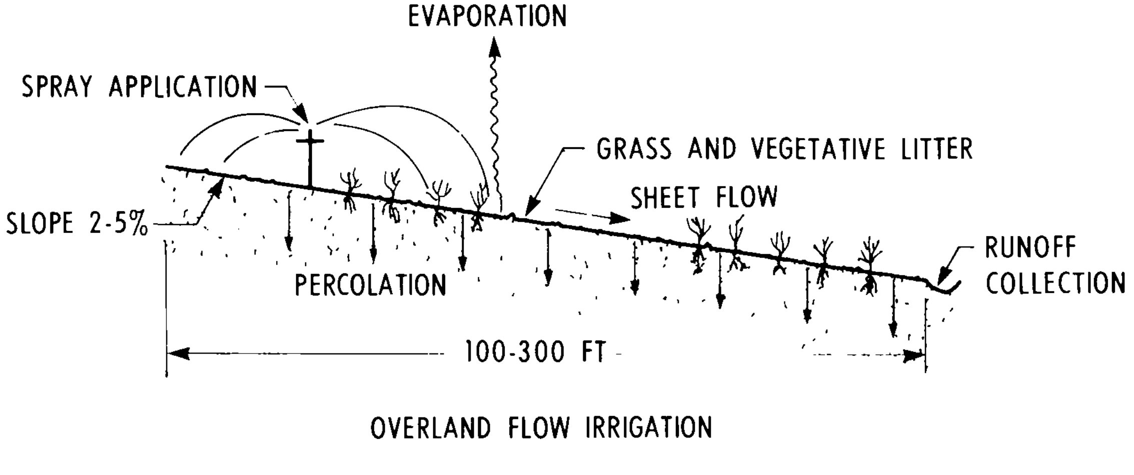


RIDGE AND FURROW

FIGURE 58 IRRIGATION TECHNIQUES



STANDARD SPRAY IRRIGATION



OVERLAND FLOW IRRIGATION

FIGURE 59 ALTERNATE METHODS OF SPRAY IRRIGATION spray irrigation approaches. The first method involves the application of a liquid to a soil surface at a specified rate such that the entire loading is disposed of via evaporation or percolation into the soil (standard spray irrigation). An alternative to this standard method involves the application of a liquid at a rate in excess of the evaporative and percolative properties of the site. This method results in liquid runoff (overland flow), which is collected and returned to the irrigation system or discharged to a receiving stream (72,73,74,75,76). Of the three alternatives, standard spray irrigation is the most commonly used for land application of domestic secondary effluent in Canada.

At present, spray irrigation sites are approved by provincial authorities on a site-specific basis. In some provinces, guidelines for such systems have been recently prepared or are currently being prepared. These should be consulted when designing a system. Due to potential health hazards, raw domestic wastewater should not be sprayed onto land, and it is generally preferred that only properly disinfected secondary effluent be sprayed. Excessive loadings which may cause runoff, ponding and odours, as well as winter application which can result in equipment problems, destruction of vegetative covers, inadequate waste treatment, etc., should be avoided.

The loading intensity of a spray irrigation system should not exceed the infiltrative capacity of the soil. Loading intensity is defined here as the instantaneous application rate of treated effluent onto the soil surface. This design parameter, usually specified in millimetres per hour, is a function of the characteristics of the irrigation site, including soil type, ground slope, crop cover and effluent quality. The loading intensity frequently ranges between 4 and 6.4 mm/h (0.16-0.25 inches/hr), and may occasionally reach 12.7 mm/h under optimum conditions.

The hydraulic loading rate is a design parameter which is a function of loading intensity, total spray field area and the duration of the spray season. This parameter, normally expressed in millimetres per hectare per day (in/acre/day) or mm per hectare per week (in/acre/wk), is affected by soil conditions, climate, crop cover and wastewater ponding, and determined by on-site pilot testing. A typical range for hydraulic loading is between 37.7 and 156.9 mm/ha/wk (0.6 and 2.5 in/A/wk) and occasionally, under ideal conditions, up to 251 mm/ha/wk (4 in/A/wk).

Organic loading rates for spray irrigation systems are defined on an individual basis (pilot testing). However, empirical rates have been developed based on experience and are normally in the vicinity of 78 kg BOD/ha/d (25 lb BOD/acre/day). Organic overloads due to spray irrigation can damage or kill vegetation, severely clog the soil surface, and cause leaching of organic material into the groundwater system.

Unless proper irrigation management is practiced, high total dissolved solids (TDS) in wastewater can cause a salinity hazard to crops and build up salt concentrations in groundwater. Local agricultural agencies should be contacted for specific information. Nitrate nitrogen in effluents sprayed on land can leach through the soil and into groundwater, where it is a potential health hazard. Because nitrogen compounds are mostly removed by crop uptake and harvesting, it is advisable to limit the nitrogen loading in a spray operation to the amount that crops can assimilate.

Table 27 shows some typical values of nitrogen uptake for a variety of crops. More accurate values pertaining to Canadian conditions may be obtained through local agricultural representatives.

TABLE 27 TYPICAL VALUES OF CROP UPTAKES OF NITROGEN (73)

Crop	Nitrogen Uptake kg/ha/yr	(lb/A/yr)
Alfalfa	173 - 247	(155 - 220)
Red clover	86 - 141	(77 - 126)
Sweet clover	177	(158)
Coastal Bermuda grass	538 - 673	(480 - 600)
Corn	174	(155)
Cotton	74 - 112	(66 - 100)
Fescue	308	(275)
Milo maize	91	(81)
Reed canary grass	753 - 402	(226 - 359)
Soybeans	105 - 127	(94 - 113)
Wheat	56 - 85	(50 - 76)

Phosphorous applied onto land in a spray irrigation process is removed from the treated effluent by fixation in the soil (adsorption and precipitation) and by crop uptake. Long-term phosphorous loadings are important because the fixation capability of some soils may be limited over the normal lifespan of the system. For this reason, the long-term fixation capacity of a soil should be estimated by a soil chemist or other qualified expert. Exhaustion of the soil-fixation property is most critical for coarse-textured soils with little calcium, iron or aluminum content. Phosphorous that reaches surface waters and/or intercepts groundwater flow may aggravate eutrophication problems.

Exchangeable cations, particularly sodium, calcium, and magnesium ions are of concern as well. High sodium concentrations in clay-bearing soils have the effect of dispersing the soil particles and decreasing the soil permeability. To assess the hazard, a sodium adsorption ratio (SAR) has been developed by the U.S. Department of Agriculture, Agricultural Salinity Laboratory, and is defined as follows:

$$SAR = Na^{+} \Big/ \left[(Ca^{2+} + Mg^{2+})/2\right]^{1/2}$$

where Na^{+}, Ca^{2+}, and Mg^{2+} are concentrations of the respective ions in milli-equivalents per litre of water. High SAR (greater than 9) values may adversely affect the permeability of soils and, as a result, decrease infiltration. Other exchangeable cations, such as ammonium and potassium may also react with soils. Occasionally, high sodium concentration in soil can also be toxic to plants, although the effect on permeability will generally occur first.

Trace materials, such as heavy metals, are retained in the soil matrix through adsorption, ion exchange and precipitation. Although many of these elements are essential for plant growth, they may become toxic to both plants and microorganisms when applied at high loading rates. If preliminary sampling shows organic chemicals or heavy metals present in significant concentrations in the sprayed effluent, potential effects of the discharged waste on the receiving environment should be investigated by a specialist.

Pathogens may be present even in disinfected secondary effluents and, as a consequence, effluent is not normally sprayed onto crops for direct human consumption (e.g., tomatoes, lettuce or root crops such as potatoes, beets, or turnips). Effluent irrigation of pasture and or forage crops used for animal consumption is often permitted where isolation or storage is provided subsequent to effluent application. Further information regarding spraying of various cover crops with secondary effluent should be obtained from local health, agricultural and/or environmental staffs.

In addition to the physical, chemical and biological properties of the wastewater and soil, climatic considerations must also be made. The effect of precipitation on the design of a spray irrigation system is twofold:

a) The average annual rainfall must be considered in determining actual hydraulic loadings on the sprayfield. The area of the sprayfield must be increased to accomodate both the hydraulic loading rate from the spraying operation as well as precipitation.

b) Intense storms of significant duration can cause extensive ponding and temporary shutdown of spray operations. Contingency storage facilities must be provided in the waste treatment system for such situations.

The rate of microbial activity (and BOD reduction) is directly related to temperature; bacterial reaction rates approximately double with every 8°C rise in temperature throughout normal Canadian operating temperature ranges (78). Microbial activity is necessary to maintain percolation properties in the receiving soil. In addition, spray systems should not be operated during below-freezing periods because of the possibility of clogging nozzles, damage to cover crop and equipment, and inadequate waste treatment. Storage facilities (five to seven months) must, therefore, be provided.

Evapotranspiration, which includes evaporation of liquid from soil and plant foliage and transpiration from vegetative growth, accounts for considerable moisture loss from the field. It should, therefore, be included along with precipitation in calculations of design hydraulic loading and land requirements. Evapotranspiration is affected by several variables including hours of sunlight, the crop being irrigated, nature of soil, etc. These variables must be investigated at each site to assess their effects on the design and operational process. Agricultural specialists should be of valuable assistance in the early stages of design.

Winds significantly increase the travel of aerosols during spray operations. Because pathogens may be carried by these aerosols, an adequate buffer zone must be provided around the perimeter of the sprayfield. The width of these buffer strips may be up to 122 m (61 m average), depending upon local conditions and regulatory requirements.

The following factors are important when assessing a spray field for disposal of a treatment plant effluent.

Vegetative cover. There are two basic approaches to growing crops on a spray field, and each has a direct effect on the design of the irrigation system. If the prime intent of irrigation is to dispose of effluent, then the maximum effluent application will be the design criterion, and higher loading rates than those required for plant growth will be used. In such cases, crops of lesser economic value can be used. However, when optimum crop yield is the design criterion, in addition to the requirement to dispose of treated effluent, then the effluent must be applied only as required for optimum crop growth. Standard irrigation design practice is followed in the latter case.

The selection of a suitable crop for a spray field requires consideration of the following factors:

a) The water requirements and tolerance of the crop must be such that it can withstand wetter-than-normal conditions, including a frequently saturated soil.
b) The nutrient requirements, tolerances and removal capabilities of the crop must be adequate for spray irrigation.
c) The sensitivity of the crop to inorganic ions (particularly salts) may be an important factor, depending on the quality of effluent being sprayed.
d) Public health considerations related to the ultimate use of the crop must be examined. For example, crops grown for direct human consumption should not be irrigated with a secondary effluent.
e) Ease of cultivation and harvesting are often important.
f) Numerous crops, even wooded areas, have been successfully irrigated using secondary effluent, but grasses such as canary reed and timothy are often preferred. In such cases crop value is not important and seeding is not required annually. Information regarding optimum cover crops for particular areas should be obtained through local agricultural representatives.

Soil. Permeability and infiltrative capacity are important soil properties with regard to allowable effluent application rates. Permeability is a measure of the ease with which effluent passes through soil, while infiltrative capacity determines rate at which effluent enters the soil. Both are partially dependent on soil texture, suggesting that soil texture is an approximate indicator of allowable hydraulic loading rate, as indicated in Figure 60. Generally, sands permit greatest hydraulic loadings while clays severely limit hydraulic loadings. Since it is difficult to relate permeabilities and infiltrative capacities directly to allowable loading rates, loading rates are best determined by pilot studies conducted at the sprayfield.

Excess sodium ion buildup in the soil, as measured by the SAR level, will destroy soil texture and permeability in soils with significant clay content. Phosphate removal by fixation in the soil is highest in fine-grained soils with plentiful supplies of calcium (for alkaline soils) or aluminium and iron (for acidic soils). Better BOD and pathogen removal is generally provided by fine-grained soils due to longer retention times.

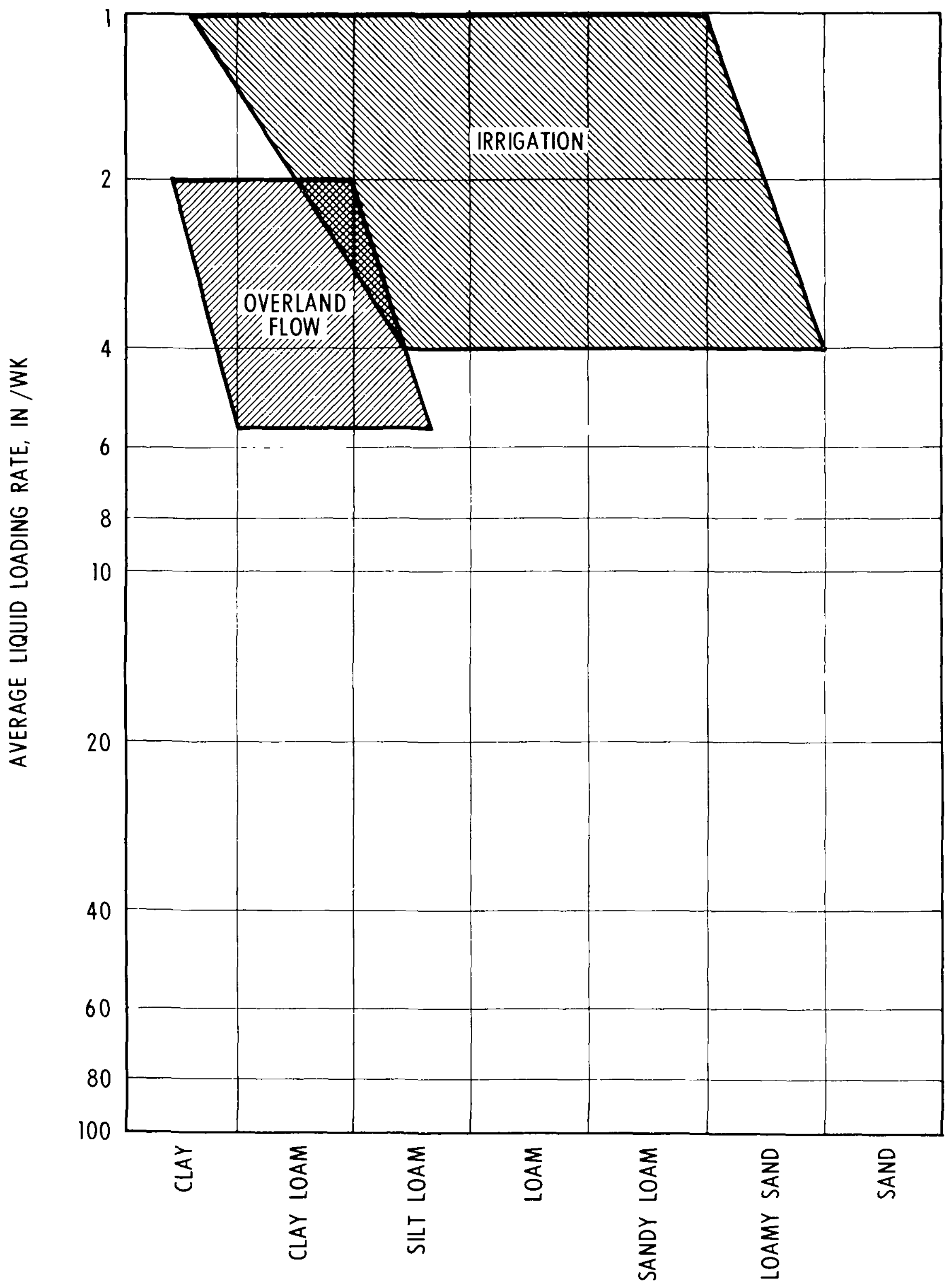


FIGURE 60 SOIL TYPE vs LIQUID LOADING RATES FOR DIFFERENT LAND APPLICATION APPROACHES (75)

(conversion factor: 1 inch = 25.4 millimeters)

Land availability and use. Adequate land area, as determined by loading limitations, must be available at reasonable cost near the existing or proposed pretreatment facilities. Unless provincial guidelines determine the buffer zone, a 65.6 m (200 ft) width is recommended for spraying of disinfected secondary effluent. A minimum distance of 328 m (1000 ft) from residences or other places of habitation is recommended unless special measures are taken to prevent drift.

Topography. To avoid runoff and erosion problems, average slopes should not be excessive. Table 28 illustrates runoff coefficients as functions of topography, vegetation and soil texture. Sites with coefficients up to 0.35 are generally suitable for spray irrigation. Sites with a runoff coefficient between 0.35 and 0.40 may be used for low-rate irrigation effluent. Inconsistencies in the topography, such as depressions or ruts, should be avoided or filled to prevent stagnation or channeling of the effluent.

TABLE 28 VALUES OF RUNOFF COEFFICIENT (72)

Topography & Vegetation	Open Sandy Loam	Clay and Silt Loam	Tight Clay
Rural Woodland			
Flat 0 to 5% slope	0.40	0.30	0.40
Rolling 5 to 10% slope	0.25	0.35	0.50
Hilly 10 to 30% slope	0.30	0.50	0.60
Pasture			
Flat 0 to 5% slope	0.10	0.30	0.40
Rolling 5 to 10% slope	0.16	0.36	0.55
Hilly 10 to 30% slope	0.22	0.42	0.60
Cultivated			
Flat 0 to 5% slope	0.30	0.50	0.60
Rolling 5 to 10% slope	0.40	0.60	0.70
Hilly 10 to 30% slope	0.52	0.72	0.82

Groundwater. A minimum soil depth of 1.97 m (6 ft) to groundwater is required to prevent groundwater pollution. (Groundwater mounding following commencement of spraying should also be considered as a possible route to the pollution of groundwater). Groundwater flow direction and velocity must be determined to assess the effect of groundwater quality impairment. A groundwater specialist should be consulted regarding the general impact of the irrigation system upon the groundwater.

Treatment and storage of effluent. The treatment and storage of an effluent destined for disposal via spray irrigation are vital factors in the successful operation of a spray irrigation system.

Most regulatory agencies require secondary treatment of domestic wastewater prior to disposal by spray irrigation. This level of treatment is desirable to:

- reduce odours;
- improve operational efficiency and reliability of the system by reducing solids content, and thus the possibility of spray nozzle and soil clogging;
- provide an effluent which may be disinfected more effectively.

The provision of an effluent which may be disinfected more effectively is an important requirement for public health reasons. Disinfection aids in the protection of health and hygiene of persons in direct contact with the wastewater and crops, as well as reducing the potential hazard of aerosol contamination. Disinfection of effluent is usually mandatory for irrigation of food crops, parks, golf courses, etc.

Storage is required mainly for:

a) winter effluents when irrigation is not being practiced (five to seven months); and
b) periods when irrigation is not being practiced due to equipment breakdown or saturated field conditions (intense storms).

Storage is commonly provided in the form of a lagoon enclosed by constructed berms. Since provision for winter storage is commonly required in Canadian lagoon systems, it follows that treatment by lagoon is quite appropriate from a cost-effective point of view for a system disposing of secondary effluent by spray irrigation.

Effluent distribution equipment. A fixed sprinkling system basically consists of a main pipeline (or several main pipelines depending on the size of the spray irrigation system) or header feeding several lateral extensions. Evenly spaced riser pipes with sprinkler heads are mounted on top of each lateral. Piping may be laid on the ground surface or buried. Above-ground systems are less costly to install but more likely to be damaged, especially during cultivation and harvesting (if not moved), than buried systems. Aluminium piping is often used for above-ground systems but may have a short working life due to corrosion. Plastic or asbestos cement pipe are most often used for buried systems. Drain valves should be located at various low points in the lines to allow water to drain away and prevent in-line freezing.

Sprinkler spacing may vary from 13 x 20 m (40 x 60 ft) to 33 x 33 m (100 x 100 ft) and be rectangular, square or triangular.

Nozzles vary in size from 0.64 to 2.54 cm (0.25 inch to 1 inch) openings. Nozzle discharge can vary from 15 to 378.5 L/m (4 to 100 gpm), with a range from 30 to 95 L/m (8 to 25 gpm) being typical. Discharge pressures can vary from 207 to 690 kPa (30 to 100 psi).

Risers are usually galvanized pipe of sufficient height to clear the crop being grown (usually 0.98 to 1.31 m for grasses).

Valves are usually installed to control the flow of wastewater to the individual laterals. These valves are usually controlled manually in small irrigation systems, but automatic and semi-automatic control systems are available.

Two types of portable sprinkling systems are also available and in use in Canada. One type resembles the fixed set system except that the irrigation pipes are moved manually from one valve to the next along the mainline. This system requires considerable labour and at least three laterals to provide continuous operation. A second type of portable sprinkler is mounted on wheels and can be moved manually to different areas of the field for spraying as required. Some self-propelled irrigation systems are available but these have not been extensively used for effluent application, with the exception of the centre pivot system which has received limited use (72,73,74,75,76).

System design and installation. The design effluent hydraulic loading can be determined from the relationship:

Precipitation + Effluent loading = Evapotranspiration + Percolation

Estimates of precipitation can be obtained from meteorological records. Evapotranspiration rates for selected cover crop(s) should be obtained from local agricultural representatives. Rates of percolation for a given site (soil) can best be determined by on-site pilot testing. One procedure for pilot testing is described in reference (76). Design percolation, in some cases, may be limited by geological or groundwater conditions at the site.

Generally, hydraulic loading rate governs sprayfield areal requirements for installations spraying secondary effluent. Basically, the area required equals the effluent flow rate divided by the design hydraulic loading. Organic loading and nutrient loadings should also be calculated to ensure they are not excessive. If such is the case, the spray area must be enlarged to reduce the critical loading to within acceptable and safe limits. Additional land area will be required to provide a buffer zone as well as space for storage facilities and future expansion.

Designing the loading intensity permissible at a spray site entails the implementation of a pilot testing program to assess the site capacity.

Alternate application-resting periods are required for successful operation of a spray disposal system. The resting periods must be of sufficient duration to: 1) allow decomposition of organic material, thus enabling the sprayfield to regain its infiltrative capacity; and 2) permit the effluent to drain from the soil. Resting periods normally vary between four and ten days. Application periods (8 to 12 hours are typical), if too long, may result in excessive losses in infiltrative capacity and ponding. The application-resting cycle will vary with soil texture, soil depth, and soil water-holding capacity. The optimum application-resting cycle for a particular system may be determined by the pilot testing (noted above) and some experimentation during initial operation of the system. Adherence to this application schedule may not be possible during periods of heavy rainfall, high winds, sewage treatment plant upset or harvesting of cover crops.

The sprayfield should be divided into sections to provide the flexibility required by the operating schedule. Based on design loading intensity, selection of rated nozzle flow rates and corresponding spray diameters, and appropriate nozzle(s) sizes can be undertaken, along with the determination of sprinkler and lateral arrangements.

The required pumping head can be determined from sprinkler pressure requirements and losses in the system due to friction, elevation variation, etc. Pumping capacity will be determined from the wastewater flows expected.

When installing a spray irrigation system, provincial policy and/or regulations pertaining to spray system location with respect to water supply wells, inhabited dwellings, etc. must be observed. Installation procedures should avoid causing excessive soil erosion or compaction of the spray field absorption area.

System management. Operation of valves on laterals and/or the movement of portable sprinklers will be required, usually on a daily basis, to maintain the specified spray application schedule. Crop cutting, harvesting, and possibly reseeding must also be undertaken with a frequency depending largely on the type of cover crop. In addition, routine pump maintenance will normally be carried out during the spraying season (as specified by manufacturers), including occasional clearing of clogged nozzles and replacement of irrigation system components due to corrosion.

Monitoring requirements may include monitoring of groundwater, soil crops, applied effluent and nearby surface waters. The monitoring necessary at a particular installation may depend on the geology, soils, groundwater hydrology, system size, and other considerations, as determined by the provincial authority responsible for approval of the system. Small systems (i.e., 455 m^3/d or 100,000 gpd) spraying disinfected secondary effluent from mainly domestic sources should not generally require extensive monitoring.

Cost. When land is purchased for use as a spray irrigation site, the capital cost of the irrigation system fluctuates as the price of land varies. In some instances, land costs may be sufficiently high to render spray irrigation unfeasible compared to advanced wastewater treatment alternatives with surface water discharge.

The capital expenditures for land may be offset, at least partially, by:

a) using the land for spray irrigation and cash crops which give annual returns on the investment,
b) forming agreements with local landowners to use their land for spray disposal of effluent in return for the benefit of having their crops irrigated at no cost.

Construction costs, even for systems having the same design capacity, vary widely due to varying site conditions, lengths of required piping, facilities, etc. Table 29 outlines some typical costs for construction of a spray irrigation system.

Labour requirements will depend mainly on the type of disposal system and its size. Table 29 estimates labour costs for a 455 m^3/d irrigation system.

Pumping equipment consumes most of the energy input to an irrigation system. Generally, the energy source is electricity with gas or diesel fuel used occasionally under special circumstances. Energy (electricity) costs can be estimated from the known rate of electricity consumption of the designated motor(s) for the pump(s), expected pumping duration, and electricity rates for the area.

Extra costs may be incurred for analysis of samples collected to comply with conditions of a specified monitoring program. Local provincial or municipal lab facilities may eliminate the need to use commercial lab facilities.

Repair or replacement of system nozzles, pipes, and pumps is frequently required. Since the frequency of such repairs is difficult to predict, it is probably best to set up a contingency fund.

TABLE 29 ACTUAL COSTS FOR 455 m^3/d SPRAY IRRIGATION EFFLUENT DISPOSAL SYSTEM IN THE U.S. (79)

Liquid loading rate, mm/wk (in/wk)	63.5 (2.5)
Required sprayfield area, ha (acres)[a]	10 (25)
Total land area, ha (acres)[b]	18.2 (45)
Capital Costs	
Land @ $1235.50/ha	$ 22 500
Earthwork[c]	8 800
Effluent pumping[d]	80 000
Transmission piping[e]	10 000
Winter storage[f]	30 000
Distribution pumping[g]	50 000
Distribution (spraying)[h]	50 000
Total capital costs	$250 000
Operating costs	
Labour	$ 4000/yr
Materials	400/yr
Power	700/yr
	$ 5 100/yr

a from (79) as are all cost estimates.
b assuming 6.07 ha for winter storage, 2 ha for buffer strips (61 m), etc.
c assuming 945 m^3 of cut per hectare.
d effluent pumping signifies pumping from secondary facilities to sprayfield or winter storage facilities - assumes 46 m pumping head.
e assuming 305 m of 15-cm buried pipe (forcemain).
f assuming seven months storage required, i.e., 30 wks x 7 days x 455 m^3 = 95 550 m^3.
g pumping from water storage facilities to sprayfield - assumes 15 m pumping head.
h a buried solid set system.

6 SMALL WASTEWATER COLLECTION SYSTEMS

The need for a treatment plant presupposes that some means has been provided to collect and transport sewage to the plant site. In sparsely populated areas, tank trucks may be used, but in most small systems there will be some type of sewer system. There are currently three types of collection systems, all of which use piping to convey the sewage:

- gravity systems,
- pressure systems,
- vacuum systems.

The gravity sewer is by far the oldest, commonest and most reliable, even when lift stations must be used to overcome natural low spots in the collection system. The other approaches are relatively recent and have application in special circumstances when conventional approaches are either too costly or technically too difficult to use.

In this section the basic properties and relative merits of each of these systems are discussed. The economics of each are compared and typical applications are noted.

In all cases, the designer of a sewage system must know how much sewage will be handled and treated over the design life of the system. This will include details on the following items:

- population served (25-year projection),
- volume of sewage per capita,
- type of system (domestic, commercial, industrial),
- rate of sewer infiltration,
- soil condition or type,
- total land area served (25-year projection),
- receiving sewage system capacity,
- proximity to water supply systems.

It is very important to design the collection system to handle the long-term needs (25 years minimum with 50 years not uncommon), even if the treatment system is designed on a short-term basis, with provision for future expansion.

6.1 Conventional Gravity Systems

The great advantage of gravity systems is reliability. A well-designed and constructed gravity sewer will last 50 years or more with minimal maintenance. Under most normal situations of soil type and topography this will be the most economic system.

Gravity systems may be classified according to the type of wastes conveyed. Combined sewer systems are designed to receive domestic sewage, industrial wastes and storm water, whereas seperate sewer systems involve two separate systems, one for the collection of sewage and industrial wastes and the other for the collection of storm water. During the past 50 years the trend has been away from combined systems because of their impact on treatment plant sizing, operation and costs, and toward the installation of separate systems. Recommended practice in small systems is to separately collect sanitary and storm water flows and to treat only the sanitary portion.

6.1.1 Sizing Gravity Sewers. The size of sewer pipe is based on the maximum sewage load expected over the design life of the system. The slope of the pipe must be sufficient to maintain a velocity of 0.6 m/s (2 fps) in sanitary and 0.9 m/s (3 fps) in storm sewers. These minimum flows ensure that the sewer is self-scouring in most normal situations. A slope of 1 in 100 (1%) will generally give 0.6 m/s velocity but will depend on pipe size and type. In general, a large sewer running to capacity needs less slope than a smaller sewer running to capacity. If the flow is the same in a large and small sewer (varying percent filled) the slope needed in each case is roughly the same. For detailed design of gravity sewers, reference should be made to the document, "Design and Construction of Sanitary and Storm Sewers", WPCF Manual of Practice No. 9 (80).

6.1.2 Pipe Materials. There are five types of sewer pipes available for use in a gravity collection system:

- plastic,
- concrete,
- vitrified clay,
- asbestos cement,
- cast iron.

With small systems, the first four are similar in cost and effectiveness. Plastic has the advantage of being lighter, and thus easier to handle during installation, than the other types of pipes. Increasing use of plastic pipe in sewer systems is evident, particulary for pipe sizes up to 46 cm (18 inch). Cast iron is perhaps the most expensive pipe in use and is utilized mainly where high structural loads will be imposed in the sewer. Infiltration should be avoided and is achieved with rubber, concrete or metal seals.

6.1.3 Maintenance. Maintenance of gravity sewers is achieved by designing a system with manholes at intervals of 100 to 150 metres. This allows the use of mechanical cleaning equipment to remove scale or build-ups of grease and to unclog plugged lines. Manholes also allow air to enter the sewer to prevent the production of anaerobic conditions and hydrogen sulphide gas. Normally, if the sewer flow is greater than 0.6 m/s and manholes are correctly designed and spaced, gas production problems will not arise.

6.1.4 Cost. The cost of sanitary sewers is most influenced by the type of material used, installation requirements, size of the system and location of the project. A survey of the installed cost of sewage collection systems in small communities indicated costs ranging from $594/person (population 500) to $1195/person (81).

6.2 Lift Stations

6.2.1 General. In many gravity systems it is necessary to first direct sewage to a local low point by gravity and then to pump it to the central treatment plant or to a main collector at a higher elevation. This pumping is performed by a lift station. Clearly this is a mechanical system and its reliability must be very high since the gravity system feeding sewage to the station can't be turned off. High reliability is essential in the design of the lift station and its ancillary equipment and services. In the event of failure, sewage will overflow the sump of the lift station and spill into the local environment. This may cause a serious health and environmental problem. Another cause of spills may be inadequate design capacity. If the sump or pipe sizing cannot handle the worst case flow, the sump will naturally overflow. Ideally, some storage or bypass capacity could be included. Standby pumps and alternate electrical power source should be included wherever possible.

6.2.2 Types of Stations. There are two main types of lift station: a dry pit and a wet pit.

In a dry pit (Figure 61) the pump shaft, which may be vertical or horizontal, takes its suction through a pipe from an adjacent sump or wet well. The exterior of the pump is dry at all times, permitting easy inspection and maintenance. Also, there is less chance of corrosion of the pump casing, shaft, bearings, and other parts.

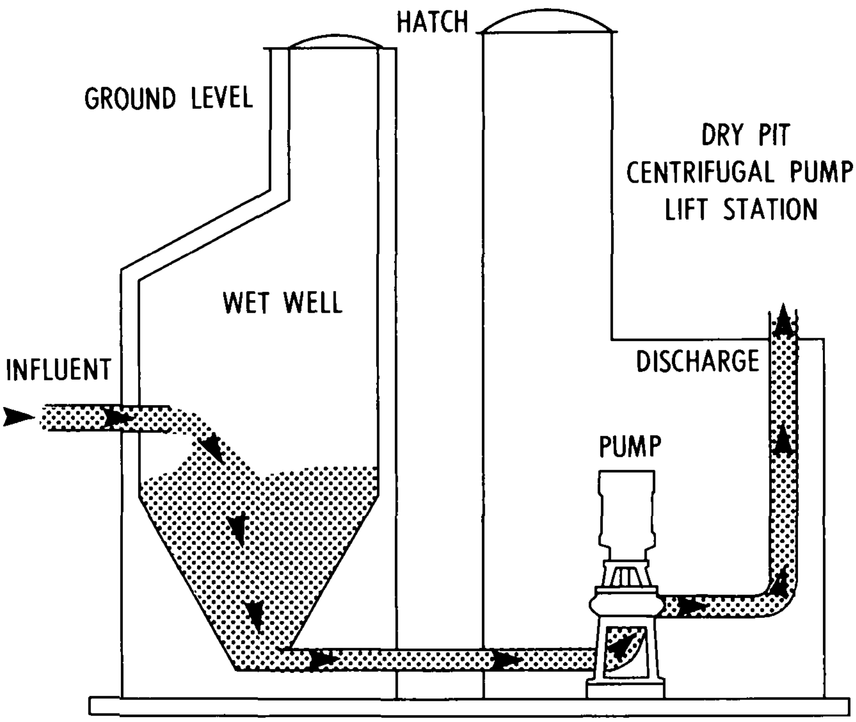


FIGURE 61 DRY PIT PUMPING STATION

In a wet pit the pump is immersed in the liquid handled. Figure 62 shows a typical pump of this type. It can be installed in a round, square, or rectangular metal or concrete sump. Where one pump does not have sufficient capacity, two or more pumps may be used in a single sump.

6.2.3 **Protection of Pumps.** Although many smaller pump stations operate efficiently without screens, pumps are such vital pieces of equipment that they should be protected. Screens are the most common form of protection. For a small plant, a basket type screen may be most applicable. Many variations of the bar screen are available but in general are applicable only for large plants. A comminutor, or alternately a macerator pump, may provide a good alternative to screens.

6.2.4 **Types of Pumps.** Raw sewage contains a variety of solids such as sticks, rags, rocks, hair, etc. These can clog the pump and damage rotating or stationary parts, reducing pump efficiency or causing complete stoppage of the unit. A number of clogless or nonclogging centrifugal pumps have been developed for use in pumping stations. Though the design details differ from one manufacturer to another, most pumps of this type have impellers with at most two or three vanes, or none at all. The impeller may be either open or closed, but the open type seems to be more popular at present. Usually, the clearance between the vanes is large enough to allow any solid entering the pump to pass out through the discharge. In some pump designs the suction pipe is 25 percent larger than the discharge, in others, both are the same size. The smallest discharge size recommended is generally 10 cm (4 inch), although some 7.6, 5 and 38 cm (3, 2 and 1 1/2 inch) pumps are also built. These smaller pumps should only be used with a macerator to prevent clogging.

Nonclogging sewage pumps are built as either horizontal or vertical units. Figures 61 and 62 show typical vertical sewage pumps with the motor mounted directly on the pump frame. Present trends in sewage system design indicate a decided preference for vertical pumps in almost all types of installations. The advantages of vertical installation include the need for less floor area, simpler piping connections, the avoidance of gas-accumulation problems in the pump suction, and the possibility of using extended shafts to isolate the motor from its pump.

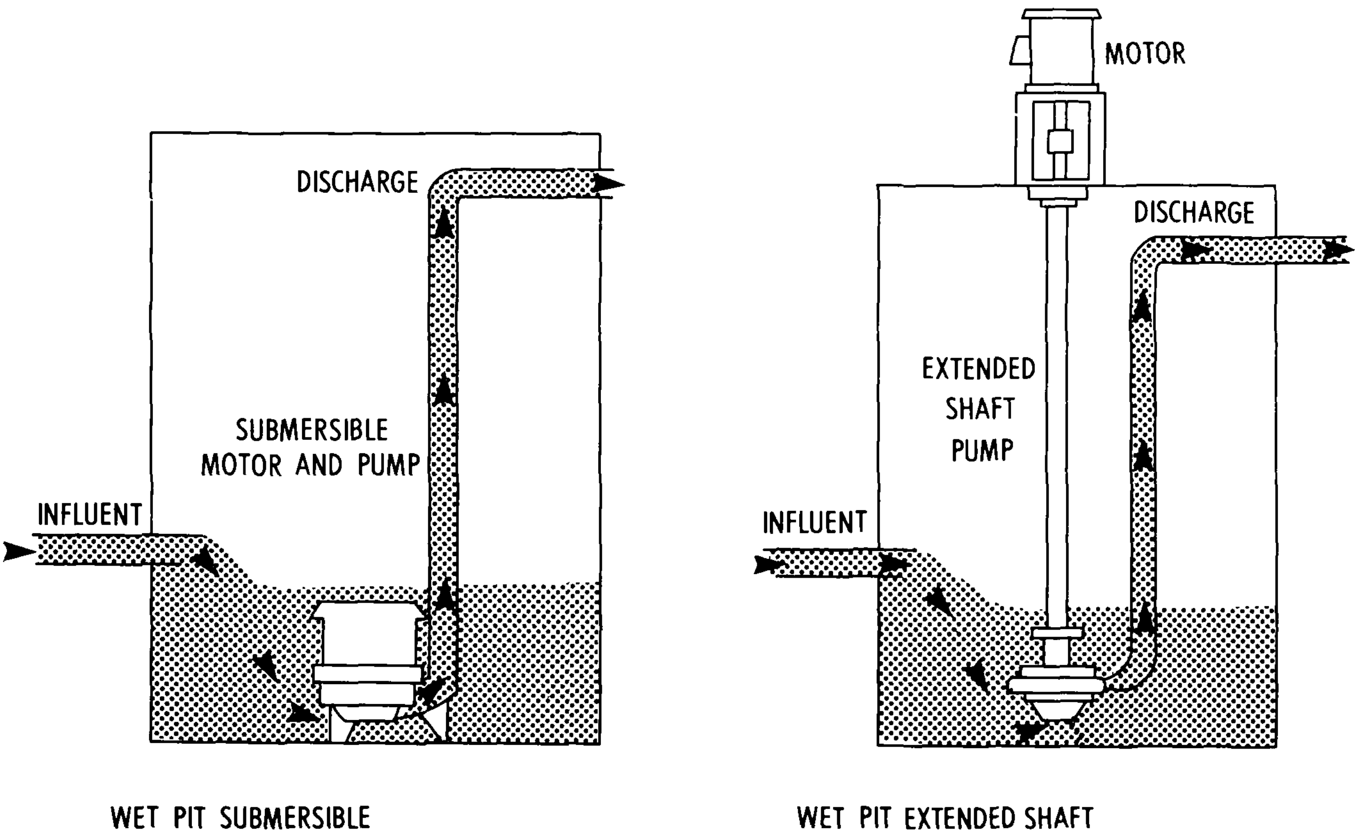


FIGURE 62 WET PIT PUMPING STATION

6.2.5 Pump Controls. A wet well has level indicators which turn the pumps on and off in response to varying sewage levels. If dual pumps are installed then some provision to alternate pumps should be included as a means of preventing failure of backup systems through lack of use. A wet well is never allowed to drain completely to avoid damaging the pumps. Some systems use variable speed pumps, which provide more continuous pump operation and less chance of failure due to control malfunction. The level control sensors should be a non-clog type such as floats or pressure-sensitive relays.

6.2.6 Wet Well Design. The design of the wet well is very important. Solids will naturally settle in the well and must be readily removed during the pumping cycle. This is usually achieved in a lift station with non-submersible pumps (Figure 61) by having an adequately sloped (1 to 1) bottom in the well. When pumping starts the sewage velocity is sufficient to dislodge any settled solids. In the second type of system (Figure 62), the pump inlet is placed above the well bottom and is not likely to plug. The overall sizing of the well must balance the need to prevent septic conditions against the need for a minimum pump cycle of five minutes. This means a typical cycle of five minutes on with a maximum of 30 minutes off. This maximum holding time also prevents any major solids build up or compaction of the solids which may settle.

6.2.7 Flow Measurement. Where it is desirable to measure the flow of pumped sewage, several methods may be considered. A counter can be used to total the number of pump cycles (approximate flow measure) or a power totalizer can be used to monitor the pump motor loading, which in turn can be related directly to flow. Ordinary flow measuring devices are not practical because of the intermittent operation of most lift stations, although they are very suitable for continuous, variable-speed pumping arrangements.

6.2.8 Power Requirements. Electrical power requirements vary with the load at each station. In general, a dual supply (one to each pump) is desirable to prevent power outages. In remote areas a backup generator should be considered. For most motors above 1 hp, a 550-volt, three-phase system is considered practical. Suitable design to ensure safety is essential in the humid atmosphere of a pumping station. Remote indication of major upsets (power out, current overload or overheating) should be considered to alert maintenance crews.

6.2.9 Force Mains. The final component of the lift system is the force main which carries the sewage, under pressure, either to the treatment plant or to another collector sewer. A major problem in force mains is the development of septic conditions. These conditions usually occur because the main is long, unvented and subject to intermittent use. Compressed air or chlorine can be injected into the main to reduce sulphide production. Most force mains are made from cast iron or reinforced asbestos or concrete pipe and sized to provide a velocity of 1 to 1.5 m/s (3-5 fps). To ensure scouring of deposited solids, a minimum velocity of 0.75 to 1 m/s (2-3 fps), two to three times per day is recommended. Cleanout plugs should be located at low points, or periodically along the main. Air venting facilities are necessary at high points in the system.

6.2.10 Safety. The safety aspects of lift stations are similar to those considered in the design of manholes. Forced ventilation of wet and dry wells is necessary to prevent sulphide buildup. Normal items such as guards on ladders and moving equipment should be provided. Design to avoid low-lying pipes and other tripping hazards should be encouraged. Corrosion-resistant materials are necessary to prevent mechanical failure of equipment and facilities, which in turn could create unsafe conditions.

6.3 Pressure Sewer Systems

6.3.1 General. Pressure sewer systems are analogous to force mains except that the entire collection system from the house basement to the treatment plant is run under low pressure (241 kPa or 35 psi). The advantage of this approach is that the collection system may be constructed almost completely independent of topography. Considerable cost savings are possible if deep excavation can be avoided because of this feature, particularly in difficult construction areas, such as rock. Figure 63 shows an idealization of these advantages.

Pressure systems function on the basis that each individual facility (e.g., household, apartment building, etc.) served by the system has a small pumping station located in its basement or shares a common pumping station located on or near a property line with a neighbour. The pumping station receives sanitary wastes from various sources throughout a facility or from neighbouring facilities, usually by gravity; and pumps the wastes into a common, pressurized collector main, eventually discharging into a sewage treatment centre or gravity collection sewer. Because piped connections between the house(s) and the collection line are small (i.e., 3.18 cm or 1-1/4 inch diameter), grinder pumps must be used in each station to prevent clogging of lines. Most control agencies still regard pressure systems as somewhat experimental and usually apply more detailed analyses to applications for their use. However, the advantages are recognized when

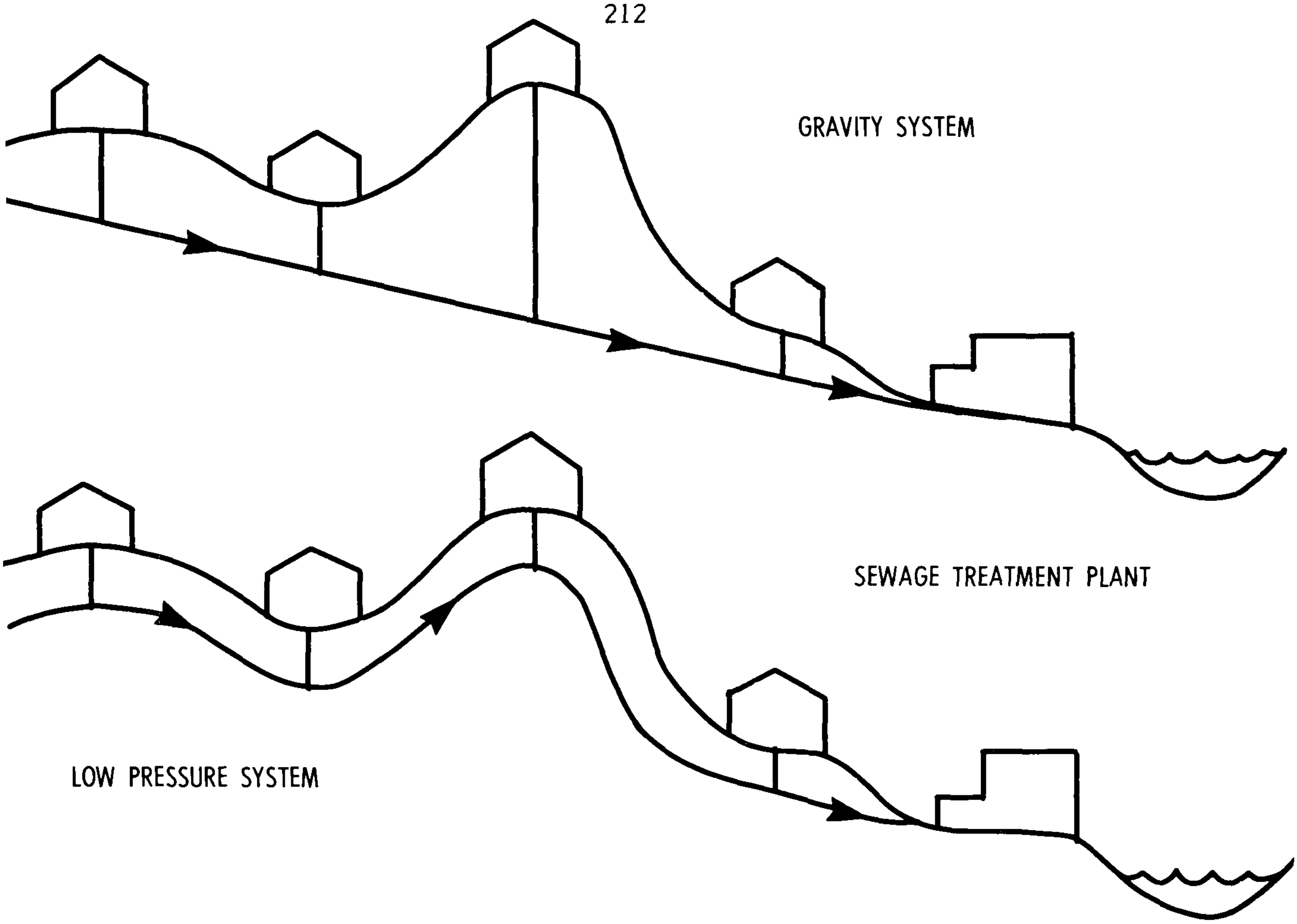


FIGURE 63 IDEALIZATION OF PRESSURE AND GRAVITY COLLECTION SYSTEMS

difficult situations, such as shallow bedrock or a high water table, are encountered. The cost effectiveness under such conditions will ultimately lead to many more applications.

6.3.2 Effect on Sewage Characteristics. Several differences exist in the design criteria of sewage treatment plants when pressure systems are used. The sewage, as discharged to the treatment plant, will be stronger (300-400 mg/L BOD) primarily because infiltration is nonexistant. Unit generation rates of 0.16 to 0.19 m per person per day (40 to 50 gpcd) have been determined for pressure systems, compared to 0.28 to 0.38 m^3 per person per day (75 to 100 gpcd) commonly associated with gravity systems, again primarily due to the absence of infiltration and inflow contributors (82). Other areas which may be of concern, depending on the nature of the treatment facility, are the shock loading effect associated with the use of pressure sewers and the delivery of anaerobic waste to the treatment process. Shock loading occurs because of the tremendous variation in flow in a pressure sewer system, resulting from activities within each individual residence of a community. Because extraneous flows are prevented from entering the collection system, no-flow conditions may periodically occur at certain hours throughout the day, followed by extreme peak flow conditions. Anaerobic conditions develop within the collection system as the waste decomposes inside the pressure pipes during no-flow or low-flow periods. Hydrogen sulphide may be detected at the discharge of the pressure main and, depending on the sulphide concentration, may affect the settleability of the waste by encouraging the growth of filamentous organisms (82). Because of the special characteristics of wastewaters discharged from pressure sewers, it is suggested that experienced designers be employed when selecting the type of treatment and any required design modifications to pressure sewer applications, at least until further data becomes available.

6.3.3 Grinder Pumps. The driving force of the pressure system, grinder pumps, are commonly located in each premise serviced by the sewer, usually in the basement. This location is preferred primarily because of ease of maintenance and increase of service life. Figure 64 shows a typical household system.

Grinder pumps are designed with the following capabilities:

a) Foreign objects in sewage are ground to produce a fine slurry which will not clog equipment or pipelines.

b) The units can feed into a pressure main at nearly a constant rate regardless of backpressure (up to 27 m or 90 ft) or can operate sequentially against a specified backpressure. These properties are necessary because of the probability of parallel operation of several units simultaneously into a common street sewer. A complete grinder-pump mounted on a holding tank is shown in Figure 65. The pump motor is usually 1 hp and may be equipped with a thermal overload and automatic restart, depending on the manufacturer. It can be run off house circuits without circuit overloading. The motor is controlled by the water level in the tank, which operates control circuits to start and stop the grinding-pumping action.

The sump or holding tank in the average home is normally sized to hold approximately 0.19 m^3 (60 gallons) of wastewater, but must be large enough to provide reserve capacity based on power outages in the area served. Excess volume is undesirable as sewage may become septic. Provision to flush the system should be available.

Upgrading an existing septic tank system to a central pressure sewer collection system presents an alternative to the standard sump and holding tank. This involves the incorporation of the septic tank into the pressure system, where it functions as a holding tank/treatment device. Wastes discharging from a residence enter the septic

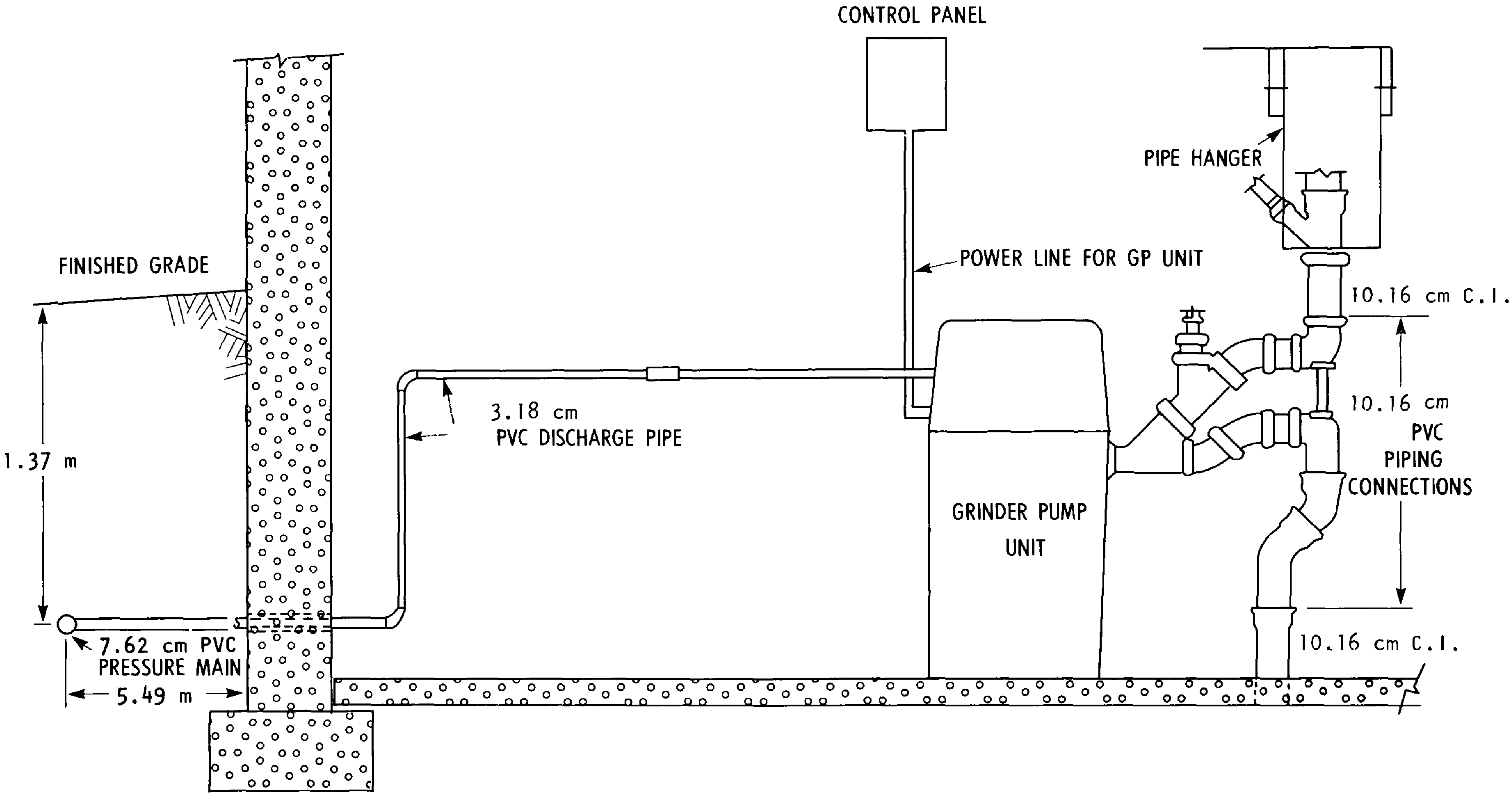


FIGURE 64 TYPICAL INSTALLATION OF GRINDER PUMP UNIT WITHIN A HOUSE

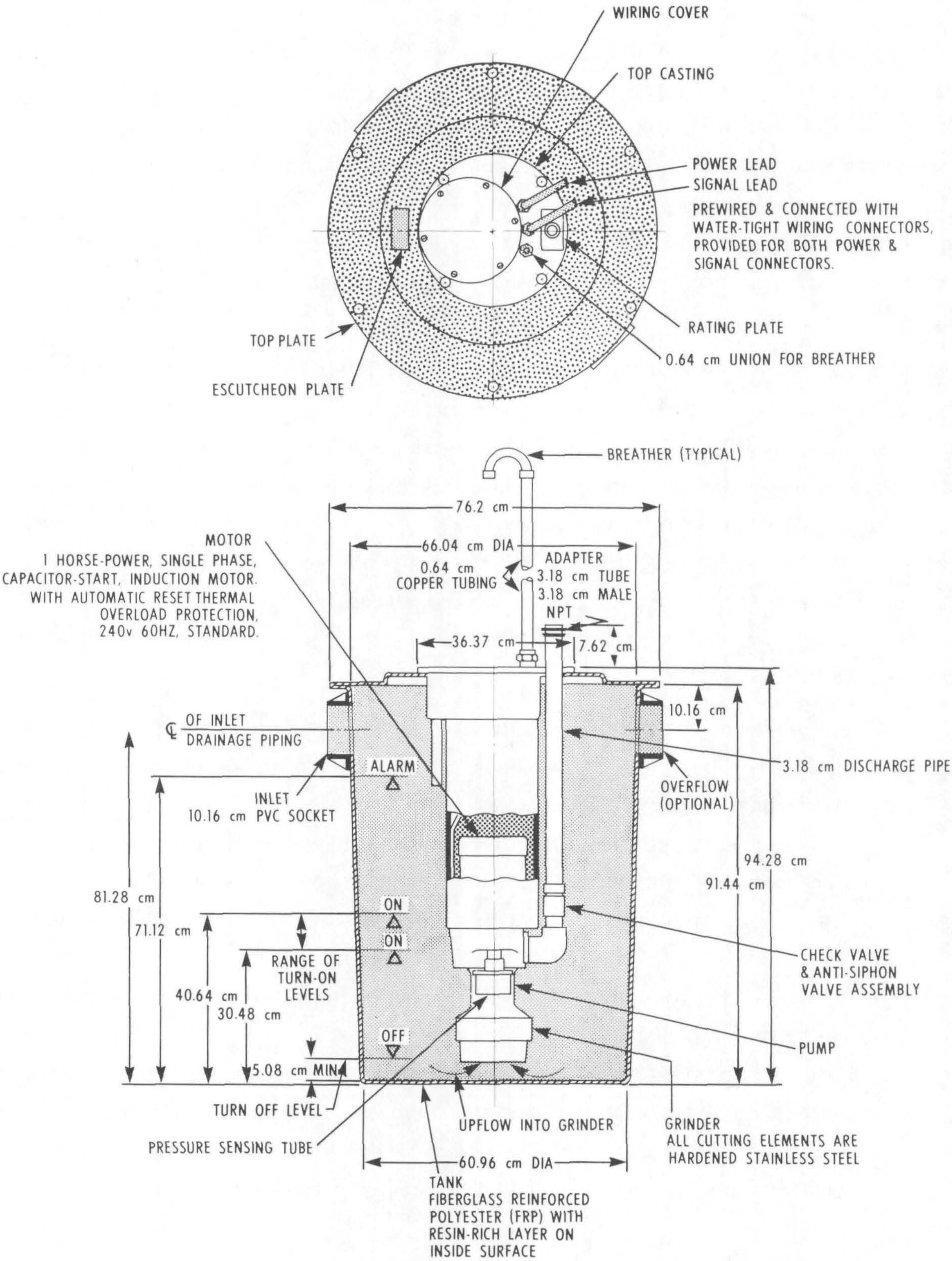


FIGURE 65 PLAN AND CROSS-SECTION OF GRINDER PUMP UNIT (From Environment/One)

tanks, where heavy solids settle and grease and other flotables collect in the scum layer. The tank effluent flows to a receiving tank, or in the case of multi-chambered septic tanks, to the final chamber fitted with pumps, sensors, and valves required for a septic tank effluent pumping (STEP) system. With removal of solids and greases occurring in the septic tank, small centrifugal pumps may be employed for the STEP system, rather than grinder pumps. The cost of a STEP system should be examined in comparison with a grinder pump system on a case-by-case basis. The final cost of a STEP system should include septic tank clean-out costs every two to three years.

6.3.4 Sewer System Description. The sewer main itself is laid out as a branched system, without loops, to insure flow in only one direction (Figure 66). The size of the collection line (usually PVC construction) must be calculated, as in the gravity system, on the basis of long-term flows from the pump units. Infiltration can be ignored. Air relief valves are required in the system to purge any air pockets. Valves are also needed every 122 to 183 m (400-600 ft) for routine cleanout and maintenance. Check valves and a manual shut-off valve at each service connection are necessary to prevent backup into the sump; these latter two valves are often duplicated at each grinder-pump station to guarantee reliability.

6.3.5 Operational Considerations. Currently available grinder-pump units are reliable in operation, requiring one to two hours of routine preventive maintenance per year. Power outages may be a more frequent problem than breakdown, but of no concern if less than one to two hours duration since the sump has adequate reserve capacity. If longer outages are common, standby power should be considered or portable pump-out facilities provided. Operation and maintenance of pressure mains includes periodic cleaning, repairing leaks, and replacement of broken sections.

To date, the operational problems which have arisen in pressure systems have not been critical if maintenance and spare parts were readily available. Clearly this feature is essential in such mechanically-dependent systems. Build-ups of grease in the sump, etc. have not caused problems in commercially-available units, although some prototypes did fail to operate properly for this reason. The elimination of highly-abrasive grit from the grinder pump is important and is basically a problem only the user can address.

The management responsibility for pressure sewer systems within a municipality must be resolved early in the planning of a project. Either the municipal authority, individual homeowner, or a conbination of the two will be responsible for the

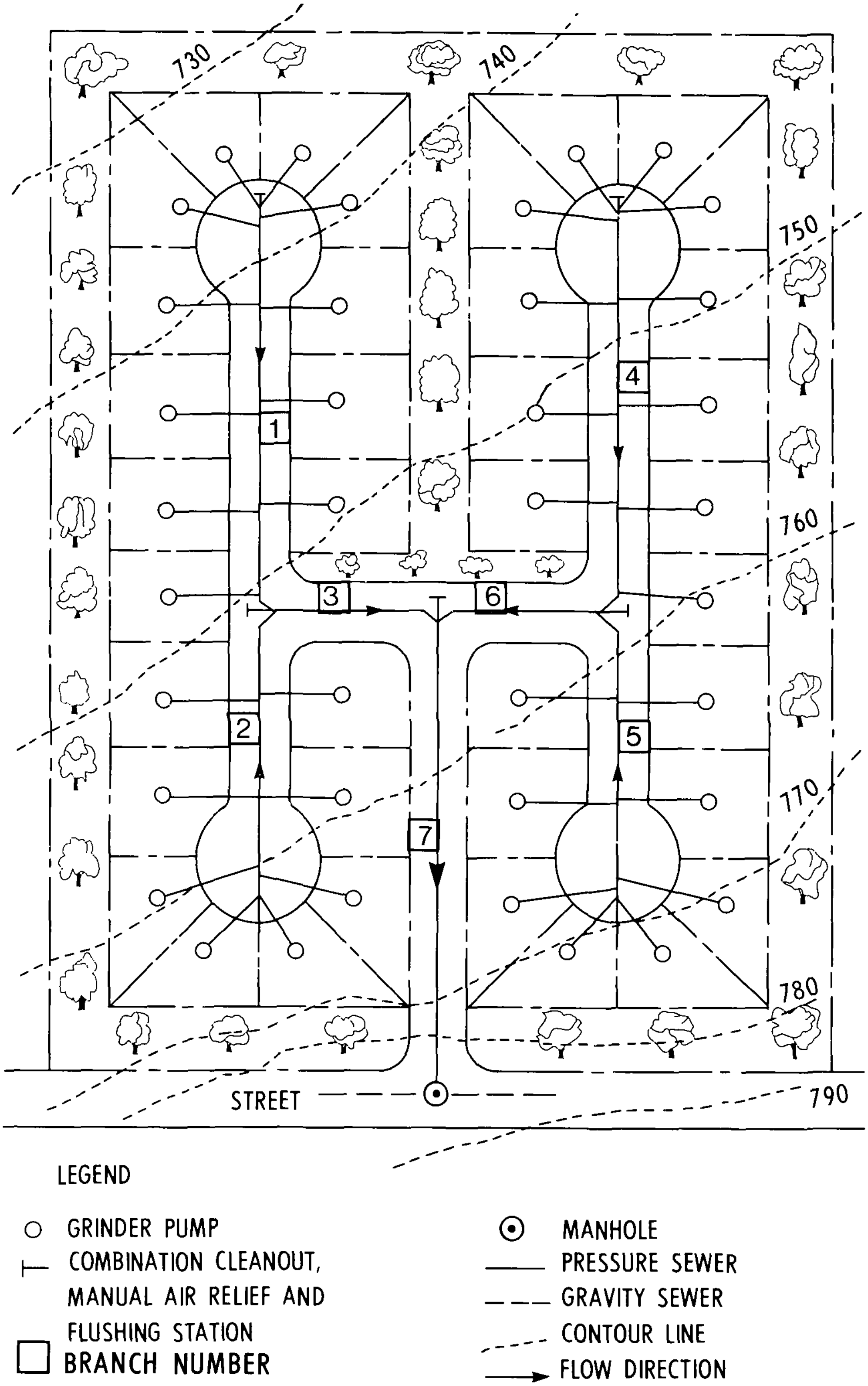


FIGURE 66 PRESSURE SYSTEM FLOW DIAGRAM

care and maintenance of the lower pressure systems (e.g., municipality responsible for pressure mains, homeowner responsible for pump station). The advantages of pumping stations outside an individual residence, as in the case of shared or STEP systems, are obvious where the municipality assumes full responsibility for the system. In some cases, however, the pump units must be located in the residence, and commitments and responsibilities must be negotiated between the homeowners and the municipality.

6.3.6 Costs. The cost of a grinder pump unit, including a prorated share for the main sewer system, is approximately \$2500 for a normal home system (82). The cost for sewer main materials is the same as for a force main of the same size. Excavation costs should be much lower. In fact, if excavation is not cheaper, the decision to use a pressure system would be questionable. The operating cost is power only and is around \$6 per year (1.2 m^3/d (300 gpd), 200 kWh/yr @ 3¢/kWh). Maintenance costs are estimated to be \$50 per pump per year. O&M cost for the pressure main will be approximately \$62/km/yr (\$100/mi/yr) (82).

6.3.7 Design Example (82). The system shown in Figure 66 can be used for a simplified design example, assuming a scale of 1 cm = 36 m. The following are also assumed: a) PVC mains of 5.1 cm (2 inch) nominal diameter, except for a 7.6-cm (3-inch) interceptor (branch 7); b) service lines of 3.2 cm (1.25 inch) nominal diameter PVC; and c) grinder pump units.

Unit installed costs are assumed to be:

7.6 cm PVC pipe	@	\$13.00/m
5.1 cm PVC pipe	@	\$10.00/m
3.2 cm PVC pipe	@	\$ 6.50/m
Service line connections	@	\$35.00 each
GP units, including electrical hookup	@	\$2000 each
Cleanouts with manual air-relief valves	@	\$500 each

A rough estimate of the capital cost of this system is:

7.6 cm pipe (240 m)	=	\$ 3 120
5.1 cm pipe (960 m)	=	\$ 9 600

3.2 cm pipe (1740 m)	=	$ 11 310
Connections (38)	=	$ 1 330
GP units (38)	=	$ 76 000
Cleanouts and valves (7)	=	$ 3 500
TOTAL		$104 860

This represents a cost per home of about $2 760.

Two things are vividly shown in this example: the economical nature of the pressure sewer, and the high cost of grinder pumps. It is because of the latter that shared pumping stations or septic tank effluent pump (STEP) systems are being used. In many rural areas, sewers are now required because poor soil conditions have obviated continued use of the original septic tank - soil absorption system. If this were the case in the example location, STEP units could be substituted for the GP units at a cost of about $1000 per installation as compared to $2000 for the GP installation. This substitution in the previous example would reduce the cost per home to about $1760.

The O&M costs for the example installation would approximately be:

Pipe: 2940 m @ $62/km/yr		=	$182/yr
GP:	38 units @ $50/yr	=	$1900/yr
Power:	38 units @ $6/yr	=	$228/yr
	TOTAL		$2310/yr

This amounts to an O&M cost per home of $60.79 or a monthly cost of about $5.07. The amortized capital cost must be added to this to obtain the total monthly cost. No engineering, legal fees, or other additional costs are considered for this example. Therefore, amortization of the $104 860 capital cost over 20 years at 10 percent interest yields an annual cost of $324.10 per home, or $27.01/month. The total monthly cost per home is then $32.08.

From the previous discussion of the O&M costs for GP and STEP systems, there is not enough evidence to justify a difference in the O&M cost estimates for these two types of pressure systems. Therefore, the substitution of STEP units for GP units in the above example would yield a monthly cost per home of $5.07 for O&M plus the amortization cost of the STEP system computed on the same basis as the GP system. The amortization of the $66 860 capital cost yields an annual cost of $206.64/home, or $17.22/month. Therefore, the total monthly cost for the STEP system is then $22.29.

The foregoing example is admittedly crude, but it gives some idea of the cost estimating procedures necessary to evaluate proposed pressure sewer systems. Additional factors will have to be evaluated to properly accomplish such an estimate in a real situation.

6.4 Vacuum Systems

6.4.1 General. Liquid transport in the vacuum system (Figure 67) is accomplished by pressure differential: atmospheric pressure at the upper side of valves, and approximately half an atmosphere of pressure in the vacuum tank and piping network (38 to 45 cm (15 to 25 inches) of mercury). A vacuum collection system is relatively independent of grade changes; up to 5 m (15 feet) of head is possible.

Advantages of a vacuum collection system are:

a) Installation costs are lower than for a gravity system because there is less trench excavation. The pipe can be installed in the same trench as a water main and can follow ground topography within limits of slope to transport pockets.
b) Material costs are also less because small-diameter pipes are used.
c) Groundwater infiltration is minimal and therefore wastewater treatment is more economical because of the reduced flows and higher concentration of solids.
d) Repair or replacement of damaged pipes can be done quickly and at low cost because the lines are close to the surface.

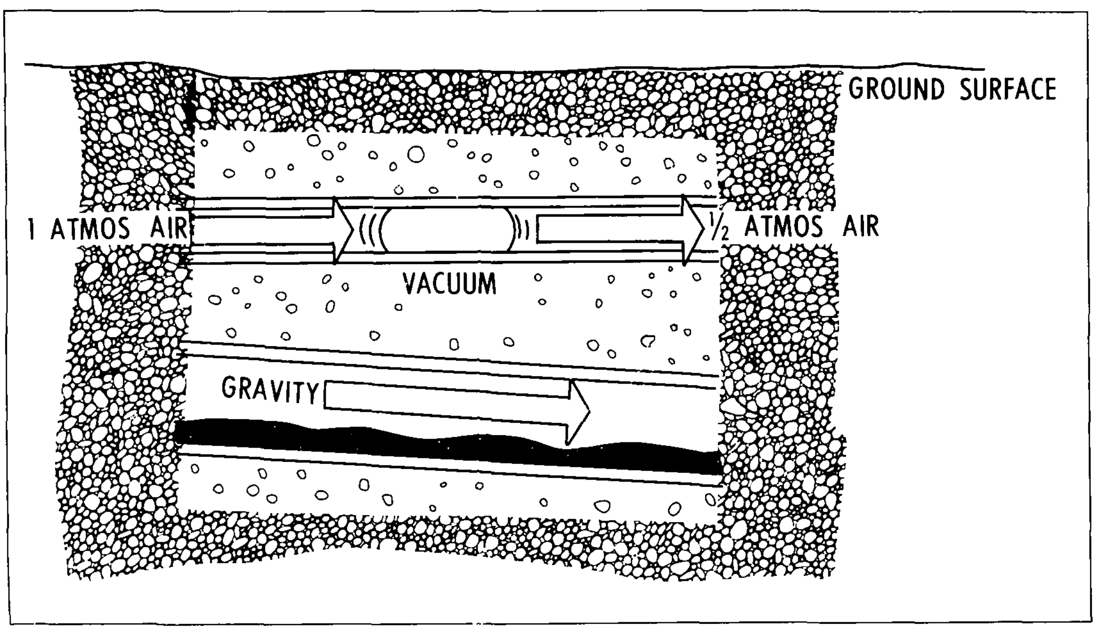


FIGURE 67 COMPARISON OF VACUUM AND GRAVITY SYSTEMS

Disadvantages of vacuum systems include:

a) The system is totally power dependent. To safeguard against power failure, a duplicate power supply may be necessary.
b) The system is sensitive (less forgiving) to proper design and installation. The pipe network must be fully sealed to maintain vacuum pressure in the system.
c) The maximum lift in the vacuum system is approximately 5 m. This restricts the topography on which the system can be used.

6.4.2 Effect on Sewage Characteristics. Flows from a vacuum system reportedly exhibit greater hourly variations than gravity flows because of the storage and intermittent discharge features of the central vacuum station (83). In addition to flow variability, the storage of raw wastewater at a central station implies that biological degradation of organic materials could occur, resulting in delivery of an anaerobic liquid to the treatment facility.

Limited investigations have been carried out to determine the composition of sewage collected by vacuum systems. The strength of the raw waste will vary with the type of system, i.e., a vacuum toilet incorporated into the plumbing of the house, or conventional plumbing in the house with an interface valve outside. BOD concentrations from the former system may be 2000 to 2500 mg/L (84); whereas the latter system may have BOD concentrations of 300 to 350 mg/L (83).

6.4.3 System Description (85,86,87). The vacuum sewer system is not actually one system, but rather a number of systems which are used to meet a variety of requirements. A "black water" system transports wastewater from toilets and urinals, while all other household wastewater ("grey water") is transported by conventional methods. Alternatively, grey water may also be transported by vacuum within a residence. The black and grey waters may be transported in separate piping networks ("two-pipe system"), or combined in the same pipe ("one-pipe system").

As can be appreciated, vacuum toilets and grey water vacuum systems have a significant effect on the volume of water used within a residence. The vacuum toilet uses less than 1.2 L (1 qt) of water per flush, compared to the 10 to 18 L used by conventional units. The water used in vacuum toilets is primarily for cleaning the bowl since very little water is required for waste transport in the vacuum main.

The on-line system, a variation of the one-pipe system, allows conventional plumbing within the home. A wastewater admittance valve (interface valve) is installed outside the residence at the end of the gravity service lateral to connect the conventional plumbing to the vacuum system. The sewage is then propelled through the vacuum main to a central collecting station. From the collecting station the sewage is discharged to a gravity sewer, force main, treatment plant, lagoon, etc.

In Canada, most communities are located in areas with plenty of available water. Thus the water-saving vacuum toilet and in-house grey water vacuum system are rarely used. However, a vacuum system in conjunction with conventional in-house plumbing may prove to be a viable alternative in many situations, including hilly or rocky terrain, low-density areas, and areas having adverse grade conditions, or high water tables.

6.4.4 Vacuum Collection Lines. Sewage collection lines in an "on-line" vacuum sewage system are categorized as follows:

a) gravity service lines,
b) vacuum service lines,
c) main and branch lines.

The gravity service line is the pipe running from the residence to the interface valve. This pipe will have a continuous downslope to the interface unit. In some instances (dry chamber interface units), the gravity service line is used to store a limited volume of wastewater, which then activates the interface valve (see Figure 68). The gravity pipe must be installed to the specifications of the vacuum system supplier.

Vacuum service lines are the laterals which branch off the main vacuum line to connect with the interface units. The pipe size for vacuum service lines is normally 50 mm (2 inches).

The main or branch lines of a vacuum system connect the central vacuum collecting station to the service lines. Pipe sizes in the main and branch lines are normally 76 mm (3 inches), 102 mm (4 inches) and 152 mm (6 inches). Vacuum lines are PVC or ABS in construction and may be up to 2440 m (8000 ft) in length.

There are two types of interface units:

a) dry chamber (wastewater storage in gravity pipe),
b) wet chamber (wastewater storage in sealed off section in chamber).

Standard interface units are used when sewage is collected from one or two houses. Design capacity of the standard unit is as follows:

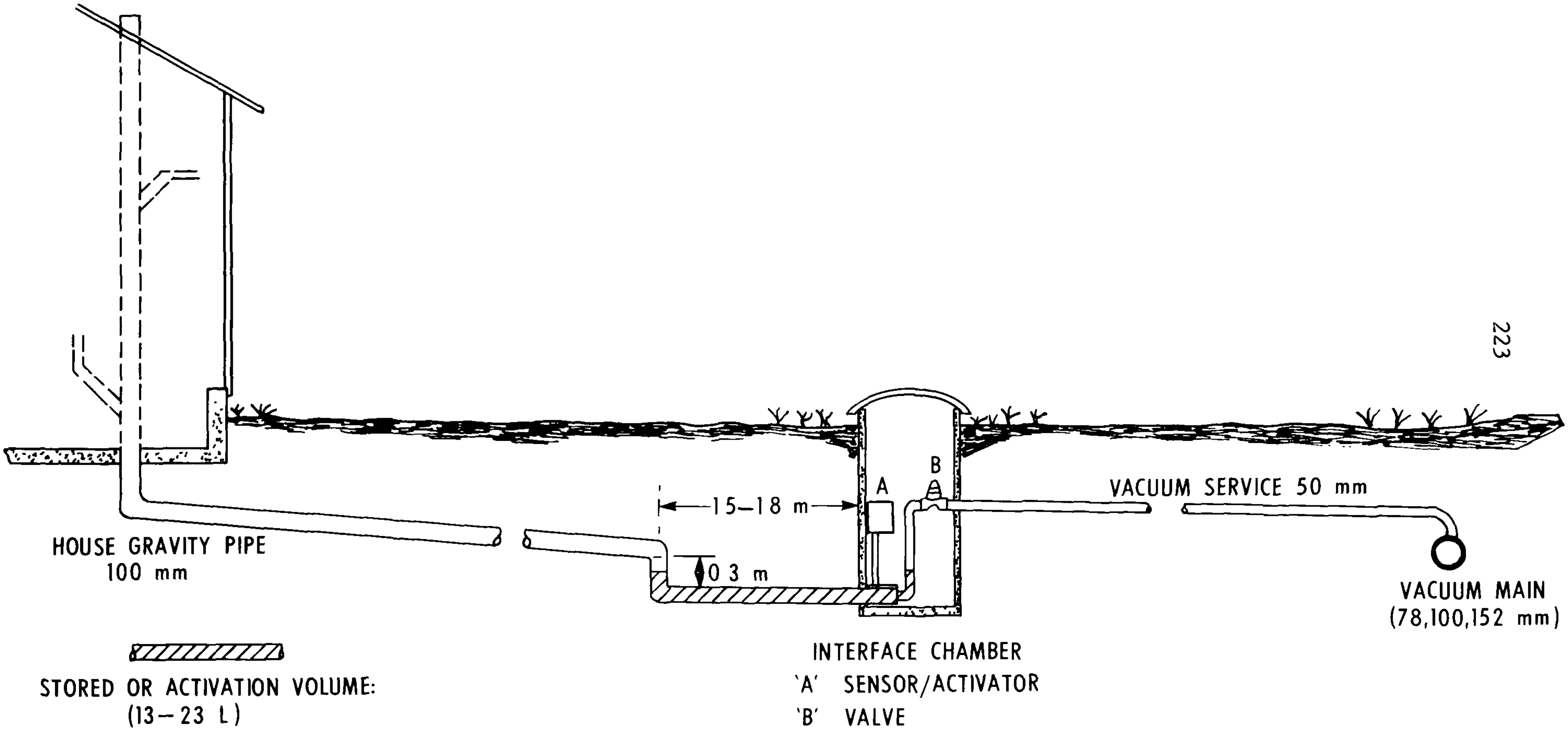


FIGURE 68 ON-LINE VACUUM SYSTEM (DRY CHAMBER INTERFACE UNIT)
(From Vacusan Systems Ltd.)

Maximum daily flow: 2390 L (525 gallons)
Maximim peak flow: 341 L (75 gallons)

When the expected sewage volume from a single source exceeds the volume allowed for an interface unit, an interbuffer unit is used. This unit allows considerable storage capacity and is designed to permit withdrawal of a large volume of sewage while maintaining the optimum air to water ratio. An interbuffer unit consists of a larger wet chamber, discharge valve sensor/activator and an air inlet valve. The vacuum service lateral is usually larger in diameter than that for an interface unit.

Interface units consist of a sensor/activator and a discharge valve which are totally vacuum operated. Storage of sewage upstream of the unit (typically 14 to 23 L) results in a slight pressure rise (10 to 15 cm) at the sensor/activator. The interface valve opens and the wastewater is pushed into the vacuum service line by atmospheric pressure. The sensor/activator opens the discharge valve only if there is at least 20 cm Hg vacuum at the valve.

A quantity of air (80 to 120 L) enters the vacuum pipe behind the sewage plug, settling up a pressure differential in the service line. This pressure differential transports the waste into the vacuum main and toward a central vacuum collection facility. Because of the friction which occurs between the liquid plug and the pipe wall the plug must eventually break down. The air which was trapped behind the plug then breaks through it and is evacuated from the system. To reform the plug, the pipe is fitted with "transport pockets" at intervals; the wastewater flows by gravity to these pockets during non-transport periods. Once the liquid plug has been reformed in the transport pocket, an increase in the upstream pressure, created by the admission of air to the system at some point upstream, will cause the plug to be transported further down the pipe until it again breaks down. The transport of wastewater in vacuum pipes is, therefore, intermittent rather than continous. Transport takes place when a sufficient pressure differential forms across a plug of wastewater.

Pipe slopes and distances between transport pockets vary according to topography and the specific details of a site. Standard transport pockets are placed at 60 m (200 ft) intervals, with the interconnecting pipe at a minimum 0.35% downslope. The standard transport pocket has a 23 cm (9 inch) vertical dimension. Should the spacing between pockets be reduced while maintaining the minimum slope of 0.35% a small net lift will result. A minimum drop of one pipe diameter from one transport pocket to the next transport pocket is required.

Lift pockets are used when uphill transport of wastewater is required and the net lift given by standard pockets is insufficient. A lift pocket is identical to the standard transport pocket except that it is larger. At a minimum spacing of 15 m (50 ft), lift pockets provide a net lift of 30 cm per pocket for 152 mm-pipe, 46 cm for 102 mm-pipe, and 61 cm for 76 mm-pipe. No pockets are required for downhill transport of wastes.

Inspection ports are another important component of the vacuum main which must be considered in the design of the collection systems. These ports allow access to the buried pipe and facilitate investigation of problems with the vacuum piping. The inspection port is connected via a riser down to the transport pocket, or in the case of downhill transport, directly to the main lines.

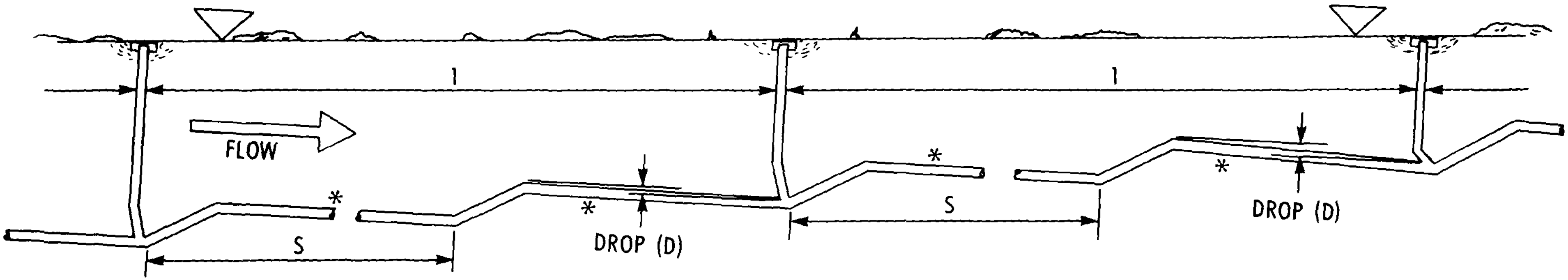
Figure 69 is a profile of a vacuum sewer designed for uphill transport of wastewater.

6.4.5 Vacuum Collection Station. The vacuum collection station is the heart of the vacuum collection system and, as such, considerable attention must be paid to designing and installing the unit. The station consists of vacuum collection tanks, vacuum pumps, sewage discharge pumps and an automatic controller (87).

Vacuum collection tanks provide a vacuum reservoir for the system, vacuum buffer for the vacuum pumps and a reservoir for the sewage discharge pumps. The vacuum pumps are completely automatic. The discharge pumps are coupled to the bottom of the collection tank in a dry pit configuration and periodically automatically discharge sewage from the tanks to the receiver. All systems are normally provided with back-up or standby pumps.

The collection station is automatically controlled via a pre-built controller. The controller accepts signals from mercury float switches in the collecting tanks and vacuum sensing switches on the tanks, and provides appropriate instructions to the vacuum pumps and discharge pumps (87). The controller provides visual indication of the operations and conditions of the system, and can be provided with a remote alarm.

6.4.6 Operational Considerations. The complexity of the vacuum equipment requires training of operating personnel to properly maintain a vacuum sewer system (83). Manufacturers recognize this and are reacting with improved technical assistance and operation and maintenance manuals to assist operating personnel. As is the case when dealing with other pieces of mechanical equipment, O&M of vacuum systems entails normal operation and maintenance, preventive maintenance and emergency maintenance.



USE OF POCKETS FOR LIFT

STANDARD POCKETS: at a minimum S of 45 7 m use of standard pockets gives a net lift of 7 6 cm per pocket in all size pipes

LIFT POCKETS: at minimum S of 15 2 m, use of lift pockets gives a net lift of 30 5 cm per pocket (15 2 cm pipe), 45.7 cm per pocket (10 2 cm pipe) and 61 cm per pocket (7.6 cm pipe)

LEGEND

'D' DROP. one pipe diameter, MINIMUM

'S' MINIMUM spacing between pockets is 15 2 m for lift pocket and 45 7 cm for standard pocket

'1' MAXIMUM distance between inspection ports 61 m

'*' Slope pipe to suit pocket spacing and elevation but NEVER LESS THAN 0 35%

FIGURE 69 PROFILE FOR UPHIL TRANSPORT IN VACUUM MAINS
(From Vacusan Systems Ltd.)

Components of the vacuum system which require special attention are the vacuum interface valves and the central collection station. It is estimated that four hours per connection per year should be allocated to operation and preventive maintenance (83). Breakdown maintenance will require time in addition to preventive maintenance tasks. Long-term operating information for vacuum systems in Canada is required before a precise breakdown of operation and maintenance requirements can be provided.

6.4.7 Costs (85). The actual cost of vacuum sewer systems in Canada cannot be determined until more installations have been built in this country. The cost will depend on local labour costs, construction materials prices, transportation costs and, for some items, import duties.

Experience in Europe and the Bahamas has indicated that, under conditions which favour the use of vacuum sewers, the capital cost of vacuum systems is roughly two-thirds that of comparable gravity systems. However, each proposed installation will require individual consideration because of the variations in the physical and economic factors which determine the ultimate cost of any system, and because the type of vacuum system used will vary according to local requirements.

Maintenance and operational costs also depend to a large extent on local conditions. The amount of time required for operation and maintenance depends primarily on the size and type of vacuum system considered, but will also be influenced by such factors as user abuse and the severity of the local climate. The cost of electric power can be estimated by determining the number, sizes, and operating periods of the vacuum and sewage pumps. The cost of replacement parts, vehicles, and consumable materials may be estimated based on life expectancies and estimated consumption rates, as well as on suppliers' price lists and transportation costs.

Comparisons of the operational and maintenance costs of vacuum sewer systems and equivalent conventional systems in Canada may also be produced when more experience is acquired. It has been reported that, over a period of eight years, the first large-scale Swedish vacuum installation has produced lower operational costs than a conventional installation of the same size (83).

7 REGULATORY AGENCY REQUIREMENTS AND THE APPROVAL PROCESS

7.1 Introduction

Fulfillment of regulatory agency requirements with respect to public health protection and pollution prevention and control is a major concern of those involved in planning wastewater disposal facilities. This concern often becomes frustration when it is realized that each level of government (provincial and federal) has its own interpretation of "fulfillment of pollution control requirements", and that departmental contacts at these respective levels may vary depending upon the circumstances involved in the project.

To alleviate some of this confusion, the following section defines the steps required in gaining approvals from regulatory agencies, specifies governmental agencies and their areas of responsibility in public health and environmental protection, and describes a method of preparation of project information for presentation to regulatory agencies when initiating a project.

7.2 Purpose of Regulatory Agency Requirements

The purpose of regulatory agency standards and guidelines is to ensure that a consistent and satisfactory approach towards wastewater treatment is maintained. Specific criteria and guidelines developed by regulatory agencies are intended to assist both the agency and the designer. To the agency, they are means of ensuring a minimum standard of treatment, an effluent of acceptable quality, a methodology in system approval, and a format for report preparation and review. To design engineers, guidelines are useful in determining the scope of the field work to be undertaken, in determining the process or processes most applicable for the area under investigation and in reducing errors of omission.

When reviewing standards or guidelines, it should be noted that there are two basic types of criteria used in Canada for the control of water pollution. These are:

a) criteria dealing with the quality of the wastewater being discharged from a wastewater treatment plant; and
b) criteria dealing with the quality of the receiving water, e.g. lake, river, bay, stream, etc.

The former are known as "effluent criteria", while the latter are known as "water quality criteria". The principal difference between effluent criteria and water quality criteria lies in the fact that the latter take into account dilution and the assimilative capacity of the receiving water, while the former deal specifically with the wastewater being discharged from a plant. In Canada, most of the criteria in use are effluent criteria. However, where a province or region experiences a wide variation in receiving water use, effluent criteria may be adjusted to take into account desired quality of a receiving water body. Criteria, often presented in the form of acceptable hydraulic and organic loadings or pollutant concentrations, are important to project initiators because they establish a minimum acceptable level of quality in design and construction of sewage works.

7.3 The Approval Process

Figure 70 illustrates the possible steps involved in gaining necessary approvals for the construction of sewage works from regulatory agencies. Briefly, it is noted that the process consists of a general planning stage, a preapplication conference with agency staff, the granting of certificates to indicate temporary and final approval of the project, and approval of tendering and construction activities. It is emphasized that the actual mechanics of regulatory agency approval processes will vary from province to province in Canada. The major steps of regulatory agency review procedures are discussed in this section. More specific information on such procedures should be available from local agencies.

7.3.1 Facilities Planning. An initial meeting with the local regulatory agency is of major importance before planning. To prepare for these discussions, applicants must familiarize themselves with the following aspects of their project:

a) Problem identification and delineation of planning area. To establish whether a project will be acceptable, and therefore whether the applicant should proceed to the first stage of the actual approval process, staff of environmental and/or health agencies will wish to discuss the following information in general terms at preapplication conferences:

   i) location of the proposed project,
   ii) proximity of existing and proposed development to the sewage project,
   iii) the possible effect of new areas of development on existing or proposed sewage facilities,
   iv) the point of effluent discharge (if applicable),
   v) the method of sludge handling (if applicable).

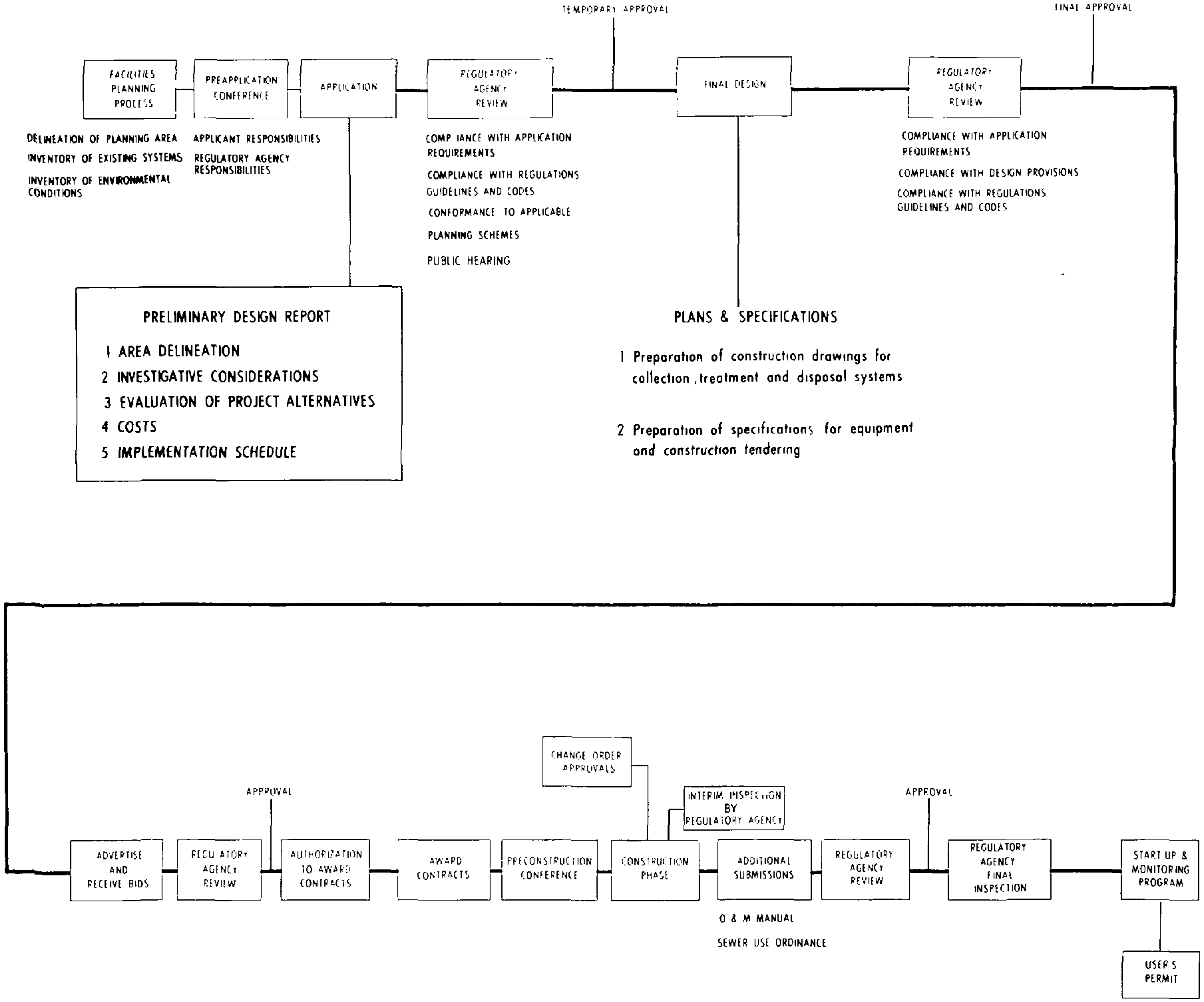


FIGURE 70 SCHEMATIC FLOW DIAGRAM OF PROJECT APPROVAL PROCESS

The first three items deal primarily with the area affected by the proposed sewage project. In defining the planning area, the applicant should be aware of all existing and proposed development, as well as any existing sewage facilities within the area concerned. Should the applicant represent a municipality or region, delineation of the planning area should encompass the entire community, including those areas subject to future development. If the applicant is proposing a major project within a community (e.g., the construction of a housing subdivision complete with sewage facilities), it is essential that the implications of such a project be presented on the basis of the impact on the entire community and the effect, if any, on existing water and sewage facilities in the community.

To assess the effect of a proposed project in an existing community, the applicant must first familiarize himself with the existing system. This is best accomplished through an inventory-type investigation. An inventory will include information on the nature of existing water supply and sewage collection and treatment systems, design capacities, actual flow conditions and any major problems frequently encountered with the systems.

b) Inventory of environmental conditions. To establish a baseline from which to measure environmental effects of proposed wastewater facilities, existing environmental values and resources should be considered in the planning stage of a sewage project. Natural resources and natural beauty, wildlife, recreational areas (e.g., beaches, parks) and historic sites within the boundary of the planning area should be flagged and discussed thoroughly with regulatory agencies at initial meetings.

7.3.2 Preapplication Conference. The type of works for which approval is to be obtained will be varied, and for this reason agency requirements for each application will differ. In some cases, the supporting information which must accompany an application for approval will be extensive and detailed, while in others, limited data will be required. It is essential that project initiators discuss their proposals for sewage works with regulatory agency personnel prior to preparing any reports or plans.

Topics of interest to regulatory agency personnel at these initial meetings have been discussed in section 7.3.1. Project initiators must determine the actual steps involved in gaining approval for their proposal, the standards and/or guidelines under which they must design and construct their proposed sewage works, and appropriate point(s) of contact within the regulatory agency throughout the project.

7.3.3 Temporary Approval. A temporary approval, sometimes referred to as an approval in principle, is required by a project initiator prior to proceeding with formal planning and design of sewage works. In some cases, as determined at preapplication conferences, a temporary certificate may be obtained simply by filling in an application form after initial discussions with approval agency staff (e.g., projects such as minor extensions to existing sewer systems or installation of a septic tank system at an individual home). Otherwise, if the proposal involves major works, agency personnel will require the applicant to develop a formal proposal for submission along with the application for temporary approval.

A formal proposal, although varying in the degree of complexity from agency to agency and from situation to situation, may be a preliminary design report. Without limiting the scope of this preliminary report, it may contain a discussion of the points itemized by the regulatory agency, as well as pertinent information on capital costs and operation and maintenance requirements. In some instances, the proposal may be subject to public scrutiny (i.e., public hearings) and in such cases it is advisable to include viable alternatives to the proposed works, as well as capital and operating costs of the alternatives. An example of the type of information required in a detailed preliminary design report is presented in Section 7.5.

Upon receiving temporary approval from the regulatory agency (after review of the application form and the supporting document(s)), applicants may proceed to the next step in the process: formal planning and design of sewage works, and application for final approval.

7.3.4 Final Approval. To obtain final approval, an application must be submitted to the regulatory agency along with adequate supporting information. The information required to constitute a complete application does, of course, vary depending on the nature and extent of the proposed works. For the most part, applications for final approval will fall into three general categories:

a) sewers (sanitary, storm, combined),
b) pumping stations,
c) treatment works (including treated effluent and sludge disposal).

The support information required for each of these categories will be specified by the regulatory agency and could include such items as background information on the population to be served within the design life of the works, sewage characteristics (hydraulic, biological, chemical), detailed drawings (civil, mechanical, electrical), design criteria, equipment specifications, unit capital costs, projected annual costs, etc. Where treated liquid is being discharged to a receiving stream, information concerning the point of discharge, stream flow data, water use, and chemical and biological characteristics may be necessary. If a spray irrigation system is proposed, evidence could be required to show that the soil and vegetation are suitable to accept proposed application rates without runoff or significant ecological effect. Where liquid or filtered sludge is to be hauled away from the site for final disposal, evidence may be necessary to indicate that satisfactory disposal sites are available.

Final approval certificates are granted when the pollution control agency has reviewed the application and agrees with submitted plans and specifications. Certificates may be issued with the condition that the approval is valid and in force for a finite period following the data of issuance. If construction of the sewage work has not commenced within this period, the certificate may be considered void and a new application may be necessary.

7.3.5 Construction Inspection and Start-Up of Works. During construction of approved sewage works, the control agency often will send out district officers for an on-site inspection. Upon completion of construction, it is the usual practice of regulatory agencies to appraise the treatment plant by monitoring plant efficiency and assessing the overall performance of the installation. Issue of a user's permit (permit to operate) may then be authorized by the agency with the terms and conditions of plant operation outlined therein.

7.4 Provincial, Territorial and Federal Pollution Control Agencies

7.4.1 Provincial Regulatory Agencies. The responsibility for the prevention and control of water pollution within many provinces in Canada is shared by two agencies, namely the health department and the environmental agency. Specific areas of responsibility may be defined as follows:

a) Provincial and municipal health departments. The early legislation pertaining to water pollution control was contained in public health acts and was administered for the most part by provincial health departments. Recent enactments of environmental legislation and the formation of environmental agencies has largely centralized within a single administrative agency all environmental activities, including water pollution control. With the exception of a few provinces private sewage disposal systems are at present the only systems remaining under the jurisdiction of or active surveillance by some provincial health departments. Although a nationally agreed definition of private sewage disposal systems does not exist in Canada, such treatment systems may be defined as systems that are privately-owned and treat wastes from privately-owned buildings or structures on-site, and have no direct discharge into a centralized collection system or treatment works. Private treatment systems may include any of the following groups:

- self-contained toilets (chemical, compost, incinerator, recirculating, portable, etc.),
- leaching pits,
- cesspools,
- septic tank - tile field systems,
- holding tanks,
- hauled sewage systems,
- package treatment plants discharging to subsurface disposal systems.

An individual wishing to construct, install, expand or repair a private disposal system should contact the local health officer or inspector to determine whether a health permit or a permit from another agency is required.

b) Provincial environmental agencies. Before a municipality, company or individual proceeds with construction of a wastewater treatment system, approval from a provincial environmental agency must be obtained. Approval of private systems, as discussed in the previous subsection, may indeed come under the authority of the provincial health department in some provinces, and this should be clarified early in the project. Otherwise, the provincial environmental agency is the foremost contact when initiating a sewage works project.

The approval mechanism for the installation of sewage works varies from province to province, but generally follows the steps previously suggested.

7.4.2 Territorial Regulatory Agencies. The territorial governments have the mandate to upgrade community wastewater treatment systems and provide financial assistance to all communities, whether by subsidy or direct grant. In addition, the governments are available to assist in the selection of consulting engineers to undertake wastewater treatment studies in northern communities. Such requests usually come from virtually all communities except those well established and organized.

When an engineering study is undertaken in a northern community, the terms of reference for the study usually require the consultant to undertake a broad environmental investigation, covering the review and analysis of water supply, water services, sewerage and sewage treatment systems, and solid waste handling and disposal systems. The consultant must make recommendations for all of these items since most of the northern communities are too small to require the full-time services of professional engineers. Upon completion, the consultant's report is reviewed by the territorial government, community council, and such federal agencies as the Department of Indian and Northern Affairs, the Department of Environment, and the Department of National Health and Welfare. If the consultant's recommendations are accepted the report will form the basis upon which the territorial government will fund the design and construction of the proposed sanitation systems.

The consultant may become involved in technical deliberations with the Yukon or Northwest Territory Water Board when, as a result of his report, a license has to be issued to make use of waters in the territories.

7.4.3 Federal Regulatory Agencies. In the federal government, there are two agencies administering statutes relating to water pollution control for land installations.

a) Department of the Environment. The Federal Department of the Environment (DOE) has the mandate to clean-up and control pollution from all land-based installations. However, in most cases in-depth approval is conducted by the provincial environmental agencies and the federal role remains an audit function. Exceptions occur when the following circumstances prevail:

   i) the final effluent from the works will be discharged to an interprovincial or international water body,
   ii) the discharge poses a potential threat to a fisheries or shellfish resource,
   iii) no active provincial programs are currently underway,
   iv) the discharge poses a potential health threat,
   v) the discharge occurs within an area bounded by federal property or from an installation owned and/or operated by the federal government.

   The Environmental Protection Service (EPS) of DOE is responsible for water pollution control in a national context. EPS, through its regional office staff, conducts thorough reviews of proposed works in many areas of the country, particularly in the above circumstances. Prevention and control of water pollution at federal installations is administered by the Federal Activities Branch of EPS. This branch assists with the clean-up of existing federal facilities and ensures that planned federal installations are designed and operated according to best practicable technology and level of operation. A document entitled "Guidelines for Effluent Quality and Wastewater Treatment at Federal Establishments" has been prepared by FAB and is available at all EPS Offices (Appendix G).

It is the policy of DOE to consult and maintain close liaison with provincial governments in projects of mutual concern, and attempt to resolve with its provincial counterparts, questions pertaining to licensing, certification and other matters arising from the construction and operation of wastewater treatment systems. For federal projects, EPS is the focal point for liaison with provincial and territorial departments.

b) Department of Indian and Northern Affairs. The Northern Inland Waters Act and sections of the Arctic Water Pollution Prevention Act authorize the Department of Indian and Northern Affairs to manage the use of all waters in the Yukon and Northwest Territories. Water Boards have been established in each territory by the Department of Indian and Northern Affairs to license those wishing to use territorial waters. The territorial government is generally responsible for community wastewater treatment but through the water use licence application and public hearing process the Water Board is also involved. DOE has a member on each of the Water Boards and there are usually discussions between the technical staffs of the Water Board and EPS to select licence conditions.

For further information, the reader is advised to contact the agencies listed in Appendix B.

7.5 Format and Contents of a Preliminary Design Report

Submission of a preliminary design report to a regulatory agency is a critical step in an endeavour to gain regulatory agency approval for construction of a new or modified wastewater collection, treatment and/or disposal system. Since acceptance of the preliminary report is the regulatory agency's indication to commence with preparation of final plans and specifications, it is essential that specific information be incorporated into the document. Regulatory agencies, in all cases, will inform applicants of the data they require to review a proposal and grant approval certificates. The following report outline contains a list of items which may be required in a preliminary design document, as well as detailing a suggested format for such a document. Although it is unlikely that applicants will frequently be required to present data on all items listed, portions of this list may prove useful.

Report Cover

Preliminary Design Report, Sewage Works Proposal

Letter of Transmittal

A one page letter bound into the report and including:

- Submission of report to the approval agency,
- Statement of feasibility of recommended project,
- Acknowledgement to those giving assistance.

Title Page

Title of project

Municipality, county, concession and lot numbers (as applicable)

Name, and address and telephone number of individual (firm) preparing report

Table of Contents

Section headings, chapter headings and subheadings

Maps

Figures

Appendices

Number all pages and cross-reference by page number

Summary

Highlight, very briefly, what was found from the investigation:

Findings.

Population - present, design (when), ultimate.

Land use and zoning - portions residential, commercial, industrial, greenbelt, etc.

Wastewater characteristics and concentrations - portions of total hydraulic, organic, and solids loading attributed to residential, commercial and industrial fractions.

Collection systems projects - immediate needs to implement recommended project, deferred needs to complete recommended project, and pump stations, force mains, appurtenances, etc.

Selected treatment process - characteristics of process and characteristics of output.

Receiving waters - existing water quality and quantity, classifications, and downstream water uses, and impact of project on receiving water.

Proposed project - total project cost, total annual expense requirement for: debt service; operation; personnel.

Introduction

Background

Reasons for report (approval process).

Scope

Guidelines for developing the report, including preliminary design of selected system and costing.

Purpose

Presentation of appropriate past history.

General

Existing development, expansion, annexation, intermunicipal service, ultimate development.

Drainage basin, portion covered.

Population growth, trends, increase during design life of facility

Residential, commercial and industrial land use, zoning, population densities, industrial types and concentrations if applicable)

Topography, general geology, and effect on project.

Meteorology, precipitation, runoff, flooding, etc., and effect on project.

Identification of environmentally sensitive conditions within the planning boundary including areas of natural beauty, wildlife, recreational areas and historic sites.

Regulations Guidelines and Ordinances

Presentation of applicable regulations, guidelines, and ordinances.

Sewer use ordinance (if applicable).

Existing contracts and agreements.

Surcharge rates and basis for surcharge (in effect).

Enforcement provisions including inspection sampling, detection, penalties, etc.

Existing Facilities Evaluation

Existing Collection System

Inventory of existing sewers.

Isolation from water supply wells (if applicable).

Structural condition, hydraulic capacity of existing system.

Determination of collection system problems - overflows, infiltration, exfiltration - by gauging and/or studying water supply relationship.

Outline repair, replacement and storm water separation requirements and/or possibilities

Establish renovation priorities.

Present recommended program to renovate sewers.

Estimate required expenditure to renovate sewers and prepare implementation outline.

Existing Wastewater Treatment Site

Area for expansion.

Terrain.

Subsurface conditions.

Isolation from habitation.

Isolation from water supply structures.

General aesthetic appearance, odour problems.

Flooding.

Existing process facilities

Capacities and adequacy of various treatment components.

Relationship and/or applicability to proposed project.

Age and condition.

Adaptability to different usages.

Structures which could/should be retained, modified or demolished.

Existing Wastewater Characteristics

Water consumption (from available records).

Wastewater flow pattern, peaks, total, design flow.

Physical, chemical and biological characteristics and concentrations.

The Existing Receiving Body of Water

Receiving water base flow (as specified by regulatory agency).

Characteristics of receiving water (as specified by regulatory agency).

Downstream water uses.

Impact of proposed discharge on receiving waters effluent objectives.

Flagging of environmentally sensitive conditions (e.g., shell fish beds, spawning grounds, beaches).

Evaluation of Project Alternatives

Regionalization

Consideration should be given to regional sewage facility planning whenever feasible. Municipalities (or private installations, institutions, etc.) may join together in co-operative regional treatment systems provided costs, as related to collection, treatment and disposal, as well as operation and maintenance commitments, are jointly shared and deemed favourable by the parties involved. Detailed investigation of regional schemes should be undertaken if cursory review indicates project feasibility. Comparison of regional versus non-regional solutions should be made and the optimum scheme selected on the basis of this comparison.

Community Systems

Site Requirements

Comparison of advantages and disadvantages of locating proposed treatment works at either existing treatment site or new site(s) based on the following criteria:

- land availability (zoning, local ordinances, etc.) and soils,
- expansion possibilities,
- hydraulic requirements,
- energy requirements,
- flood control,
- accessibility,
- aesthetics (odour problems, landscaping, etc.),
- protection afforded to public health and the receiving environment (environmental sensitivity, etc. of receiving environment),
- public opinion,
- costs (land, services).

Selection of Preferred Site

Justification of selection based upon economic feasibility, environmental compatability, compliance with applicable ordinances, regulations and guidelines.

Collection System Alternatives

Inventory of proposed additions.

Consideration of area of service.

Unusual construction problems.

Alternate Collection System Costs

Flow/waste reduction considerations.

Selection of preferred collection method and/or combination of collection methods.

Justification of selection based upon economic feasibility, environmental compatability, compliance with applicable ordinances, regulations and guidelines.

Treatment Process Alternatives

Tabulation of required plant performance based upon receiving water quality criteria and effluent criteria.

Description and delineation (line diagram) of applicable treatment alternatives.

Discussion of advantages and disadvantages of each (e.g., reliability, flexibility, complexity of operation, resource requirements, operation and maintenance requirements, aesthetics, adaptability to future needs, availability of power).

Characteristics of process output (e.g., continuous or seasonal effluent discharge, volume and stability of waste solids).

Comparison of process performances.

Estimates of alternate process costs including capital, construction, installation and operation and maintenance estimates (all costs on an annual basis, i.e. amortized capital and O&M).

Selection of preferred process.

Justification of selection based upon economic feasibility, environmental compatibility, water quality objectives, compliance.

Ordinances, regulations and guidelines.

Preliminary Design of Process Facilities

Discussion of design criteria (average daily flow, flow variances, waste characteristic, effluent quality, treatment efficiency, design life).

Sizing of collection system and treatment process components on basis of above discussion and design parameters.

Liquid effluent disposal requirements.

Waste solids processing and disposal requirements, support equipment and facilities (flow measurement devices, sampling equipment, process control, work and laboratory area, and laboratory equipment, safety equipment).

Cost Summary and Implementation Schedule

Tabulation of capital expenditure and annual operational and maintenance requirements (capital expenditure broken down into unit costs for various components of collection, treatment and disposal system, and amortized over design life of system (20 years).

Implementation schedule, including immediate and deferred construction (and effect on costs), interruption of existing utilities and traffic interference, restoration of pavements, lawns, etc., operation of existing treatment and disposal facilities during construction period, scheduling of design and construction phases.

Appendices: Technical Information and Design Criteria

Raw Wastewater Characterization Program

Description of program including location of monitoring stations, duration of study, type of sampling equipment utilized, frequency of sample collection, sample storage, method of sample analyses, and analyses performed.

Tabulation of data.

Data analysis including diurnal flow variations, organic loadings, nutrient concentrations.

Collection System Design

Design tabulations - flow, size, velocities, etc.

Pump station calculations.

Special appurtenances.

Construction problems.

System map (report size).

Process facilities design.

Criteria selection and basis.

Hydraulic and organic loadings - minimum, average, maximum and effect.

Unit dimensions.

Rates and velocities.

Detention.

Concentrations.

Recycle.

Chemical additive control.

Physical control.

Removals: effluent concentrations, etc. Include a separate tabulation for each unit to handle solid and liquid fractions.

Process Diagrams

Process configuration, interconnecting piping, flexibility, etc.

Hydraulic profile.

Solids control system.

Flow diagram with capacities, etc.

Operation and maintenance.

Routine and special maintenance duties.

Process control and laboratory analysis.

Time requirements.

Tools, equipment, safety, etc.

Personnel requirements - number, type, qualifications, salaries, benefits (tabulate).

Chemical Control

Processes needing chemical addition.

Chemicals and feed equipment.

Tabulation of amounts and unit and total costs.

Support Data

Outline unusual specifications, construction materials and construction methods.

Maps, photographs, diagrams (report size).

Other.

8 SELECTION OF SMALL WASTEWATER TREATMENT SYSTEMS

8.1 Introduction

Sewage treatment systems serving small communities present unique design problems. Frequently those charged with design are not specialists in sewage treatment and are not aware of the applicable government regulations, the operational problems, or the design constraints of the systems. Even with a knowledge of capabilities and limitations of alternate processes, the selection of the most appropriate treatment system is a challenge for experienced designers.

While it is not possible to provide an explicit selection procedure, this section summarizes many of the parameters and variables that a designer must consider in the selection process. The parameters have been categorized into six basic groupings.

a) Treatment Alternatives
b) Site Characteristics
c) Effluent Requirements
d) Raw Waste Characteristics
e) Operational Characteristics
f) Cost Effectiveness

The list of selection requirements which follows describes the extent to which various treatment processes will meet specific requirements. After the requirements for specific site treatment and the effluent have been identified, the designer can examine the compatibility of the various processes. By proceeding through the list, incompatible processes may be eliminated, considerably reducing the problem of selection.

After the number of viable alternatives have been established, preliminary design calculations, based upon the design parameters in Section 4, will allow a cost comparison to be made which will lead to the final selection.

In some instances, various treatment alternatives might appear to be incompatible with site conditions. However, one should be cognizant of the fact that appropriate engineering design can overcome existing deficiencies and provide a more appropriate long-term solution to a waste management problem. For example, leaching beds for septic tanks can be located in areas where soil cover is unacceptable by building a raised pervious bed. Similarly, although aerated lagoons are indicated to be temperature-sensitive and consequently could provide less than satisfactory treatment in cold weather, several have been installed in the Canadian Arctic.

Following is a detailed listing of selection considerations which may be used as a national approach to evaluating a specific site. An in-depth case history is included to illustrate the selection process.

8.2 Selection Requirements

Parameter	Required Information	Reference
1. Treatment Alternatives		
a) Planning	It is important that regional or municipal planning boards be consulted to review the desirability of regional or central treatment schemes versus on-site treatment for the specific application.	Section 7
	If on-site treatment is contemplated, the legal and governmental steps leading to construction approval (i.e., new system or extension of existing facility) must be determined.	Section 7, Appendix B
b) Existing Services	The existence of facilities that could be utilized or extended to meet requirements should be investigated. The review should include applicability, desirability and availability.	Sections 7.2 and 7.5
2. Site Characteristics		
a) Topography	A topographical map and site inspection to determine the general lay of the land are required. These will help to eliminate certain alternatives and will affect the choice of a collection system if required. Drainage should be investigated.	Section 6
b) Soil Conditions	A soils map must be obtained or an actual soils survey conducted to determine permeability, ease of excavation and location of	Sections 3 and 7

Parameter	Required Information	Reference
	any impervious layers (rock, clay, etc.).	
c) Water Table	The maximum water table elevation must be determined. Location of wells, springs, and surface waters should be charted. Protection of the potable water supply is a primary concern.	Sections 3 and 7
d) Climate	The meteorological history of the area should be obtained. Winter and summer temperature extremes should be noted since severe heat or cold may cause process efficiency to decrease for several alternatives. Annual precipitation should also be obtained.	Sections 4 and 7
e) Land Area Requirement	Area constraints should be determined as this may eliminate some alternatives. Included in this would be a consideration of aesthetics and adjacent land use.	Sections 4 and 7
3. Effluent Requirements		
a) Disposal	The feasibility of the disposal methods should be evaluated in view of site considerations and receiving water availability. The choices are basically surface (land or water) and sub-surface.	Sections 3 and 5
	Surface Land	Section 5.2
	Surface Water	Section 5.1
	Sub-surface Land	Section 3.4
b) Further Treatment Prior to Disposal	The availability of an authorized site for disposal of waste accumulations from holding tank and septage sludge haulage systems must be investigated.	Section 3
c) Characteristics for Discharge	Federal and/or provincial regulations, guidelines or objectives should be obtained.	Section 7

8.2 Selection Requirements (Continued)

Parameter	Required Information	Reference
	These should be compared to the expected effluent quality from each process alternative.	Sections 4 and 5
4. Raw Waste Characteristics		
a) Sources of Waste	A complete inventory of all wastewater sources contributing to the total flow is required. For an existing system, a review of the treatment facilities and/or collection systems is necessary to determine additions or changes required. A sampling or monitoring program is an integral part of this inventory. For a new system, the proposed individual sources of waste are used to estimate organic and hydraulic loadings.	Section 2
b) Hydraulic Loading	The actual hydraulic load in total daily or annual flow, must be determined. As with organic loading this is determined by i) sampling and flow measurement programs for and existing system or ii) empirical estimates by source for new systems.	Section 2.2
	Equal in importance is the flow variability on a daily or yearly basis. Some systems are better able to handle the various possibilities of continuous, cyclic or intermittent flows.	Section 4
c) Organic Loading	The design of any selected system will require an estimate of the organic loading. At this point it is important to note any industrial input to the system as this input could be the source of toxic substances or non-domestic organics and inorganics. For an existing system, the loading could	Section 4

8.2 Selection Requirements (Continued)

Parameter	Required Information	Reference
	be determined through analysis of collected samples. For a new system, estimates are made from empirical values, tabulated by type of source.	
5. Operational Characteristics		
a) Complexity	Certain process alternatives are more complex in process design and operation than others. The availability of skilled operators for such schemes in the site vicinity should be determined along with the economic implications.	Section 4
b) Consistent Effluent Quality	The desirability or necessity of a consistent effluent quality should be determined. Most well operated and designed systems are reliable in this respect. However, costs may be decreased if periodic poor effluent quality can be tolerated.	Section 4
c) Effects of Shock Loads	The probability and frequency of shock loadings (hydraulic and/or organic) should be determined. Where shock loading is a significant factor the selection of a system with built-in resistance is an asset.	Sections 2 and 4
d) Sludge Disposal Requirement	Primary, secondary and physicochemical systems normally required regular disposal of sludge. The availability of suitable sites should be determined. Disposal problems are eased if the sludge is stabilized.	Section 4.5
	Others forms of treatment require regular or periodic disposal of accumulated waste, for example, pump-out of a septic tank. Provision for disposal of this material should be made.	Section 3.7

8.2 Selection Requirements (Continued)

Parameter	Required Information	Reference
e) Flexibility for Higher Loadings	The long-term expansion potential of the community or site must be estimated and should include population and industrial growth forecasts. The ability of an alternative to be modified to an increased loading in foreseeable future should be considered. This is important in the design of collection systems.	Section 6
f) Equalization Provided	The alternatives should be analyzed to determine their ability to provide equalization at time of high peak flows on an hourly basis. This might occur during large storms. It should be determined if this is a likely eventuality for the site.	Section 4 and 6
6. Cost Effectiveness Analysis		
a) Capital Costs	Capital costs of the various treatment alternatives will vary over a wide range. Construction and installation costs should also be included in the cost analysis. Alternatives selected as viable must be costed as this will be a major function in ultimate selection. Preliminary design is therefore required to complete this analysis.	Section 7.5
b) Operating Costs	The routine annual costs of a treatment system, including operation, maintenance and depreciation must be evaluated and should be capitalized over the life expectancy of the facility.	Section 7.5

8.3 Case History - Provincial Park Treatment Facility

8.3.1 Introduction. A parks commision in Ontario wishes to proceed with plans to construct and operate a tent and trailer camp on a piece of land to be called Riverfront Park. The camp is to be located in Lot H, Concession 1, Township of Charlotte in Glen County. The site plan for the 32 hectare park is presented in Figure 71.

Plans for the camping facilities call for the following:

a) 100 campsites: 40 tent sites, and 60 trailer sites;
b) a central comfort station providing toilet, washroom and laundry facilities for all campers (i.e., no direct trailer hook-ups);
c) a trailer dump-out facility located adjacent to the comfort station;
d) provision for the operation of a swimming pool;
e) seasonal operation from mid-May to mid-October annually.

The operation of the comfort station and dump-out facility necessitate the consideration of a treatment system for the disposal of sanitary wastes.

8.3.2 Treatment Alternatives. The park site is located approximately 32 km from a major city and approximately 2 km from a small village. The collection system for the city's sewage treatment plant does not extend as far as the park. Incorporation of the park into a regional sewer system is deemed impractical and uneconomical. The village on the other hand is served by septic tanks and tile fields. As a result of these considerations, on-site collection of park wastes is indicated.

With this decision made, treatment and disposal options must be addressed. Basically, two options exist: an on-site treatment facility with local discharge of effluent, and on-site collection with haulage and ultimate disposal of wastes to an authorized disposal site.

On-site treatment facilities require some form of ultimate disposal or discharge of liquid effluent. The discharge options are sub-surface (land) and surface (land or water). Under Part 7 of Ontario's Environmental Protection Act, subsurface discharge disposal systems for the treatment of sewage fall under the jurisdiction of the Ministry of the Environment. Under the Water Resources Act, surface discharge of liquid effluent must also be approved by the Ministry of the Environment. Park officials contacted the Ministry of Environment for advice and direction.

Preliminary planning by park officials indicated that there were no existing facilities that could be extended or utilized practically or economically to meet the requirements of the park.

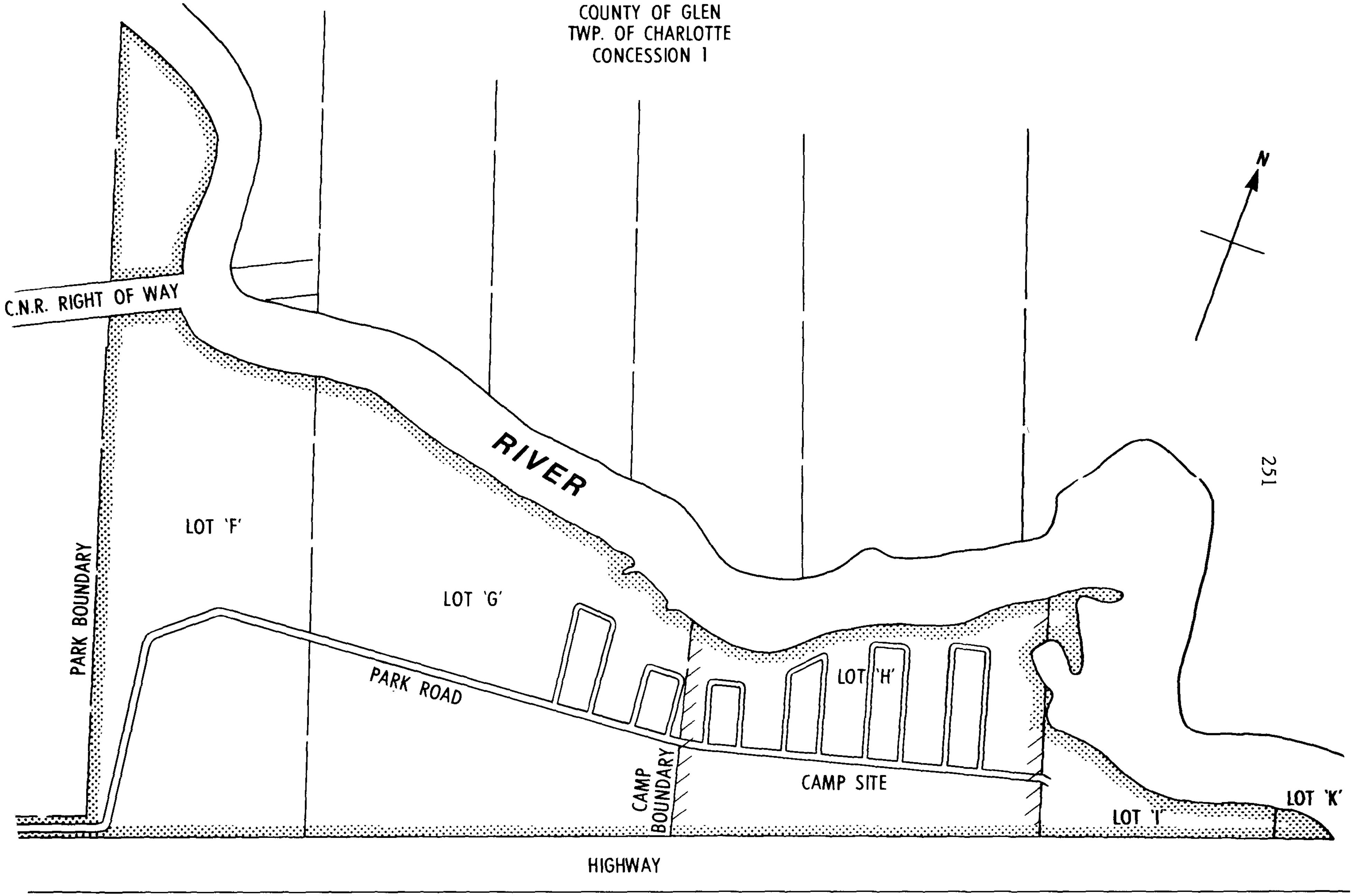


FIGURE 71 SITE PLAN FOR CASE HISTORY

8.3.4 Site Characteristics. The park will be located on a river approximately one mile upstream from its confluence with the St. Lawrence River. The park site is heavily wooded and possesses significant aesthetic value in its present, natural state. The local river is widely used for recreational purposes including boating, fishing and swimming. There are also a number of cottages in close proximity to the proposed park area. Fish species utilizing the resources of the river for spawning and general habitat include yellow pickerel, yellow perch, bass and bullhead. Protection of area aesthetics and recreational resources are considered to be a great importance in the selection of a treatment system to service the campsites.

a) Topography. The area topography is rated as smooth and level to gently rolling. Elevation at the site is less than 53 m above mean sea level. Local relief is virtually non-existent.

b) Soil conditions. A soils map of the area indicates that the soil in the majority of the park is a fine, sandy loam of the l'Achigan Series. This series is the imperfectly drained member of the St. Thomas catena. This soil develops from sorted outwash materials on level to gently sloping topography. An undisturbed profile is described below:

A_0	-	5-0 cm of partially decomposed vegetable matter.
A_1	-	0-7.5 cm of very dark grey (10YR 3/1) fine sandy loam; friable; fine crumb structure; pH - 5.0.
A_2	-	7.5 - 12.7 cm of grey (10YR 6/1) fine sand; single grain structure; pH - 4.8.
B_2	-	12.7 - 25 cm of yellow-red (7.5YR 5/6) fine sandy loam; some mottling; friable; weak crumb structure; evidence of weak ortstein formation may be found in some areas; pH - 5.4.
B_3	-	25 - 46 cm of yellowish brown (10YR 5/4) fine sand; very friable; single grain structure; pH - 5.6.
C	-	Light brownish grey (10YR 6/2) fine sand; pH - 5.8.

This soil layer can be approximately one metre in thickness above the underlying clay.

As mentioned above, surface drainage through this soil is imperfect. The soil moisture content indicates a very slight moisture deficit in general with short periods of saturation of soils (less than 120 days). Due to the low natural fertility and low organic content of this L'Achigan fine sandy loam, artificial drainage improvement is seldom warranted and agricultural activities are generally restricted to hay or pasture production. This soil is, however, more useful in producing trees.

c) Water table. Ministry of the Environment personnel report that the geology of the site consists of limestone bedrock overlain by 12 to 15 meters of hard clay containing odd layers of gravel. Above the clay is a thin surface layer of soil which was discussed in the previous section.

Most wells in the area are drilled wells. The depths range from 15 to 24 m and thus are utilizing the bedrock aquifers as a water source. Water quality reduces with depth into bedrock as increases in hydrogen sulfide, salts and iron occur. Thus, most of the drilled wells are confined to the upper layers of bedrock.

The groundwater table in the area is quite high, at times approaching within 30 cm of the surface. However, the level fluctuates greatly throughout the year and therefore there are few shallow or dug wells.

d) Climate. The climate of the area is classified as moderate due to the influence of the Lower Great Lakes. Annual precipitation ranges from 81 to 91 cm, of which approximately 25 cm, reduced to rain equivalent (10 cm of snow to 1 cm of rain), falls in the form of snow. Since the park will be operated only from mid-May to mid-October, the winter climatic extremes of the area will not be a factor in treatment system selection.

During the period of park operation, mean daily temperatures will generally range from 10°C to 20°C with maximums up to 35°C. Thunderstorms dropping significant quantities of rain in a small period of time are not uncommon in the summer. For open-type treatment systems such as lagoons, the hydraulic surge from a severe rainfall could be a significant factor.

e) Land area requirement. The size of the park is 32 hectares (80 acres) of which the majority is heavily wooded. The park authorities wish to maintain the high aesthetic value of the site and, therefore, the characteristics of the selected treatment system must be consistent with minimum disturbance of the park resources. Systems requiring a small cleared area would be preferred.

8.3.5 Effluent Requirements.

a) Disposal. Because of the high water table, impermeable soils and poor drainage of the area, the applicability of a septic tank system or other sub-surface treatment process, such as aerobic tanks and leaching cesspools, would be restricted. Site limitations could be overcome by a raised tile bed and other design modifications. However, Ministry of Environment officials indicated that the impact of odour control chemicals, which are used in the holding tanks of trailers and campers, on the physical, chemical and biological processes which occur in a subsurface disposal system was a potential problem. Given this consideration, as well as the fact that the bed area would be of the order of 0.5 ha in size, this alternative was dismissed as an option.

Of the two methods of surface discharge available, surface water discharge to the local river was judged to be the most attractive and economical. Surface land disposal by spray irrigation was not considered in detail by park officials because of the additional equipment and man-hour requirements and the necessity to obtain an irrigation site within or close to the park.

b) Further treatment prior to disposal. The adoption of a storage/haulage system would require an authorized treatment site for ultimate disposal. The neighbouring city's treatment plant was 32 km from the park site. Preliminary consideration of haulage deemed it to be unsatisfactory as a long term final solution.

c) Characteristics for surface water discharge. There are no specific regulations in Ontario for discharges from sewage treatment plants with respect to individual parameters, except for phosphorus discharges to the Great Lakes. Objectives for BOD and suspended solids for discharge to receiving streams are 15 mg/L. The phosphorous regulation of 1 mg/L is not applicable in this particular situation because of the size of the plant (less than 4554 m^3/d) and the fact that the ultimate receiver, Lake St-Francis, is not part of the Great Lakes system.

Regional staff of the Ministry of the Environment investigated the assimilative capacity of the local river with regard to the BOD loading from the park's treated effluent. During the summer months the Conservation Authority on the river maintains the flow in the river via a 0.6 m^3/s (20 cfs) diversion. This flow was judged sufficient to assimilate expected BOD loads. If chlorine is used to disinfect the wastewater, river flow would also be sufficient to dilute residual chlorine in the effluent to below 0.02 mg/L most of the time. The chlorine level is significant with respect to toxicity considerations since the river is a productive spawning habitat for several fish species.

8.3.6 Interim Conclusions. On-site treatment and disposal of the park's sanitary wastewater is favoured by the Parks Commission and the Ontario Ministry of Environment. The central comfort station and trailer dump-out station are the only sources of waste, and a collection system would be simple and inexpensive.

A holding tank/sewage haulage system is judged unattractive by the Parks Commission. Soils and groundwater conditions in the area are not conducive to the operation of a standard septic tank - tile field system. An elevated tile filed has been considered as an alternative to the standard subsurface soil disposal system, but the potential risk of system failure due to unknown reactions with odour control chemicals eliminated further consideration of this alternative.

The disposal of treated wastewaters via surface discharge was reviewed in terms of two alternatives: spray irrigation onto land; and discharge to surface waters. The spray irrigation alternative was dismissed by parks officials as unfeasible because of the additional equipment and man-hour requirements. Discharge of treated effluent to a body of water recognized as a recreational and sport fisheries resource deems that the effluent be of an acceptable quality, i.e., in the 15 mg/L range for BOD and SS. This effluent objective precludes the use of a primary sedimentation treatment process or an aerated lagoon.

8.3.7 Raw Waste Characteristics.

a) Sources. The park consists of 100 campsites of which 60 are for trailers and 40 are for tents. There is a central comfort station including toilet, washroom and laundry facilities. There is also an adjacent trailer pump-out facility to be located some 90 -120 m from the comfort station.

b) Hydraulic load. The flow would be cyclic on a daily basis and intermittent on a yearly basis.

The estimated loading from the comfort station, as stipulated by the Ministry of Environment is:

365 L/site-day x 100 sites = 36 500 L/d

(80 gal/site-day x 100 sites = 8000 gal/day).

The loading from laundromats is estimated to be 1820 L per machine per day (400 gallons per machine per day). For two machines, the additional hydraulic input would be:

1820 L/machine-day x 2 machines = 3640 L/d

(400 gal/site-day x 2 machines = 800 gal/day).

The volume of a single pump-out of a trailer holding tank, including rinse, is estimated to be 180 L (40 gallons). At a rate of one pump-out per trailer site per day, the additional hydraulic input would be:

180 L/pump-out x 10 pump-outs/day = 1800 L/d

(40 gal/pump-out x 10 pump-outs/day = 400 gal/day).

The maximum hydraulic loading on the system is estimated to be:
36 500 + 3640 + 1800 = 41 940 L/d.

c) Organic load. Estimates of the load from the comfort station would be:

0.05 kg BOD/person•d x 3 persons x 100 sites = 15 kg BOD/d

(0.12 lb BOD/person/day x 3 persons x 100 compsites = 36 lb BOD/day).

The strength of laundry wastewater varies considerably. However, the BOD has been estimated to be in the order of 200 mg/L. Therefore, the anticipated organic input from the laundry would be:

200 mg/L BOD x 3640 L/d = 0.73 kg BOD/d
(200 mg/L BOD x 800 gal/day = 1.6 lb BOD/day).

The BOD contribution from the trailer "pump-out" is based on the empirical rate of three people per trailer and a normal holding capacity of three days. On a total daily basis, 10 pump-outs per day has been assumed. The additional loading would be:

0.05 kg BOD/person/d x 3 persons/trailer x 10 pump-outs/d = 1.5 kg BOD/d

(0.12 lb BOD/person/day x 3 persons/trailer x 10 pump-outs/day = 10.8 lb BOD/d).

The maximum BOD loading on the system is estimated to be:
15 + 0.73 + 1.5 = 17.23 kg BOD/d.

8.3.8 Interim Conclusions. The estimated hydraulic loadings represent a further limitation to the selection of holding/haulage treatment alternatives. Large holding tanks and/or frequent removal of accumulations would be required. Individual compost, incinerator or chemical toilets are not practical for the steady use made of the facilities in a central comfort station.

8.3.9 Operational Characteristics

a) Complexity. The park is located in a fairly populated area. The Parks Commission plans to provide operation manpower from the park's work force if this is required for the eventual selection. Employment of a previously trained operator is not foreseen. Therefore, the chosen process must be simple to operate and free of mechanical intricacies.

b) Consistent effluent quality. Consistent effluent quality is an important consideration for a surface discharge system because treated effluent will be directed to a recreational water body. Thus, the chosen treatment process must be reliable.

c) Effects of shock loads. The potential for hydraulic and organic shock loadings is an important consideration at this facility because of the variability the number of campers that can be on-site at any time. Resident fisheries and recreational (water contact) activities on the river could be seriously affected by poor quality effluent discharges. The impact that odour control chemicals (found in trailer wastewater dumpings) have on the performance of biological treatment process is a further concern. Shock loadings of these chemicals to a small biological treatment plant could affect the biodegradation of organic material, reduce the biomass oxygen uptake rate, lower the settleability of sludge and/or discolour the effluent from the system. In general, the quality of effluent discharged to the river could be seriously affected.

Given these conditions, it is evident the park requires a treatment system that is resistant to hydraulic and organic shock loads. In addition, the system should be designed such that trailer wastewaters are: i) segregated from other camp wastewaters and separately disposed of at an authorized disposal site or treated in a large municipal sewage treatment system; ii) stored in an on-side holding tank and metered into a treatment system at a rate which allows the biomass to assimilate the chemical additives; or iii) buffered by a sufficiently large volume of wastewater so that the inhibitory affect of the chemical additives will have negligible impact on the treatment process.

d) Sludge disposal requirement. This is not considered a major problem since the amount of sludge will be small. Stabilized sludge could be disposed of at authorized disposal sites used by septic tank cleaning operators in the area or treated at a large municipal treatment plant.

e) Flexibility for higher loads. Extensive future growth of the park is not likely, although some area to the west of the proposed campsites is available for development.

f) Equalization provided. The selected system should be designed to withstand periodic surges from the comfort station.

8.3.10 Interim Conclusions. The following biological treatment processes can be eliminated from further consideration because of the cyclic and intermittent nature of the wastewater flows: activated sludge, contact stabilization, and trickling filters. Operational complexity and lack of long-term operational experience leads to the elimination of a physical-chemical treatment system as a viable alternative for this situation.

Although the quality of effluent from a rotating biological contactor (RBC) is adversely affected by hydraulic and organic surges, the fact that the process is simple and operation is routine implies that further review be made. A flow equalization device would be required at the head of the RBC treatment unit. Segregation of trailer dumpings or controlled feed may be required.

Suspended growth biological systems remaining eligible for implementation at the camp include an oxidation ditch and an extended aeration plant. Equalization of flows can be designed into each of these systems. Segregation of trailer wastewaters or controlled input into the treatment system may be required.

Waste stabilization ponds with seasonal discharge is the final alternative to be considered. The problem with lagoons is the requirement to clear a large area of land in order to facilitate construction. Further, the lagoon site must be removed (1000 m) from the camping area. Additional costs would be incurred to fulfill this requirement. However, the advantages of the system are obvious. Operational requirements are minimal, segregation of wastewater or controlled input of trailer wastewater to the treatment plant would not be required, and the impact on the receiving water would be minimal.

Thus, in review, four practical alternatives remain: an RBC plant complete with equalization facilities; an extended aeration plant; an oxidation ditch; and waste stablization ponds.

8.3.11 Cost Effectiveness Analysis. Four treatment system alternatives remain as a result of consideration of the foregoing selection parameters. It is anticipated that costs for the package extended aeration plant and the oxidation ditch would be similar and therefore only the former system need be costed.

a) Capital costs. The capital costs of three alternative treatment schemes are presented in Table 30. The total estimated cost for the package extended aeration plant is \$37 660, the rotating biological contactor, \$57 590, and the waste stablization pond, \$61 710. A major expense for the lagoon system is piping between the source of the wastes and the proposed lagoon site.

   A capital item not listed in the calculation of costs for the mechanical plants is the storage tank for receiving and holding wastewater dumps from trailers and campers. Assuming that the wastes could be hauled from the site on a weekly basis, a 13.6 m^3 storage tank is required. This tank would cost approximately \$3000 installed. The cost of hauling stored wastes off-site would be approximately \$100/wk or \$2200 per annum. A system designed to receive trailer wastes and slowly discharge such material to a mechanical treatment plant would cost \$4000, including storage tanks, piping, pumps and auxilliary equipment. O&M costs would be included as part of the total system O&M costs.

b) Operating costs. Operating costs are not major items of expense for any of the alternatives since the park will be operated only for five months per year. For the package plant, an operator would be required to inspect the system daily. In addition, electrical power for pumps, air blowers and/or shaft drives would be required. The operator must possess sufficient skill to take the necessary steps to control plant operations and to prevent upsets. The waste stabilization pond would require only periodic inspection and grass cutting. The operating costs for the package plants and stablization pond are \$7500 and \$2000 per year, respectively. Manpower would be provided out of the park's work force.

8.3.12 Summary. The capital costs of the alternatives indicate that the package extended aeration plant is the most attractive treatment scheme. Cost effectiveness of each alternative with respect to total costs over the expected life of the system is summarized in Table 31. The total cost shown favours the waste stabilization pond system.

Because of aesthetic considerations, the Parks Commision eliminated waste stablization. The extended aeration system was choosen in the final analysis. No storage tank or metering facilities were provided for trailer wastes. The rationale used in this

TABLE 30 SUMMARY OF CAPITAL COSTS OF TREATMENT ALTERNATIVES FOR CASE STUDY

Alternative	Item	Cost ($)
Package Extended Aeration Plant	45.5 m^3/d package plant excluding excavation, installation and piping from comfort station to plant, and plant to river.	$31 660
	Estimated installation and piping costs - 305 m of pipe, concrete foundation.	$ 5 000
	Engineering and Contingency (estimated @ 20%)	$ 1,000
	Total Cost:	$37,660
Rotating Biological Contactor	45.5 m^3/d plant exluding excavation, installation and piping from comfort station to plant, ana plant to river.	$37 990
	Estimated installation and piping costs - 305 m of pipe, concrete foundation.	$ 5 000
	18.2 m^3 equalization tank installed, complete with pumps and piping.	$ 5 000
	Engineering and contingency (estimated at 20%).	$ 9 600
	Total Cost:	$57 590
Waste Stabilization Ponds	Site cleaning and grubbing (1.2 ha)	$ 4 500
	Excavation (14 880 m^3 @ $2/$m^3$).	$29 760
	Feed piping (7.6 cm PVC) 1000 m @ $15/m (buried to 1 m depth).	$15 500
	Discharge piping 50 m @ $45/m.	$ 2 250
	Engineering and contingency (@ 20%).	$10 300
	Total Cost:	$61 710

TABLE 31 PRESENT VALUE AND TOTAL COST ESTIMATES FOR THE WASTEWATER TREATMENT ALTERNATIVES FOR CASE STUDY

Treatment Alternative	Capital Cost	Present Value* of O&M	Total Cost
Package Extended Aeration Plant	$37 660	$63 852 ($7 500)	$111 512
Rotating Biological Contactor	$57 590	$63 852 ($7 500)	$121 442
Waste Stabilization Pond	$61 710	$17 027 ($2 000)	$ 78 737
Storage Tank and Haulage Operation	$ 3 000	$18 730 ($2 200)	$ 21 730
Storage Tank and Controlled Feed Operation	$ 4 000		$ 4 000

* The "present value" of annual operating costs was calculated by assuming the following:

- projected life of each of the treatment systems of 20 years. After this period it was assumed additions or alterations would be required.
- interest at 10%.

Figures in brackets represent estimated annual operating and maintenance cost or base figure for present value calculation.

decision was that, should the discharge of these wastes prove to have an adverse affect on the operation of the extended aeration plant, remedial actions would then be taken.

8.3.13 Governmental Approval. Formal application to the Ministry of the Environment for the approval of the proposed treatment scheme for the River Front Park was made in October, 1975. Ministry approvals staff suggested modifications to the initial design which were incorporated into the final system design. Upon completion of Environmental Ministry review and subsequent revisions to the application, a meeting of the Environmental Hearing Board was called on February 10, 1976. At this meeting the general public as well as specific local government representatives were given full opportunity to raise questions, venture opinions and otherwise be heard concerning the installation of the treatment facility. As a result of this meeting, the Environmental Hearing Board recommended that approval be granted. The Municipal and Private Approvals Section of the Approvals Branch concurred with this recommendation and informed the Parks Commision of their decision on February 27, 1976. Final approval was granted on March 30, 1976.

REFERENCES

1. Marselek, J., "Instrumentation for Field Studies of Urban Runoff", Research Report No. 42, Canada-Ontario Agreement on Great Lakes Water Quality, Project No. 73-3-12, Ottawa, 1976.

2. Daugherty, R.L. and Franzini, T.B., Fluid Mechanics With Engineering Applications, sixth edition, McGraw Hill Inc., 1965.

3. Leupold, N., Jr. and Blois, R.S., "Measurement of Small Discharges Through V-Notch Weirs", Water and Sewage Works Journal, R-148, 1975.

4. Gurnhan, C.F. and Beach, M.I., "Application of Automatic Sampling To Today's Water Quality Control Programs", Presented at the 17th Ontario Industrial Waste Conference, Niagara Falls, Ontario, June 1970.

5. U.S. Environmental Protection Agency, "Methods for Chemical Analysis of Water and Wastes", Report No. EPA 625/6-74-003, 1974.

6. British Columbia Ministry of Health, Sewage Disposal Regulations, B.C. Regulation 577/75, Filed September 3, 1975.

7. Ontario Ministry of the Environment, "Septic Tank Systems", Information Services Branch, 135 St. Clair Ave. West, Toronto, n.d.

8. Alberta Department of Manpower and Labour, "Private Sewage Disposal", 5th edition, Plumbing Inspection Branch, Edmonton, Alta., 1972.

9. Brandes, M., "Accumulation Rate and Characteristics of Septic Tank Sludge and Septage", Journal WPCF, May 1978.

10. Webster, L.F., "Cottage Country: An Environmental Frontier", Water and Pollution Control, February 1974.

11. Bernhart, A.P., "Treatment and Disposal of Waste Water From Homes by Soil Infiltration and Evapo-transpiration", University of Toronto Press, Toronto, 1973.

12. U.S. National Sanitation Foundation Testing Laboratory, "Report on the Performance of Flygt Mini-Plant Model 4291-4", Report No. S40-1, Ann Arbor, Michigan, October 1972.

13. Chowdhry, N.A., "Evaluation of "Aquarobic" Sewage Treatment System", Ontario Ministry of the Environment, Applied Sciences Section, Pollution Control Planning Branch, Publication No. W. 52, Toronto, July 1974.

14. U.S. Department of Health, Education and Welfare, "Manual of Septic Tank Practice", Bureau of Community Environmental Management, Rockville, Maryland, 1972.

15. Goldstein, S.N. and Moberg, W.J. Jr., "Wastewater Treatment Systems For Rural Communities", Commission on Rural Water, Washington D.C., 1973.

16. Otis, R.J. et al., "On-site Disposal of Small Wastewater Flows", prepared for the U.S. Environmental Protection Agency, Technology Transfer, Washington, D.C., 1977.

17. Healy, K.A. and Laak, R., "Factors Affecting The Percolation Test", Journal WPCF, 45(7), 1973.

18. Kropf, F.W. et al., "Equilibrium Operation of Subsurface Absorption Systems", Journal WPCF, September, 1977.

19. Salvato, J.A. Jr., Environmental Sanitation, John Wiley and Sons Inc., New York, 1958.

20. McGill University, School of Architecture, "Stop the Five Gallon Flush. A Survey of Alternate Waste Disposal Systems", The Minimum Cost Housing Group, Montreal, April 1976.

21. Orr, R.C. and Smith, D.W., "A Review of Self-Contained Toilet Systems With Emphasis On Recent Development", paper presented at Symposium on Utilities Delivery in Arctic Regions, Co-sponsored by Environment Canada, Northwest Region and University of Alberta, Edmonton, March, 1976.

22. McKerman, J.M. and Morgan, D.S., "Experiences with the Clivus Multrum and Mull-Toa Toilets in Northern Manitoba", Manitoba Department of Northern Affairs, Interim Report, August 1976.

23. Grainge, J.W. et al., "Management of Waste From Arctic and Sub-Arctic Work Camps", Information Canada, Cat. No. R 72-10173, September 1973.

24. Pollutech Pollution Advisory Services Limited, "An Assessment of Odour Control Chemicals For Use In Marine Holding Tanks and Toilets", prepared for the Department of the Environment, Ottawa, Project No. 1233, March, 1974.

25. Environment Canada. Environmental Protection Service, Northwest Region, and Heinke, G., "Some Problems of Solid and Liquid Waste Disposal in the Northern Environment", Environmental Social Committee, Northern Pipelines, Task Force on Northern Oil Development, Report No. 74-10, July 1974.

26. James F. MacLaren Ltd., Environmental Consultants, "Feasibility Study of Holding Tanks and Sewage Haulage Systems For Individual Premises", Ontario Ministry of the Environment, Private Waste and Water Management Branch, Toronto, March 1973.

27. U.S. Environmental Protection Agency, "Alternatives for Small Wastewater Treatment Systems - OnSite Disposal/Septage Treatment and Disposal", EPA Technology Transfer Publication, EPA-625/4-77-011, October 1977.

28. Elliot, L.F. and F.J. Stevenson (eds.), Soils to Management of Organic Wastes and Waste Waters, published by American Society of Agronomy, Crop Science Society of America, Soil Science Society of America, Madison, Wis., 1977.

29. Feige, W.A. et al, "An Alternative Septage Treatment Method: Lime Stablization/Sand-Bed Dewatering", U.S. Environmental Protection Agency Technology Series, September 1975.

30. Tilsworth, T., "The Characteristics and Ultimate Disposal of Waste Septic Tank Sludge", Institute of Water Resources, University of Alaska, Fairbanks, Alaska, Report No. IWR-56, November 1974.

31. Carrol, R.G., "Planning Guidelines for Sanitary Wash Facilities", CH2M/Hill for U.S. Department of Agriculture, Forest Service, California, January 1972.

32. Feng, T. and Li, H.L., "Combined Treatment of Septage with Municipal Wastewater by Complete Mixing Activated Sludge Process", Water Pollution Control, Mass. Water Resources Commission, Report No. Env. E. 50-75-4, May 1975.

33. Chuang, F.S., "Treatment of Septic Tank Wastes by Anaerobic-Aerobic Process", Deeds and Data Supplement, WPCF Highlights 13(7):3, 1976.

34. Howley, J.B., "Biological Treatment of Septic Tank Sludge", M.S. Thesis, Department of Civil Engineering, University of Vermont, 1973.

35. Perrin, D.R., "Physical and Chemical Treatment of Septic Tank Sludge", M.S. Thesis, Department of Civil Engineering, University of Vermont, 1974.

36. Benjes, H.H. Jr., "Small Community Wastewater Treatment Facilities - Biological Treatment Systems, prepared for the U.S. Environmental Protection Agency Technology Transfer Seminar on Small Wastewater Treatment Systems, 1977.

37. U.S. Environmental Protection Agency, "Wastewater Treatment Facilities For Sewered Small Communities", ERIC Technology Transfer, EPA-625/1-77-009, October 1977.

38. Water Pollution Control Federation, Manual of Practice No. 8, Wastewater Treatment Plant Design, Water Pollution Control Federation, Washington DC, 20016, 1977.

39. Baumann, E.R., K.L. McRoberts and C.E. Smith, "Estimating Staffing and Cost Factors for Small Wastewater Treatment Plants Less than 1 MGD", Part 1, Staffing Guidelines for Conventional Municipal Wastewater Treatment Plants less than 1 MGD, U.S. Environmental Protection Agency Grant No. 5P2-WP-195-0452, 1973.

40. WPCF Manual of Practice No. 11, Operation of Wastewater Treatment Plants, Water Pollution Control Federation, Washington D.C. 20037, 1976.

41. Metcalf and Eddy, Inc., Wastewater Engineering, McGraw-Hill, Toronto, 1972.

42. Fair, G.M., J.C. Geyer and D.A. Okun, Water and Wastewater Engineering, John Wiley and Sons, Inc., New York, 1968.

43. U.S. Environmental Protection Agency, "Flow Equalization", EPA Technology Transfer Seminar Publication, 1974.

44. Eckenfelder, W.W., Water Quality Engineering for Practicing Engineers, Barnes and Noble, New York, 1970.

45. Tchobanoglous, G. "Wastewater Treatment for Small Communities", Parts 1 and 2, Public Works, July and August, 1974.

46. Guo, P.H.M., Thirmurthi, D. and Jank, B.E., "An Evaluation of Extended Aeration Activated Sludge Package Plants", Paper presented at the AQTE-FACE Conference, Quebec City, May 14-17, 1978.

47. Given, P.W. and Smith, D.W. "Critical Evaluation of Extended Aeration Systems In Arctic and Subarctic Regions", Environmental Protection Service, Economic and Technical Review Report EPS 3-WP-77-10, Water Pollution Control Directorate, Environment Canada, September 1977.

48. WPCF Manual of Practice No. 5, Aeration in Wastewater Treatment, Water Pollution Control Federation, Washington D.C. 20016, 1971.

49. Gibbon, D.L. (editor), "Aeration of Activated Sludge in Sewage Treatment", Ferro Corporation, Independence, Ohio, Pergamon Press Inc., Toronto, 1974.

50. Great Lakes - Upper Mississippi River Board of State Sanitary Engineers, Health Education Service, "Recommended Standards for Sewage Works", Albany, New York, 1971 (revised edition).

51. Murphy, K.L., Nilson, R.W., "Pilot Plant Studies of Rotating Biological Contactors Treating Municipal Wastewater", prepared for Central Mortgage and Housing Corporation, Ottawa, May, 1978.

52. Gloyna, E.F., "Waste Stabilization Ponds", World Health Organization, Monograph Series No. 60, Geneva, 1971.

53. U.S. Environmental Protection Agency, "Wastewater Treatment Ponds", Construction Grants Program Requirement, Office of Water Program Operations, Municipal Construction Division, Report No. EPA 430/9-74-011, Washington D.C. 20460, March 1974.

54. Middlebrooks, E.J., Falkenbong, D.H., Lewis, R.F. and Ehreth, D.J., "Upgrading Wastewater Stablization Ponds to Meet New Discharge Standards", Proceedings of a Symposium sponsored by Utah Water Research Laboratory, Utah State University and U.S. Environmental Protection Agency, Utah Water Research Laboratory, College of Engineering, Utah State University, Logan, Utah 84322, Report No. PRWG 159-1, November 1974.

55. Patterson, W.L. and R.F. Banker "Estimating Costs and Manpower Requirements for Conventional Wastewater Treatment Facilities", U.S. Environmental Protection Agency, Water Pollution Control Research Series 17080 DAN 10/71, 1971.

56. U.S. Environmental Protection Agency, "Sludge Treatment and Disposal", Technology Transfer Report EPA 625-1-74-006, October 1974.

57. Environment Canada/Ontario Ministry of the Environment, Proceedings of the "Sludge Utilization and Disposal" Conference, sponsored under the Canada-Ontario Agreement on Great Lakes Water Quality, Toronto, Ontario, February 1978, Conference Proceedings No. 6, Ottawa, 1978.

58. Ahlberg, N.R. and B.J. Boyko, "Evaluation and Design of Aerobic Digesters", Journal WPCF, 44(4), 1972.

59. Seto, P. and P. DeAngelis, "Concepts of Sludge Utilization on Agricultural Land", proceedings of the Sludge Utilization and Disposal Seminar held in Toronto, Ontario, February, 1978, sponsored under the Canada-Ontario Agreement on Great Lakes Water Quality, Conference Proceedings No. 6, Ottawa, 1978.

60. Vajdic, A.H., "Viruses and Parasites in Sewage Sludge", proceedings of the Sludge Utilization and Disposal Seminar held in Toronto, Feburary, 1978, sponsored under the Canada-Ontario Agreement on Great Lakes Water Quality, Conference Proceedings No. 6, Ottawa. 1978.

61. U.S. Environmental Protection Agency, "Process Design Manual Municipal Sludge Landfills", ERIC Technology Transfer Office of Solid Waste, EPA-625/1-78-010, SW-705, October 1978.

62. U.S. Environmental Protection Agency, "Disposal of Sewage Sludge Into A Sanitary Landfill", Solid Waste Management Report Series SW-71d, 1974.

63. Personal Communication with G. Tchobanoglous, Department of Civil Engineering, University of California, Davis, California, 1976.

64. U.S. Environmental Protection Agency, "Physical-Chemical Wastewater Treatment Plant Design", EPA Technology Transfer Seminar Publication, 1973.

65. WPCF Manual of Practice No. 4, Chlorination of Wastewater, Water Pollution Control Federation, Washington D.C., 20037, 1976.

66. Diamond Shamrock Corporation, Instruction Manual Sanuril Wastewater Chlorinator, Cleveland, Ohio, undated.

67. Ontario Ministry of the Environment, "Evaluation of Sanuril Wastewater Chlorinator", R.P.W. 2038, Toronto, September, 1972.

68. Ontario Ministry of the Environment, "Basic Gas Chlorination Workshop Manual", Training and Licensing Section, Sanitary Engineering Branch, Toronto, 1974.

69. Environment Canada, Environmental Protection Service, "Wastewater Disinfection In Canada", Economic and Technical Review Report EPS 3-WP-78-4, Water Pollution Control Directorate, Ottawa, May 1978.

70. White, G.C., Disinfection of Wastewater and Water For Reuse, Van Nostrand Reinhold Environmental Engineering Series, Van Nostrand Reinhold Company, New York, N.Y., 1978.

71. Fair, Geyer and Okun, Water and Wastewater Engineering, Volumes 1 and 2, John Wiley and Sons, Inc., New York, 1968.

72. Pound, C.E. and Crites, R.W., "Wastewater Treatment and Reuse by Land Application", Volumes I and II, EPA-660/2-73-006 a,b. Office of Research and Development, Unites States Environmental Protection Agency, August, 1973.

73. U.S. Environmental Protection Agency, "Evaluation of Land Application Systems", EPA-430/9-75-001, Office of Water Program Operations, March, 1975.

74. Wisconsin Department of Natural Resources, "Guideline Document for the Design, Construction, and Operation of Land Disposal Systems for Liquid Wastes", (Draft Copy), November, 1975.

75. U.S. Environmental Protection Agency, "Land Treatment of Municipal Wastewater Effluents (Design Seminar)", Parts I and II, Technology Transfer Program, 1975.

76. Ehlert, N.G., "Site Considerations and Pilot Studies for Land Disposal of Treated Sewage Effluent", Ontario Ministry of Environment, Toronto, December, 1975.

77. Neurohr G., "Land Application of Wastewater", Proceedings of the Seminar on Spray Irrigation of Treated Wastewater Effluents, sponsored by the Environmental Protection Service, Environment Canada, Harrison Hot Springs, British Columbia, April, 1975.

78. Effluent Irrigation Under Prairie Conditions, Technology Transfer Seminar sponsored by Western Canada Water and Sewage Conference, Saskatchewan Environment, Federation of Associations on the Canadian Environment, and Environmental Protection Service, Environment Canada, Regina, Sask., January 1979.

79. U.S. Environmental Protection Agency, "Cost of Wastewater Treatment by Land Application", Office of Water Program Operations, EPA-43019-75-003, June 1975.

80. Water Pollution Control Federation and the American Society of Civil Engineers, Design and Construction of Sanitary and Storm Sewers, WPCF, 3900 Wisconsin Ave., Washington, D.C., 1969.

81. U.S. Environmental Protection Agency, "Construction Costs for Municipal Wastewater Conveyance Systems: 1973-1977", Office of Water Program Operations, Washington, D.C. 20460, Report No. 430/9-77-015, May 1978.

82. Kreissl, J.F., "Status of Pressure Sewer Technology", U.S. Environmental Protection Agency Technology Transfer Design Seminar for Small Flows, 1977.

83. Cooper, I.A. and Rezek, J.W., "Vacuum Sewer Technology", U.S. Environmental Protection Agency Technology Transfer Seminar Program on Small Wastewater Treatment Systems, 1977.

84. Jespersen, F., "The Vacuum Sewage System", Paper presented at Symposium on Utilities Delivery in Artic Regions, Edmonton, Alberta, sponsored by Environment Canada and the University of Alberta, March, 1976.

85. Averill, D.W. and Heinke, G.W., "Vacuum Sewer Systems For Northern Applications", paper presented at an International Symposium on Wastewater Treatment in Cold Climates, Saskatoon, Saskatchewan, sponsored by the University of Saskatchewan and Environment Canada, August, 1973.

86. Moore, R., "Vacuum Sewage Systems", Technical Report, MWS&D Board, Sydney Australia, October 1978.

87. Vacusan Systems Limited, "On-Line Vacuum Sewage System: Design and Installation Manual", Calgary, Alta., May 1979.

APPENDIX A
GLOSSARY

APPENDIX A

GLOSSARY*

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 specific 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 surface area and adsorptive 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. It is through these microorganisms that the organics in the wastewater are decomposed to a simpler and more stable form.

Activated sludge process:

A biological wastewater treatment process in which the wastewater is brought into contact with the activated sludge in an aeration tank. The sludge is subsequently separated from the mixed liquor by sedimentation and wasted or returned to the process as needed. The supernatant is discharged over the weir of the settling tank for disposal.

Adsorption:

A taking up of gases, liquids or dissolved substances by the surfaces of solids with which they are in contact.

Aeration:

The supply of oxygen by introduction of air into the treatment system.

---

* From "Pollution Abatement in the Fruit and Vegetable Industry", Volume I - Basics of Pollution Control, published by the U.S. Environmental Protection Agency, Office of Technology Transfer, Washington DC, 1975, and "Alternative Methods for Treatment and Disposal of Community Wastewaters", by R.D. Wetter and M.W. Slezak, published by the British Columbia Water Resources Service, May 1975.

Aeration tank:

The tank where the wastewater is mixed with return sludge and aerated.

Aerobic:

(1) A condition characterized by the presence of dissolved oxygen in the aquatic environment.

(2) Living or taking place only in the presence of molecular oxygen.

Algae:

Organisms containing chlorophyll which grow in the presence of sunlight, inorganic nutrients and carbon dioxide.

Alkalinity:

The capacity of water to neutralize acids, a property imparted by the water's content of carbonates, bicarbonates, hydroxides, and occasionally borates, silicates and phosphates. It is expressed in milligrams per litre or equivalent calcium carbonate. Domestic sewage is usually slightly more alkaline than the water from which it is derived.

Anaerobic:

(1) A condition in which dissolved oxygen is undetectable in the aquatic environment. Usually characterized by formation of reduced sulphur compounds such as sulphides under septic conditions.

(2) Living or taking place in the absence of molecular oxygen.

Assimilative capacity:

The capacity of a natural body of water to receive wastewaters without deleterious effects to aquatic life or human activities.

Available chlorine:

A measure of the total oxidizing power of chlorine compounds.

Autotrophic bacteria:

Bacteria which derive their carbon and energy from the oxidation of inorganic matter.

Biochemical oxygen demand (BOD):

The quantity of oxygen used in the biochemical oxidation of organic matter in a specified time, at a specified temperature, and under specified conditions. A test used in measuring the biodegradable organic components in the wastewater as specified in Standard Methods for the Examination of Water and Wastewater, 14th edition, American Public Health Association, American Water Works Association, and Water Pollution Control Federation, 1975.

Biological oxidation:

The process whereby living organisms in the presence of oxygen convert the organic matter contained in wastewater into a more stable or mineral form.

BOD load:

The BOD content, usually expressed in pounds per unit of time, of wastewater passing into a waste treatment system or to a body of water.

BOD:N:P ratio:

The ratio, based upon analysis of wastewater passing into a waste treatment system, of the BOD to total nitrogen to total phophorus contained in the waste stream. To assure a nutrient balance within a biological treatment system, a ratio of 100:5:1 is generally recommended.

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 action:

The action exhibited by certain chemicals that resist a change in the acidity or alkalinity of a solution. In surface water the primary buffer action is related to carbon dioxide, bicarbonate and carbonate equilibria.

Bulking sludge:

Sludge floc having a low density which settles poorly.

Chemical oxygen demand (COD):

A measure of the oxygen-consuming capacity of wastewater. It is expressed as the amount of oxygen consumed from a chemical oxidant in a specific test. It does not differentiate between organic and inorganic matter, and thus does not necessarily correlate with biochemical oxygen demand. Also known as oxygen consumed (OC) and dichromate oxygen consumed (DOC).

Chloramines:

Compounds formed by the action of chlorine on nitrogenous compounds.

Chlorine demand:

The difference between applied chlorine and residual available chlorine in aqueous media under specified conditions and contact time. Chlorine demand varies with dosage, time, temperature and nature of the impurities.

Clarification:

The action of reducing the concentration of suspended matter in a liquid.

Coagulants:

Chemicals added to wastewater to destabilize, aggregate, and bind together colloids, emulsions and some dissolved materials.

Coagulation:

The process of modifying chemical, physical or biological conditions to cause flocculation or agglomeration of particles.

Coliform group:

A group of bacteria predominantly inhabiting the intestines of man or animal, but also occasionally found elsewhere. It includes all aerobic and facultative anaerobic, gram-negative, non-spore forming bacilli that ferment lactose with production of gas. Also included are all bacteria that produce a dark, purplish-green colony with metallic sheen by the membrane-filter technique used for coliform identification. The two groups are not always identical, but they are generally of equal sanitary significance. Their presence in water is presumptive evidence of contamination by fecal material.

Colloids:

Finely divided solids which will not settle due to the electrical charge on the particles. They may be removed by coagulation, biochemical action or membrane filtration; they are intermediate between true solutions and suspensions.

Combined available chlorine:

The concentration of chlorine which is combined with ammonia as chloramine or as other chloro derivitives, yet is still available to oxidize organic matter.

Composite wastewater sample:

A combination of individual samples of wastewater taken at selected time intervals, to minimize the effect of the variability of the individual sample. Individual samples may be equal volume or may be proportioned to flow at time of sampling.

Denitrification (biological):

The conversion of nitrate to molecular nitrogen by specific microorganisms under toxic conditions, i.e., free of molecular oxygen.

Diffuser:

A porous plate, tube, or other device through which air is forced and divided into minute bubbles for diffusion in liquids. Commonly made of carborundum, metal or plastic materials.

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.

Dispersed growth:

Non-flocculating microorganisms whose presence in treated wastewater results in a turbid effluent.

Dissolved oxygen (DO):

The amount of oxygen dissolved in a liquid, usually expressed in milligrams per litre, parts per million (ppm) or percent of saturation. In unpolluted water, oxygen is usually present in amounts of up to 10 ppm. Adequate dissolved oxygen is necessary for fish and other aquatic organisms.

Dissolved solids (total):

The total amount of dissolved material, organic and inorganic, contained in water or wastes.

E. Coli:

Abbreviation of Escherichia coli, a species of bacteria in the coliform group and normal inhabitants of the intestine of man and animals. Its presence is considered indicative of fecal contamination.

Effluent:

(1) A liquid which flows out of a containing space.

(2) Wastewater partially or completely treated, or in its natural state, flowing out of a reservoir, basin, treatment plant or part thereof.

Eutrophication:

The normally slow aging process by which a lake evolves into marsh and ultimately becomes unsuitable for human activities and aquatic life. In the course of this process the lake becomes overly rich in dissolved nutrients (for example, nitrogen and phosphorus), so that an excessive development of algae results. A process in which water becomes murky and noxious odours and unsightly scum occur. In the lower layers dissolved oxygen levels become depressed, and bottom-dwelling fauna change from clean water forms to pollution-tolerant forms.

Facultative bacteria:

Bacteria that can grow under aerobic or anaerobic conditions. Many organisms of interest in wastewater stabilization are included in this group.

Filtrate:

The liquid which has passed through a filter.

Filtration:

The process of passing a liquid through a porous medium for the removal of suspended or colloidal material contained in the liquid by a physical straining action.

Five-day BOD:

The amount of oxygen used in the biochemical oxidation of organic matter in five days under specified testing conditions.

Fixed film process:

A biological process, such as a trickling filter or rotating biological contactor, which utilizes a microbial population attached to a solid surface (captive film) to remove organics from the wastewater.

Fixed solids:

The residue remaining after ignition of solids at 550°C for 20 minutes. It is a measure of the inorganic material in the solids.

Floc:

Gelatinous or amorphous solids formed by chemical, biological or physical agglomeration of fine materials into large masses that are more readily separated from the liquid.

Flocculation:

The agglomeration of colloidal and finely divided suspended matter by chemical, physical or biological means.

Flotation:

The rising of suspended matter to the surface of the liquid in a tank as scum - by aeration, the evolution of gas, chemicals, electrolysis, heat, or bacterial decomposition - and the subsequent removal of the scum by skimming.

F/M ratio:

Food to microorganisms ratio: the weight ratio of BOD (food) in wastewater to suspended solids (microorganisms) in a biological treatment system. This value is used as an operational control criterion for activated sludge processes.

Free available chlorine:

The concentration of aqueous molecular chlorine, hypochlorous acid and hypochlorite ion. The relative proportion of the three species is pH dependent. At the pH of most waters and wastewaters, the hypochlorous acid and hypochlorite ion will predominate.

Free residual chlorination:

The application of chlorine or chlorine compounds to water or wastewater to produce a free available chlorine residual.

Grease:

A collective name for fats, waxes, free fatty acids, calcium and magnesium soaps, mineral oils and certain other non-fatty materials in the wastewater.

Grit:

Heavy inorganic particles of 0.2 mm in size or over, such as sand, gravel, cinders.

Hardness:

A characteristic of water, imparted by salts of calcium, magnesium and iron such as bicarbonates, carbonates, sulphates, chlorides and nitrates, that causes curdling and increased consumption of soap, deposition of scale in boilers, damage in some industrial processes and sometimes objectionable taste. It is expressed in mg/L of equivalent calcium carbonate. Waters containing less than 50 mg/L of hardness are considered soft, those containing more than 50 mg/L are considered hard waters.

Heterotrophic bacteria:

Bacteria which obtain their source of carbon and energy from the breakdown of organic matter.

Infiltration:

(1) The penetration of water through the soil from surface precipitation, stream or impoundment boundaries.

(2) The entrance of groundwater into a sewer through breaks, defective joints or porous walls.

Influent:

Water, wastewater or other liquid flowing into a reservoir, basin, or treatment plant or any unit thereof.

Integrator:

A device for indicating the total quantity of flow through a measuring device, such a Parshall flume or weir.

Leaching:

A process of draining of soluble salts, alkali and other constituents from soils or other media by natural percolation or abundant irrigation.

Loading:

The quantity of waste, expressed in cubic metres or gallons (hydraulic load), or in kilograms or pounds of BOD, COD, suspended or volatile (organic load) which is discharged to a wastewater treatment facility or water course.

Membrane filtration:

A method of quantitative or qualitative analysis of bacterial or particulate matter in a sample by filtration through a membrane capable of retaining bacteria.

Mesophiles:

Bacteria that grow best at moderate temperatures, having an optimum of 20° to 40°C.

Metabolism:

Chemical changes brought about by microorganisms in their use of food.

mgd:

Abbreviation for million gallons per day.

mg/L:

Abbreviation for milligrams per litre. A unit used for measuring the concentration of wastewater constituent.

Mixed liquor:

A mixture of sludge and wastewater undergoing activated sludge treatment in the aeration tank.

MLVSS:

Abbreviation for mixed liquor volatile suspended solids, a measure of the quantity of organic solids contained in the mixed liquor of an activated sludge treatment system.

Most Probable Number (MPN):

That number of organisms per unit volume that, in accordance with statistical theory, would be more likely than any other number to yield the observed test result, or that would yield the observed test result with the greatest frequency. Expressed as density or organisms per 100 ml. Results are computed from the number of findings of coliform-group organisms resulting from multiple-portion decimal-dilution platings.

Natural purification:

Natural processes occurring in a stream or other body of water that result in the reduction of bacteria, satisfaction of the BOD, stabilization of organic constituents, replacement of depleted dissolved oxygen, and the return of the stream biota to normal. Also called self-purification.

Nitrogenous waste:

Wastes of animal or plant origin that contain nitrogenous compounds.

Nutrient:

Elements or chemical compounds absorbed by living organisms and used in synthesis of cellular material. The major nutrients include carbon, hydrogen, oxygen, nitrogen, sulphur and phosphorus. Nitrogen and phosphorus are of major concern because they are frequently deficient in wastewaters, particularly industrial wastewaters.

Outfall:

(1) The point, location or structure where wastewater or drainage discharges from a sewer, drain or other conduit.
(2) The conduit leading to the ultimate disposal area.

Oxidation process (treatment):

Any method of wastewater treatment for the oxidation of the putrescible organic matter. The usual methods are the activated sludge and trickling filter processes. Living organisms in the presence of air convert the organic matter into more stable or mineral form.

Oxygenation capacity:

In treatment processes, a measure of the ability of an aerator to supply oxygen to a liquid.

Oxygen demand:

The quantity of oxygen utilized in the oxidation of organic matter in a specified time, at a specified temperature and under specified conditions. See BOD and COD.

Oxygen sag curve:

A curve that represents the profile of dissolved oxygen content along the course of a stream, resulting from deoxygenation associated with biochemical oxidation of organic matter and reoxygenation through the absorption of atmospheric oxygen and photosynthesis. Also called dissolve-oxygen-sag curve.

Parshall flume:

A device for measuring the flow of liquid in an open conduit. It consists essentially of a contracting length, a throat, and an expanding length. Flows through the device are determined by measuring the head of water at a specific distance from a sill over which water passes.

Parts per million (ppm):

The quantity of a minor constituent present in one million units of the major constituent of a solution or mixture. It is used to express the concentration of different constituents in the wastewaters. 1 ppm is equal to 1 mg/L if the specific gravity of the solution or mixture is 1.

Percolation:

(1) The movement or flow of water through the interstices or the pores of a soil or other porous medium.
(2) The water lost from an unlined conduit through its sides and bed.

Permeability:

The ability of a soil to transmit liquid, commonly measured as the rate of liquid movement through the soil. It is commonly expressed in centimetres (inches) per hour and is sometimes used in the same sense as perviousness.

pH:

The reciprocal of the logarithm of the hydrogen-ion concentration. pH values reflect the balance between acids and alkalies and range from 0 to 14.

Photosynthesis:

A process in which carbon dioxide and inorganic substances are converted into oxygen and carbohydrates with the aid of chlorophyl, utilizing sunlight for energy.

Pollutional load:

The quantity of material in a waste stream that exerts an adverse effect on the receiving system.

Polymer:

A high-molecular-weight, water-soluble flocculating agent. When added to water, it forms a flocculant precipitate which will agglomerate or coagulate suspended matter and expedite sedimentation.

Population equivalent:

A means of expressing the strength of wastewater. Domestic wastewater consumes, on an average, 0.8 kg oxygen/person/d (0.17 lbs of oxygen per capita per day), as measured by the standard BOD test. This figure has been used to measure the strength of organic industrial waste in terms of an equivalent number of persons. For example, if an industry discharges 454 kg (1000 lbs) of BOD per day, its waste is equivalent to the domestic wastewater from 5700 persons (454/0.8 or 1000/0.17).

Potable water:

Water that does not contain objectionable materials and infective agents and is considered satisfactory for domestic consumption.

Precipitate:

The formation of solid particles in a solution, or the solids that settle as a result of chemical or physical action.

Primary settling tank:

The first settling tank for the removal of settleable solids through which wastewater is passed in a treatment plant.

Proportional composite sample:

A combination of individual samples of wastewater taken at selected intervals and in proportion to the flow at time of sampling.

Psychrophiles:

Bacteria that grow best at relatively low temperatures, having an optimum of 10° to 20°C.

Raw sewage:

Untreated wastewater.

Raw sludge:

Settled, undigested sludge from settling tanks in a treatment system.

Receiving Water:

A natural watercourse, lake or ocean into which treated or untreated wastewater is discharged.

Residual chlorine:

Chlorine remaining in water or wastewater at the end of the specified contact period as combined or free chlorine.

Respiration:

The physical and chemical processes by which an organism takes up oxygen and releases carbon dioxide whereby energy is generated from life processes.

Retention time:

The length of time that wastewater is held in a tank or basin for treatment. It is calculated by dividing the tank volume by rate of flow. Also called detention time.

Runoff:

(1) That portion of rainfall or melted snow which runs off the surface of a drainage area and reaches a water body or a drainage system.
(2) The discharge of water in surface streams.

Scum baffle:

A verticle baffle dipping below the surface of wastewater in a tank to prevent the passage of floating matter. Also called scum board.

Sedimentation:

The process of allowing solids in the liquid to sink to the bottom for easy removal. Also called settling or clarification.

Sedimentation or settling tanks:

Tanks or basins in which sedimentation takes place.

Self-cleaning velocity:

The minimum velocity in sewers necessary to keep solids in suspension, thus preventing their deposition and subsequent nuisance from stoppages and odours of decomposition.

Settleable solids:

(1) Those solids in wastewater which settle to the bottom of a sedimentation tank.
(2) The volume of solids that settle to the bottom of an Imhoff cone in one hour.

Sloughing:

A phenomenon associated with fixed film biological treatment processes where biological solids build up to a varying degree and then slough off into the discharged flow.

Sludge:

Settled solids produced by wastewater treatment.

Sludge blanket:

The layer of sludge formed in a sedimentation tank.

Sludge bulking:

Sludge occupying excessive volumes and having poor settling characteristics.

Sludge conditioning:

Treatment of sludge to improve dewatering and enhance drainability, usually by the addition of chemicals.

Sludge digestion:

The process by which organic matter in the sludge is converted into more stable compounds through the activities of either anaerobic or aerobic organisms.

Sludge treatment:

The process used to remove water and/or to reduce the organic components in the sludge.

Sludge volume index (SVI):

The volume in millilitres occupied by one gram of sludge after 30 minutes of settling.

Sparger:

An air diffuser designed to give large bubbles, used singly or in combination with mechanical aeration devices.

Sphaerotilus:

A filamentous, sheath-forming bacterium, often encountered in the biological treatment system. The bulking sludge problem in the suspended growth systems is usually associated with the growth of this microorganisms.

Standard methods:

Standard Methods for the Examination of Water and Wastewater, 14th edition, published jointly in 1975 by the American Public Health Association, the American Water Works Association and the Water Pollution Control Federation.

Substrate:

The substances (food) used by organisms for the growth (synthesis) of new cellular material and the production of energy (respiration).

Supernatant:

(1) The liquid overlying deposited solids.

(2) The liquid in a sludge-digestion tank that lies between sludge at the bottom and floating scum at the top.

Surface loading:

One of the criteria for the design of settling tanks in treatment plants; expressed in cubic metres of wastewater per day per square metre (or gallons per day per square foot) of surface area in the settling tank.

Suspended growth process:

A biological treatment process such as activated sludge process, in which microbial population is kept in suspension by compressed air or mechanical methods.

Suspended solids (SS):

(1) Solids which either float on the surface, or are in suspension in liquids, and which are largely removable by laboratory filtering.

(2) The quantity of material removed from wastewater in a laboratory test, as prescribed in Standard Methods for the Examination of Water and Wastewater and referred to as non-filterable residue.

Synthesis:

The growth or development of cellular material by the degradation of substrate.

Thermophiles:

Bacteria that grow best at relatively high temperatures, having an optimum of 45°C or higher.

Total dissolved solids (TDS):

See dissolved solids.

Total organic carbon (TOC):

A test expressing the concentration of the organic carbon in the sample.

Upflow contact clarifier:

A settling tank in which water enters the bottom and is discharged at or near the surface.

Vacuum filter:

A filter consisting of a cylindrical drum mounted on a horizontal axis, covered with a filter cloth, and revolving with a partial submergence in liquid. A vacuum is maintained under the cloth for the larger part of a revolution to extract moisture. The cake is scraped off continuously.

Venturi meter:

A differential meter for measuring liquid flow through closed conduits or pipes. It consists of a gradual contraction to a throat which causes a pressure reduction. The difference in velocity heads between the entrance and contracted throat is an indication of the rate of flow.

Volatile acids:

Fatty acids containing six or less carbon atoms, which are soluble in water and which can be steam-distilled at atmospheric pressure. Volatile acids are commonly reported as equivalent to acetic acid.

Volatile solids:

Apparent loss of a residue ignited at 550°C for a period of time sufficient to reach constant weight of residue; this is generally specified as 20 minutes. It is a measure of the organic material in the solids.

Water quality standards:

Limits set by authority on the basis of water quality criteria required for beneficial uses. Limits are imposed on the physical, chemical and bacteriological characteristics required for specific beneficial use.

Weir:

An overflow structure built across an open channel for the purposes of measuring the flow.

Total solids (TS):

The summation of dissolved and suspended solids in the wastewater. Commonly determined on a weight basis by evaporation to dryness and expressed as milligrams per litre (mg/L).

Toxic substance:

A poisonous substance which inactivates or kills living organisms. Examples are cyanides found in plating and steel mill wastes, phenols from coke and chemical operations, pesticides, herbicides and heavy metal salts.

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.

APPENDIX B

PROVINCIAL, TERRITORIAL AND FEDERAL ENVIRONMENTAL AGENCY OFFICES

PROVINCIAL ENVIRONMENTAL AGENCIES

British Columbia

Ministry of the Environment,
Water Resources Service,
Pollution Control Branch,
Parliament Buildings,
Victoria, British Columbia
V8V 4S5

Alberta

Alberta Environment,
Oxbridge Place,
9820 - 106th Street,
Edmonton, Alberta
T5K 2J6

Saskatchewan

Department of the Environment,
1855 Victoria Avenue,
Regina, Saskatchewan
S4P 0R9

Manitoba

Department of Mines,
Natural Resources & Environment,
139 Tuxedo Avenue,
Winnipeg, Manitoba
R3N 0H6

Ontario

Minsitry of the Environment,
135 St. Clair Avenue West,
Toronto, Ontario
M4B 1P5

Quebec

Services de Protection de L'Environnement,
2360 Chemin Ste-Foy,
Ste-Foy, Québec
G1V 4H2

Prince Edward Island

Department of Environment and Tourism,
2nd Floor, Health Building.
Box 2000,
Charlottetown, Prince Edward Island

Newfoundland

Department of Consumer Affairs and Environment,
Elizabeth Towers,
St. John's, Newfoundland

TERRITORIAL WATER BOARDS

Northwest Territorial Water Board

Chairman, N.W.T. Water Board,
Department of Indian and Northern Affairs,
P.O. Box 1500,
Yellowknife, N.W.T.

Yukon - Territorial Water Board

Chairman,
Yukon Territory Water Board,
200 Range Road,
Whitehorse, Yukon

REGIONAL AND DISTRICT OFFICES OF THE ENVIRONMENTAL PROTECTION SERVICE DEPARTMENT OF THE ENVIRONMENT

<u>Atlantic Region</u>

Environmental Protection Service,
5151 George St., 6th Floor,
Bank of Montreal Tower,
Halifax, Nova Scotia
B3J 1M5

Tel. 426-6132

<u>District Office</u>

Environmental Protection Service,
365 Argyle Street,
Fredericton, New Brunswick
E3B 1T9

Tel. 452-3286

<u>District Office</u>

Environmental Protection Service,
Dominion Building,
c/o D.R.E.E.,
P.O. Box 115,
Charlottetown, P.E.I.
C1A 4A9

Tel. 892-8551

<u>District Office</u>

Environmental Protection Service,
P.O. Box 5037,
St. John's, Newfoundland
A1C 5V3

Tel. 737-5488

<u>Québec Région</u>

Environmental Protection Service,
1550 Maisonneuve Blvd., West,
Montréal, Québec
H3A 2A5

Tél. 283-7377

Ontario Region

Environmental Protection Service,
25 St. Clair Avenue East,
Toronto, Ontario
M4T 1M2

Tel. 996-7510

District Office

Environmental Protection Service,
River Road Laboratories,
Ottawa, Ontario
K1A 0H3

Tel. 998-3420

Western and Northern Region

Environmental Protection Service,
9942 - 108th Street
Room 804
Edmonton, Alberta
T5K 2J5

Tel. 425-4580

District Office

Environment Canada,
800 Kensington Building,
275 Portage Avenue,
Winnipeg, Manitoba
R3B 2B3

Tel. 985-2961

District Office

Environmental Protection Service,
P.O. Box 2310,
Yellowknife, N.W.T.
X0E 1H0

Tel. 873-3456

District Office

Environmental Protection Service,
975 Avord Tower,
2002 Victoria Avenue,
Regina, Saskatchewan
S4P 2R7

Tel. 522-6671

Pacific and Yukon Region

Environmental Protection Service,
Kapilano 100, Park Royal,
West Vancouver, B.C.
V7T 1A2

Tel. 666-6711 Ext. 240

District Office

Environmental Protection Service,
Room 225,
Federal Building,
Whitehorse, Yukon
Y1A 3A4

Tel. 667-6487