ENVIRONMENTAL HEALTH PROGRAM

HANDBOOK FOR APPROPRIATE WATER AND WASTEWATER TECHNOLOGY FOR LATIN AMERICA AND THE CARIBBEAN

PAN AMERICAN HEALTH ORGANIZATION
INTER-AMERICAN DEVELOPMENT BANK

December 1988

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by

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PREFACE

In Latin America and the Caribbean, the provision of potable water and sewerage systems represents one of the most important contributions to health and well-being as well as a stimulus to economic development. Consequently, both the Pan American Health Organization (PAHO) and the Inter-American Development Bank (BID) have a vested interest in promoting and supporting increased coverage and improved service levels. During the period 1961-1985, the Bank contributed about 60 percent of the total external financing in this sector, and PAHO has continuously provided technical cooperation to Member Countries so that they might achieve reliable, adequate levels of service. Both organizations are collaborating in increasing their effectiveness in this area during the International Drinking Water Supply and Sanitation Decade of the 1980s.

PAHO and IDB, first entered into a non-reimbursable agreement of technical cooperation in October 1981 and updated it in June 1984, with a view towards improving Member Country investment in the sectors of environmental health, health and nutrition, and supporting the preparation and execution of projects. This agreement was again renewed and updated in September 1985 and includes an environmental health component which provides for studies in the following five priority areas:

1. Review and formulation of design criteria
2. Technical information systems
3. Drinking water quality
4. Appropriate technology
5. Community participation

The terms of reference to conduct the studies were jointly prepared by PAHO's Environmental Health Protection Coordination and IDB's Sanitary Engineering Section of the Infrastructure Division of the Project Analysis Department. Under the technical supervision of these units, technical expert consultants are conducting studies and preparing respective reports and manuals. Upon completion, PAHO and IDB will distribute publications in each of the aforementioned five areas.

The Handbook for Appropriate Water and Wastewater Technology for Latin America and the Caribbean is the fifth and last in a series of publications produced under the September 1985 Agreement for technical cooperation between the Inter-American Development Bank and the Pan American Health Organization. These publications are intended to serve as practical references for engineers and planners engaged in the improvement and expansion of water and sanitation services during the International Drinking Water Supply and Sanitation Decade.

The need for a handbook to be used for the appropriate selection of technology for water supply and wastewater facilities is felt throughout the Region. More than 150 technologies were considered and 69 included in the handbook. Most of the technologies are conventional; some are innovative.

Their appropriateness lies in their correct application and adaptation to specific conditions so as to take advantage of their strong points and to avoid their use under situations in which their weaknesses would be an overriding factor.

This Handbook is the first attempt at consolidating, prioritizing and synthesizing information and data from the Countries of the Region in relation to the assurance of their appropriate application. Only the most salient features which are felt by the author to be of importance to the appropriate selection of technology have been included. The treatment of the technologies is not exhaustive because the intention is not a design manual. For specific situations the users may need to supplement this document with additional references. This reference is intended to be used in conjunction with the "Water and Wastewater Cost Analysis Handbook" and the "COSTEVAL" manual along with their respective computer programs which were issued previously as part of the series.

It is hoped that this contribution will facilitate country efforts to improve coverage and level of water supplies and sanitation services.

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SUMMARY

The Handbook for Appropriate Water and Wastewater Technology for Latin America and the Caribbean is the fifth and last in a series of publications produced under the September 1985 agreement for technical cooperation between the Inter-american Development Bank and the Pan American Health Organization.

This publication covers both water and wastewater but particular emphasis is given to wastewater technologies because service in this sector lags behind water service in Latin America and the Caribbean.

The handbook is divided into 3 sections. The first reviews some 43 different technologies for water supply and wastewater collection and disposal which were determined to be particularly appropriate for developing countries. This section provides a description of their salient features and lists the advantages and disadvantages, costs, operation and maintenance characteristics, control required and special factors for each of them. It also gives recommendations for their use. The second section provides case studies of 26 technologies which although not in widespread use do have good potential for application in Latin America and the Caribbean. The third section is devoted to assiting the reader in making an evaluation and appropriate selection of technologies for the conditions under which they are to be used. It includes both matrices as well as tables to facilitate comparison.

The major purpose of this document is to assist engineers and planners in arriving at an appropriate selection of the technology to be used in water supply and sanitation projects for small cities and towns. It should also serve as a convenient general reference for the various technologies which have been included.

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PREFACE

The idea of an appropriate technology handbook arose on my first trip to Latin America, which was about seven years ago. It was first necessary to grapple with the definitiion of "appropriate" as it might apply to the development and management of water supply and wastewater. "Suitable for the application or situation," is probably the best working definition for these purposes.

Appropriate technology is a need worldwide, not just for developing nations. Many water treatment plants and wastewater facilities are virtually non-operational in the U.S., as well as in other nations, for various reasons. The problem of obtaining, training and keeping good ooperators for example, is a universal one. Thus, the need to match the available resources and capabilities is also universal. System administrators and designers must try constantly to achieve this match.

Therefore, I began with the lofty goal of trying (in my wisdom) to develop the material for a handbook on the subject. I proposed the idea several years ago to my friend Mr. Juan Alfaro at the Interamerican Development Bank, and about a year and a half ago we began work. It is of course, much harder to achieve such a goal than to state it. There were a number of collaborators on this work, technical journals were searched, old files from past trips were sifted through, etc. The initial objective was limited to a few special technologies. I originally felt that the presentation should be limited to about twenty technologies. As the reader will see, the number worthy of consideration is far greater than this. As time passed, I found that I was always adding to the list rather than shortening (and perhaps strengthening) it. As I would review a different technology or a "new" application, I could also sense a possible application in a place I had visited, or one I could imagine. I feel certain now that the list is still very incomplete. Furthermore, the readers of this document will have their own additions and critical comments.

Finally, even though the work is not complete, the decision has been made to "publish" a "pre-publication" version of the work as it is now. The handbook will be reviewed by a number of individuals. The Pan American Health Organization and the Interamerican Development Bank are soliciting comments, additions and suggestions from readers. The existing handbook will then be revised and republished. Perhaps the work will someday be completed -- perhaps not.

Ed Martin

ACKNOWLEDGEMENTS

Help was received from many sources during the preparation of the handbook. My thanks first to Mr. Juan Alfaro at the Interamerican Development Bank who allowed this work to happen, and for his encouragement throughout.

My son, Edward T. Martin, III, went with me on several trips as an engineering graduate student, and drafted many of the sections.

There were several collaborators in the work. These special people helped by reviewing their files and making visits and reviews of existing technologies. The problem with using the word collaborators is the reader may get the idea that they are responsible for the content. All the errors of omission and commission are mine alone ! Many thanks to Ing. Jose M. de Azevedo Netto of Sao Paulo, Ing. Julio Burbano of Bogota, Ing. Adilio Luiz Monteiro de Barros of Rio de Janeiro, Ing. Humberto Olivero of Guatemala City, and Eugene J. Kazmierczak of Washington, D.C.

My thanks to Messrs. Fred Reiff and Ray Reid of PAHO in Washington, who provided extensive review comments to draft material.

Thanks also to the many individuals whom I met on trips and who shared their valuable time, information and data, especially Ing. Ricardo Nitroso of Montevideo.

INTRODUCTION

The word "appropriate" may mean many things, especially in the context of technology applications, as in this case to development and management of water and wastewater - quantity and quality. The technology is appropriate if it is suitable to the application in which it is intended, from the viewpoints of cost, operability, and simplicity of design (given the intended function).

Functional success is probably the most important consideration. If all engineered systems work properly, then enough water of suitable quality is available at all times - for all water uses, and the quality of the ground and surface waters receiving wastewater is preserved. Rather than insisting on lowest cost technology, the goal should be lowest cost technology that works. Unfortunately, we have many more examples of the former than the latter.

Other words have crept into use by practitioners of water technology applications and are thus important. Two are especially important - alternative and innovative. The U.S. Environmental Protection Agency (EPA) funds special technology projects for wastewater management, and the effort is called the Innovative & Alternative I/A program. Some of the technologies developed there are presented herein.

The word innovative applied to water management suggests newness. The applications presented in the handbook are more likely to be innovative applications of old, rather than new technologies. The word alternative suggests an option to what is currently being regularly done. Technology applications tend to fall in and out of fashion. Imhoff tanks are not being designed for application to wastewater treatment currently. Many older cities in the world have Imhoff tanks, vintage the 1930s, and some are still functioning. In any case, whatever the terms being applied, there is room for consideration of "other" technologies, especially if the same objectives of water supply and/or quality can be achieved at lower cost.

There is a danger to applying "new" technologies, if they have not been proven in practice. There is sometimes an urge on the part of the designer to "do something different." Technologies are presented herein if they have been used for the applications reported on. There was an attempt to include technologies only if there have been several applications.

There are design criteria for most of the technologies presented. There is a temptation to apply the criteria directly to designs elsewhere. Often pilot plant studies are not conducted because of the cost involved. If pilot plant studies are designed properly, most of the physical facility will become a part of the final design, and local conditions of influent quality and effluent requirements will be taken into account. The time required for the pilot plant analysis is never wasted.

Some technologies were included which are felt to have strong potential for application, even though the applications up to this time have been limited, i.e., there is a high probability for success. Some technologies, such as steep slope sewers, are virtually necessary applications in the future because of the high potential cost savings, and because changes in construction materials allow such applications.

The handbook is in two sections. The first section contains technologies which are more certain of applicability because there are more data and information available. The second section contains case studies or technologies which are very promising. The information presented is all that was available at the time.

This is not a manual for detailed design. Thus, details such as inlet and outlet structures, etc., are not included. The information in this manual should be used for preliminary design, planning, review of submitted designs, and comparison of alternatives.

No separate sections are offered for water and wastewater. The distinction between technologies suitable for potable water production and those suitable for wastewater treatment is disappearing, but unfortunately not fast enough. The technologies presented in the handbook are suitable for application in any case involving a need to improve water quality to the degree possible by applying the technology in question. Thus, any unit process (a technology performing one function in the series referred to as a treatment plant) will perform equally well on either water or wastewater, provided the influent characteristics to the unit process, and the expected effluent quality are within the limits of its performance capability. In other words "a filter is a filter" whether treating water or wastewater. Furthermore, it may be expected to perform adequately, as long as the suspended solids loading to the filter is controlled, and the system is properly operated. Today, there are many existing examples of filtration applied to wastewater treatment.

A distinct separation between water management for water supply purposes, and approaches to wastewater management is a luxury reserved for water-rich countries. Others must consider water needs and services as a continuum - ranging on the one hand from treating a groundwater source perhaps, to on the other hand treating water used once for waste carriage, especially in preparation for direct or indirect reuse.

Wastewater treatment must be viewed as a necessity in the continuum, but only as a "necessary evil" otherwise. Wastewater treatment is often considered to be "very expensive," or "not a high priority." Once-through water usage is very costly over the long term. Many countries are experiencing potentially serious water shortages, or at least, shortages of water of sufficient quality for required uses. One example is falling ground water tables; another is increasing salinity in available supplies. Even countries with relatively high rainfall, but characterized by high intensity - short duration storms, are becoming water short as populations grow and water use expands. It is difficult to maintain consistent water supplies in such situations.

To help in the evaluation and selection process, the discussion of each technology is presented in several subsections which may be considered as selection criteria. These are:

description; written material which describes the function and operation of the technology. Design criteria are usually presented in this section. Where design criteria are presented elsewhere, e.g., in the operation and maintenance (O&M) section, the criteria have special importance to O&M.

limitations/advantages and disadvantages; processes have differing application potential. These should be evaluated before application from the perspective of the actual performance expected, given the water quality and service requirements.

costs; costs are often considered to be the most important consideration. Function should be considered primarily, and then if equal functional performance can be expected from say two different processes, then cost may be evaluated and the lower cost option may be chosen. Often a process is applied because it has a lower first cost. Operating and maintenance costs should also be considered.

availability; this criterion addresses the availability of materials, equipment and supplies in remote areas. Many applications of technology have been made without consideration of follow-up services, replacement parts, and regular maintenance programs. These judgements must usually be made from the perspective of the local area where the technology will be applied. It is not possible to comment in general on availability.

operation and maintenance; O&M factors are addressed in this subsection. Processes may be easier or more complex, thus more or less operator training may be required. Preventive maintenance is often very important to continued function. These considerations should be made before a process is chosen.

reliability; this is an especially important selection factor in remote areas. Performance over the long term with little maintenance is only an ideal. Processes with more mechanical equipment may be expected to require more attention. Selection should not be made on the basis of less mechanical equipment but on functional performance.

special factors; considerations which are not included elsewhere are mentioned here.

recommendations; a summary statement about the selection criteria and the technology is presented here. Recommendations are only general in nature. It is important to consider local water quality, flow variability, and other factors.

The information available for the technologies varies for each. Thus the subsections are not always presented. A subsection on "control" is sometimes presented in those cases where process control is a special consideration in the process selection. Other subsections may be found in special cases.

Wherever cost data are given, the values have been adjusted to May, 1988, using the Engineering News Record, Construction Cost Index.

Other applications exist than those reported herein, and readers and reviewers are asked to submit information, design criteria and cost data from their own experience. The address for submittal of comments, and additions or suggestions is:

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1. SLOW SAND FILTERS

1.1 Description

Slow (0.05 to 0.13 gpm/ft^2 - 0.12 to 0.32 m/h) (48) sand filters have a high degree of efficiency for solids and turbidity removals in the case of raw water with low turbidity and color (turbidity up to 50 NTU and color up to 30 units). Taste and odor are removed in low quantities also. If raw water quality is poor, roughing filters are often used preceding the slow sand filters. The slower filtration rate also means a greater efficiency in the removal of bacteria (as compared to rapid filters). Bacterial removal may be considered the strong point of slow sand filtration. Chemicals are typically not used. The flow rates for slow sand filters are many times slower than rapid and roughing filters. The operating filter bed is not stratified.

The effective size of the sand used is about 0.2 mm, and may range between 0.15 and 0.35 mm, and a unifromity coefficient between 1.5 and 3.0. In contrast, the range of effective size for rapid sand filters is 0.35 to 1.0 mm, with a uniformity coefficient of 1.2 to 1.7. The media size used for roughing filters is much larger.

A distinguishing feature of slow sand filters is the presence of a thin layer at the surface of the bed called "schmutzdecke." This layer forms on the surface of the sand bed composed of large variety of biologically active micro-organisms. The breakdown products (organic matter) fill the interstices of the sand, so that solid matter is retained more effectively than rapid and roughing filters. The cleaning of the filter bed is carried out by manually scraping (removing) the top layer of the filter bed when it comes clogged with impurities.

In general, during filtration through a porous substance, the water quality is improved by mechanical straining of suspended and colloidal matter, by reducing the number of bacteria and other microorganisms, and sometimes by changing the chemical constituents. The porous substance may be any chemically-stable material, but beds of sand (silica and garnet) are used for water supply and wastewater treatment in most cases. Sand is cheap, inert, durable, and widely available. It has been extensively tested and has been found to give excellent results. The design bed thickness varies from 1.2 to 1.4 meters (m), but after successive cleanings, the resultant thickness may be 0.6 and 1.2 m. (See Figure 1.1).

Essentially, a slow sand filter consists of a water tight box provided with an underdrain system which also serves the purposes of supporting the filtering material, and distributing the flow evenly through the filter. Many different media have been used for the underdrain system. Bricks, stone and even bamboo have been for this purpose. Bamboo however, requires frequent renewal because it is organic and unstable.

The successful performance of a slow sand filter is dependent mainly on the schmutzdecke layer. In a mature bed, the layer, generally consisting of algae, plankton, and bacteria forms on the surface of the sand. Inorganic suspended matter is retained by straining action of the sand as well as by the schmutzdecke layer. The schmutzdecke organisms also may accomplish a certain amount of organic material breakdown.

The walls of the filter can be concrete or stone. Sloping walls dug into the earth and supported or protected by chicken wire reinforcement and sand or sand-bitumen could be a cost effective alternative to concrete. Inlets and outlets should be provided with controllers to keep the raw water level and the filtration rate constant.

Filtration rates usually employed for developing countries are between 2.5 and 6.0 m^3/m^2/day. Higher rates may be applied

Slow Sand Filter

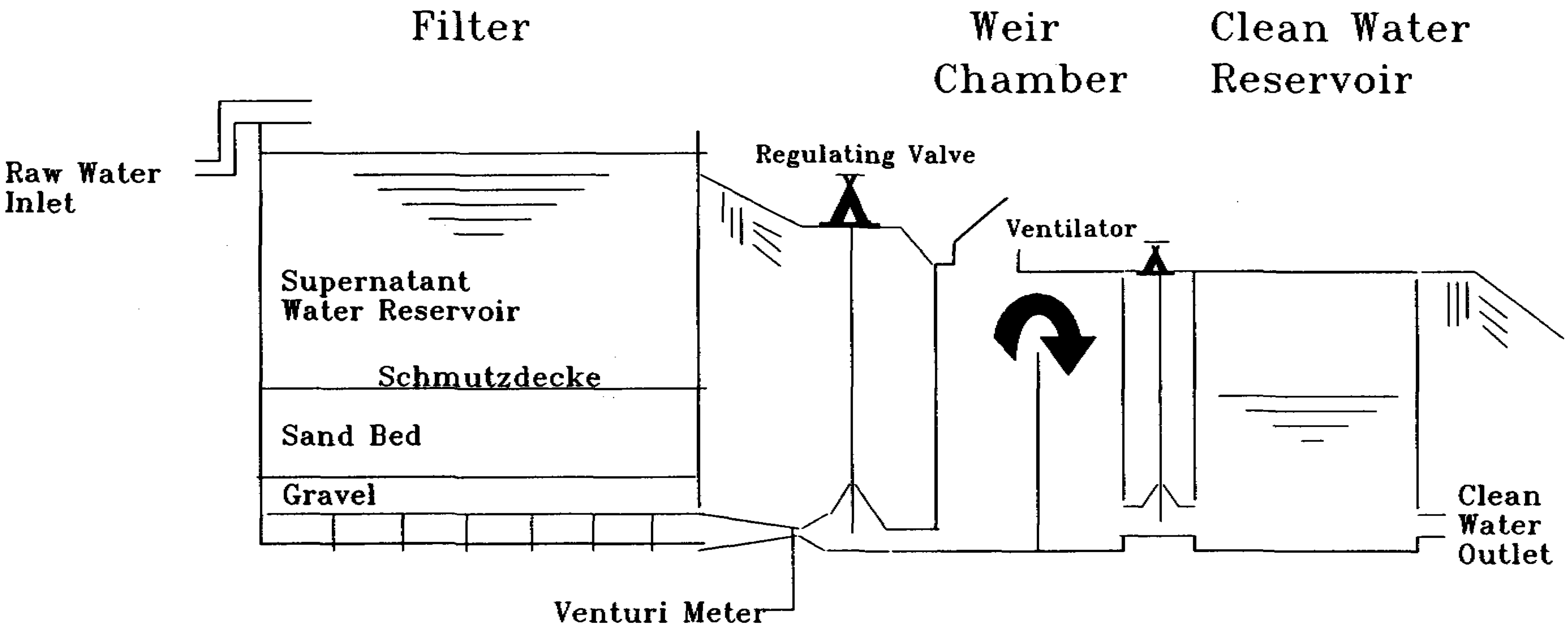


Figure 1.1

(File:Mart50)

after a series of tests are performed which yield good effluent quality results.

The system should be designed for ease of operation and flexibility. The design should consist of a number of separate units. The suggested number of units for given populations is as follows (9):

Population	Units
2000	2
10000	3
60000	4
200000	6

Bottom drains consist of a system of manifold and lateral pipes sized according to Table 1.1 (9).

1.2 Advantages and Disadvantages

In order to get good results from slow filtration the raw water must not be too contaminated with suspended solids (generally below 50 mg/l). This generally includes raw water from lakes and reservoirs, but only pretreated water from flowing streams.

Advantages for developing countries are: low construction cost using manual labor, simplicity of design and operation, unskilled maintenance labor, no chemicals required and sand can usually be found locally, power is not required, large quantities of wash water not required, sludge disposal is simpler (only because less contamination is removed during treatment). Disadvantages are: operation is suggested only with low contamination levels, pretreatment is probably required in many applications.

TABLE 1.1

LATERAL AND MANIFOLD PIPE SIZE AND FLOWS FOR BOTTOM DRAINS

Filtration Rate ($m^3/m^2/day$)		2,80	.3,75	4.70	5.60	7,50	9,35	14,00
Lateral	2"	7,4	6,5	0,0	5,5	4,9	4,5	3,7
	3"	16,8	14,9	13,7	12,8	11,4	10,6	8,7
	4"	30,1	26,8	24,6	22,8	20,3	18,6	15,6
	5"	48,2	42,8	39,1	36,3	32,0	29,4	24,8
	6"	69,7	62,3	56,8	53,0	46,5	42,8	36,2
	8"	112,0	112,0	102,0	94,0	84,0	76,0	64,0
Principal	10"	320	280	250	230	205	185	160
	12"	455	400	360	335	300	270	220
	15"	720	640	575	540	475	430	360
	18"	1 040	930	840	770	690	620	520
	21"	1 420	1 260	1 145	1 060	930	850	710
	24"	1 860	1 650	1 500	1 390	1 230	1 120	930
	27"	2 360	2 080	1 890	1 750	1 540	1 105	1 120
	30"	2 930	2 580	2 355	1 180	1 925	1 750	1 460

Source: Reference 9

On the other hand, provisions must be made for storing used sand permanently or temporarily until it washed, for washing used sand, moving sand from the filters to a wash site. If sand is to be washed, a separate backwash facility is required and a water supply is required; treated water may be used for washing. Close operational control of head loss is required to prevent air binding which is a potential problem in all types of filters.

1.3 Costs

See Table 1.2 (1) and Table 1.3 for operating and maintenance costs (57).

1.4 Availability

Slow sand filtration is perhaps the most common of the filtration technologies in developing countries. It has been proven both mechanically and economically many times. Systems are being replaced with rapid sand filters. See rapid sand filtration section.

1.5 Operation and Maintenance

The initial resistance (loss of head) of the clean filter bed is about six centimeters. During filtration, impurities deposit in and on the surface layer of the sand bed, and the loss of head begins to increase. When the loss of head has reached a pre-set limit (the head loss is usually not allowed to exceed the depth of water over the sand, about 1 to 1.5 m), the filter is put out of service and cleaned. The period between cleaning is typically 20 to 60 days. The method of cleaning can be either scraping off the surface layer of sand and washing and storing cleaned sand for periodic resanding of the bed, or washing the surface sand in place with a washer traveling over the sand bed. If sand is readily available the former method is favored. Men with flat wide shovels do the scraping and remove from one to two

TABLE 1.2

PER CAPITA COST PARAMETERS ($U.S.) AND
OPERATION AND MAINTENANCE MANPOWER REQUIREMENTS

PROCESS: SLOW SAND FILTER

Population Scale	Type of Cost	Cost Range	
1	Construction	17.08	27.00
(500 - 2,499)	Operation and Maintenance	1.80	6.75
2	Construction	12.19	19.28
(2,500 - 14,999)	Operation and Maintenance	0.81	3.04
3	Construction	8.55	13.51
(15,000 - 49,999)	Operation and Maintenance	0.45	1.69
4	Construction	5.33	8.44
(50,000 - 100,000)	Operation and Maintenance	0.27	1.01

Source: Reference 1

TABLE 1.3

ESTIMATED OPERATION AND MAINTENANCE COSTS FOR SLOW SAND FILTERS*

Loacation	Average Operational Flow (MGD)	Labor for Scraping (man-hour/year)	Labor for Resanding (man-hours/year)	Labor for Day-to-Day Activities (man-hours/year)	Total Labor Costs ($/year)	Total Operation and Maintenance Unit/Cost (¢/1000 gal)
Auburn	6.0	1007	618	365	11,400	0.5
Geneva	2.5	374	218	365	8,000	1.2
Hamilton	0.3	224	NA	365	6,400	5.7
Ilion	1.5	905	563	365	20,000	3.6
Newark	2.0	143	226	365	8,200	1.2
Ogdensburg	3.6	8736	[1]	365	25,700	2.2
Waverly	1.2	582	420	365	14,800	4.0

*All cost figures are based on a $10/hr wage rate except at Auburn, where $3/hr was used.

[1]Scraping and resanding may be done simultaneously.

Source: Reference 57

centimeters of top material. The work of cleaning by hand is usually completed in one or two days. After washing, the sand is stored and replaced on the bed when, by successive cleanings, the thickness of the sand bed has been reduced to about 50-80 centimeters. About 0.2 to 0.6 percent of the water filtered is required for washing purposes. When resanding, a process of "throwing over" is carried out. During this process, an additional depth of old sand is added, and the old sand replaced on top of the cleaned sand. The purpose of this process is to retain much of the active material and enable the resanded filter to become operational with a minimal amount of reripening.

After being cleaned, the beds are slowly refilled with filtered water from below until the sand is completely covered. This prevents entrapment of air in the sand. Cleaning experience is given in Table 1.4 (57).

1.6 Control

Process control can be based on effluent quality, usually some established level of suspended solids for a given water use, or in the case of wastewater - some level acceptable for discharge or for subsequent treatment. The desired effluent quality is typically related to head loss in the filter. Thus quality is indirectly measured by process performance. If this technique is used and the product water quality is important, i.e, for water supply, the correlation between effluent quality and operational head loss should be checked regularly and often.

1.7 Special Factors

Whenever raw water influent turbidity values of higher than 50 JTU are encountered. Slow sand filtration should be preceded with pretreatment, such as sedimentation, rapid filtration, or roughing filters.

TABLE 1.4

SUMMARY OF FILTER SCRAPING DATA

Location	Plant Size MGD	Average Filter Run Water Production (gal/ft^2)	Average Frequency of Filter Scraping Operations (Number per year)	Amount of Sand Removed in Scraping Operation (in.)	Method(s) Used in Removing Sand from Filter Surface	Time Required to Scrape Filters (man-hours/100 ft^2
Auburn	6.0	6,844	4.3*	0.5	Shovels, hydraulic	4
Geneva	2.5	15,718	2.0	1.0	Shovels, motorized buggy	4-5
Hamilton	0.3	4,302	2.0	1.0	Shovels, 50 gal drums, backhoe	8-9
Llion	2.0	15,487	1.8	3-4	Shovels, hydraulic	23-42
Newark	3.6	10,122	3.3	1.0	Shovels, motorized buggy	2
Ogdensburg	N/A	2,978	12.0	1.0	Shovels, hydraulics	4-5
Waverly	1.2	3,200**	9.7**	1.0	Shovels, wheelbarrows	5

* Two scraping operations per year are actually occasions when the filters are raked and no sand is removed.

** Water production and scraping frequency estimated by the Waverly personnel for the future using data from a 9-month operations study.

Source: Reference 57

1.8 Recommendations

Slow sand filters have a significant water cleaning capacity. Product water is often bacteriologically safe and free from solid impurities. Disinfection should always be practiced for water supply applications. Some typical removal rates achieved in developing countries are as follows: turbidity, 97-99%; color, 20-30%; iron, 50-60% (removed incidentally); bacteria, 95-97%.

2. RAPID SAND FILTERS

2.1 Description

When large amounts of water or very turbid water must be treated, slow sand filters are at a disadvantage because solids may be stored only in the relatively thin layer at the top of the bed. More rapid filtration and filtration of more turbid waters is made possible by making available more of the bed depth during filtration. This is in turn possible using coarser and in particular more uniform sand grains. Rapid sand filter media may range in size from 0.35 to 1.0 mm. A typical size might be 0.5 mm, with an effective size of 1.3 to 1.7 (1, 51). This range of media size has demonstrated the ability to handle turbidities in the range of 5 to 10 NTU at rates up to 2 gpm/sq ft (4.88 m/h) (51). Common filter rates for rapid filters may be as high as 100 to 300 $m^3/m^2/day$ (m/day), that is about 50 times the rates used with a slow rate filter.

The number of filters used for a specific plant capacity is as follows (1):

Plant Capacity (L/sec)	No. of Filters
50	3
250	4
500	6
1000	8
1500	10

In general, coagulation and sedimentation (settling) may be required pretreatment for rapid sand filtration. So typically, rapid sand filtration plants consist of chemical pretreatment, followed by rapid sand filtration, and disinfection (see Figure 2.1). The gravity rapid sand filter is commonly used to remove nonsettleable floc and other impurities remaining after coagulation and settling. The removal action is a combination of

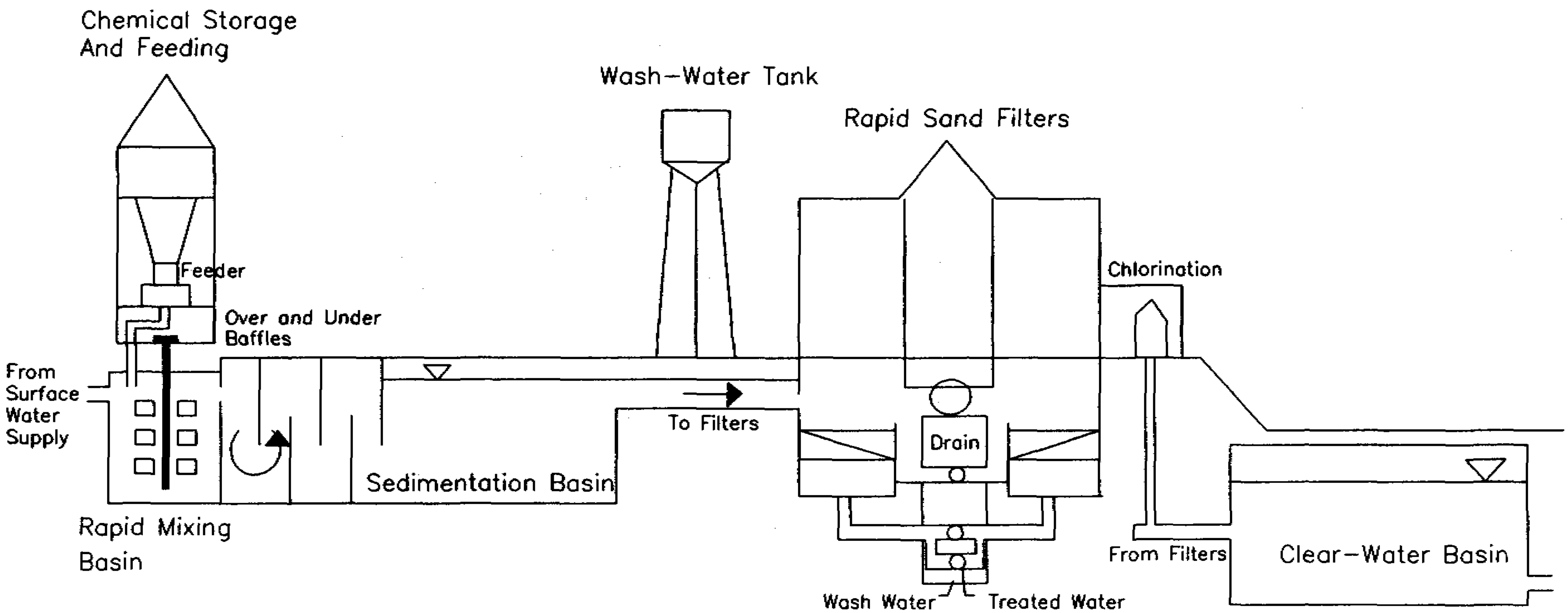


Figure 2.1: A Rapid Sand Filtration Plant

(Reference: 1)

(File:Mart56)

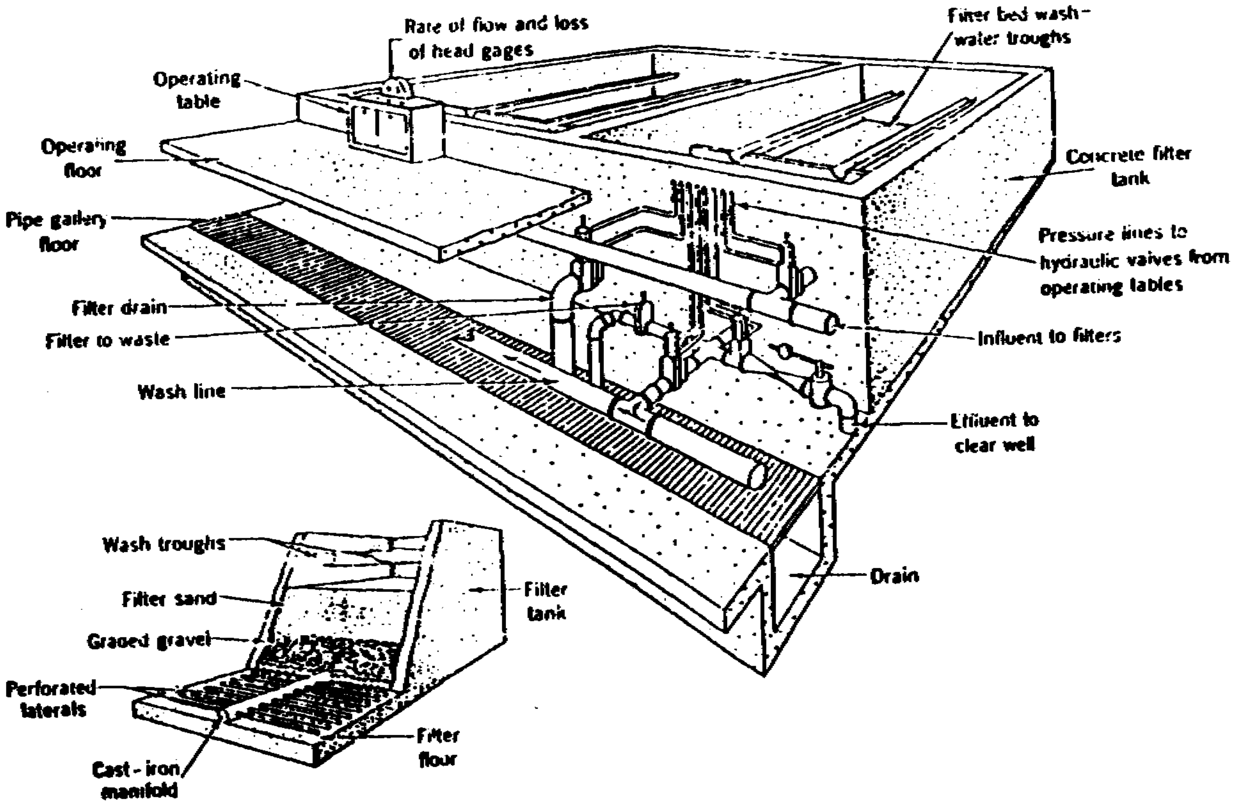


Figure 2.2: Rapid Filters and Accessory Equipment

mechanical straining, flocculation and sedimentation, all of which participate within the filter to remove particles (1).

A rapid sand filter consists of an open watertight basin containing a layer of sand 60 to 80 cm thick, supported on layers of gravel. The gravel in turn, is supported by an underdrain system. In contrast to a slow sand filter, the sand is graded in a rapid rate filter configuration. The sand is regraded each time the filter is backwashed - the finest at the top of the bed. The underdrain system, in addition to the functions served for the slow rate filter, serves to uniformly distribute the wash water to the bed. The underdrain system may be of various types, perforated pipes, pipe and strainer, vitrified tile block with orifices, porous plates etc. There are as many types as manufacturers. A clear well is typically located beneath the filters (or in a separate structure), to provide consistent output quantity.

The minimum number of filters in a system is two. The surface area of a unit is normally less than 150 m^2. The ratio of length to width is 1.25 to 1.35.

Filter design flows and velocities are given in Table 2.1.

2.2 Advantages and Disadvantages

Rapid sand filtration plants are complicated to operate. Operator training is required for consistent quality and quantity of output.

The filters require frequent backwashing to maintain satisfactory operating heads in the system (filter runs may vary from only a few hours, to as many as 24 to 72 hours, depending on the suspended solids in the influent. During filtration, the depth of water above the bed is 1.0 to 1.5 m. The total head available for filtration is represented by the difference in

TABLE 2.1

FILTER PIPING DESIGN FLOWS AND VELOCITIES

Description	Velocity FT/SEC (M/S)	Maxium Flow, GPM/SQ FT (M/H) of Filter Area
Influent	1-4 (0.305-1.22)	8-12 (19.5-29.3)
Effluent	3-6 (0.92-1.83)	8-12 (19.5-29.3)
Washwater supply	5-10 (1.52-3.05)	15-25 (36.6-6-61)
Backwash waste	3-8 (0.92-2.44)	15-25 (36.6-61)
Filter to waste	6-12 (1.83-3.66)	4-8 (9.8-19.5)

Source: Reference 48

water levels between the water surface above the filter and the level in the clear well, typically three to four meters. Backwashing rates are typically 0.6 $m^3/min/m^2$ and higher, for a period of several minutes. In addition, the initial production following backwashing is wasted for several minutes. Thus, the water usage for backwashing can be significant - ranging from a few percent to as much as 10 or 15 percent of the total plant output.

Rapid sand filter plants (including chemical treatment) can effectively treat higher solids loadings and produce higher outputs than slow sand filters. The land area requirements are significantly lower.

2.3 Costs

See Table 2.2.

2.4 Availability

Conventional rapid sand filtration plants are widely available and widely used in Latin America and the world. Data and information related to design, construction and operation can be found at most operating utilities.

2.5 Operation, Maintenance and Control

There are a number of problems which can upset the consistent operation of rapid rate filters. These problems often result form poor design. The problems may be solved by persistent and thoughtful operation (48, 56).

Surface clogging and cracking - Caused by overloading of solids at the thin filter layer, typically found in sand filters. The problem can be alleviated by dual, or multiple media, which allow deeper penetration of solids into the bed, and generally

TABLE 2.2

PER CAPITA COST PARAMETERS ($U.S.) AND
OPERATION AND MAINTENANCE MANPOWER REQUIREMENTS

PROCESS: RAPID SAND FILTER--CONVENTIONAL

Population Scale*	Type of Cost	Cost	Range
1	Construction	12.84	15.12
(500 - 2,499)	Operation and Maintenance	2.43	5.40
2	Construction	10.08	11.88
(2,500 - 14,999)	Operation and Maintenance	1.22	2.70
3	Construction	5.73	6.75
(15,000 - 49,999)	Operation and Maintenance	5.72	2.36
4	Construction	3.04	3.58
(50,000 - 100,000)	Operation and Maintenance	0.91	2.03

* at 120 gpcd

Source: Reference 1

longer run times. Often rapid increases in head loss are evident also. The same problem may result from use of filter aids, e.g., polyelectrolytes. Lower dosage may help.

Short runs due to floc breakthrough - This problem can be avoided by use of mixed media. (Mixed media or "coarse-to-fine particle" filters are not covered in depth in this handbook but the following comparisons are added for completeness.) There is typically a much greater surface area of the grains in mixed media systems, than sand, or even dual media. Also, there is a greater total number of fine particles and smaller pore openings at the bottom, than with dual or sand filters. Floc storage depths in filter beds will be much deeper (using more of the available bed depth), in mixed, perhaps as much as twice the depth of storage in dual media. In a sand filter, there is virtually no storage depth.

Variations in effluent quality with changes in flow rate or input quality - A dual or mixed media system will improve the operational consistency.

Gravel displacement or mounding - Mounding can be alleviated by placing a 3 inch (76 mm) layer of coarse garnet (any of several silicate materials which are generally crystallized; generally red or brown in color) between the gravel supporting the media and the fine bed material. Reducing the total flow and head available for backwashing will also help.

Mudball formation - Increasing the backwash flow rate (say up to 20 gpm/ft^2, 48.8 m/h), and providing for auxiliary water or air scour or surface wash capability will help. Very fine size sand particles are found to a higher degree in mudballs.

Growth of filter grains, bed shrinkage, and media pulling away from sidewalls - These are related problems which can be alleviated again, by providing adequate backwashing capability.

Calcium carbonate adherence to filter grains may actually be controlled by adding filter aids.

Sand leakage - This problem may be alleviated by adding the garnet layer.

Loss of media - This occurs typically to coal grains in dual media filters and is difficult to control. Increasing the distance between the top of the expanded bed and the wash water troughs may help. Auxiliary scour should be cut off a few minutes before the end of the backwash cycle.

Negative head and air binding - The more depth between the top of the expanded bed and the wash water troughs, the better, say 5 ft (1.5 m). The filter should not be operated to final headlosses which are greater than the depth of submergence of the filter media. When the input water contains high concentrations of dissolved oxygen, and the pressure is reduced by siphon action, the potential for air binding increases (there is a discussion of siphon based filtration in the case study section). Accumulation of bubbles in the bed increase significantly the resistance to flow. Maintaining high water depths in the filters, and frequent backwashing may help. Often, there is no solution.

2.6 Special Factors

There are a number of considerations to be taken into account for good filter design. An approach is presented in Table 2.3.

2.7 Recommendations

Rapid sand filters are more complex then their slow sand filter counterpart, but they are widely used in developing countries in areas of high turbidity and where land requirements are an important consideration.

TABLE 2.3

FILTER EVALUATION CHECKLIST

1. Filter media sizing and selection should be based on pilot tests. If this is not possible, data should be obtained from similar applications to determine the suitability of the media design.
2. In dual-and mixed-media filter systems, provisions should be made for the addition of polyelectrolytes directly to the filter influent.
3. The turbidity of each filter unit should be monitored continuously and recorded.
4. The flow and headloss through each filter should be monitored continuously and recorded.
5. Provisions should be made for the optional addition of disinfectant directly to the filter influent.
6. Provisions should be made for complete filter backwash cycle. The filter controls and pipe galleries should be housed.
7. The backwash rate selected must be based upon the specific filter media used and the wastewater temperature variations expected.
8. Filter backwash supply storage should have a volume at least adequate to complete two filter backwashes.
9. Adequate surface wash or air scour facilities must be provided.
10. There should be adequate backwash and surface wash pump capacity available with the largest single pumps out of service.
11. Backwash supply lines must be equipped with air release valves.
12. A means should be provided to indicate the backwash flow rate continuously and to enable positive control of the filter backwash rate. A means should also be provided to limit the filter backwash rate positively to a preset maximum value.

Source: Reference 48

2A. CHEMICAL TREATMENT FOR USE WITH FILTRATION

2A.1 Description

Chemical treatment is usually required as pretreatment for filtration, especially rapid rate filtration. Rapid rate filtration is usually considered to operate on higher turbidity input water than slow rate filters. Actually, the rapid rate systems can treat higher input suspended solids concentrations, at higher rates than slow rate systems, largely due the use of chemical pretreatment ahead of the filters.

In general then, as stated in Section 2, coagulation and settling are required pretreatment for rapid sand filtration. The gravity filtration step may be viewed as "polishing" as impurities remaining after coagulation and settling are removed.

Chemical treatment proceeds in three stages: rapid or flash mixing, coagulation (usually taking place partly in the mixing stage and partly in the flocculation stage, followed by flocculation or slow mixing. In the rapid mixing process, a coagulant is rapidly and uniformly dispersed through the mass of water. In the subsequent flocculation process, a readily settleable floc is built up (floc growth).

The flocculation stage involves slow and gentle stirring with sufficient time allowed to build up the floc. Detention times range from 20 to 60 minutes, and velocity gradients range from 5 to 100 1/sec with optimum values between 30 and 60 1/sec. Too high a velocity gradient will shear floc particles, and too low a velocity gradient will fail to provide sufficient agitation to allow floc formation. Baffled flocculation basins of horizontal (around-the-end) or vertical (over-and-under) types are the most suitable for small plants in rural areas. They have the advantages of little short-circuiting and no mechanical flocculating equipment, e.g, rotating paddles. The depth of the basins is around three to five meters. Spacing between the baffles is around 60 centimeters to facilitate cleaning.

Design for sedimentation which follows flocculation depends on the settling characteristics of the floc formed in the coagulation process. A general range of detention time is two to four hours. The overflow rates (surface loading) used in floc settling vary from 20 to 40 $m^3/m^2/c$ day (m/day), and the horizontal velocity is commonly below 30 cm/min to minimize the disturbances caused by such things as density currents and eddy currents in the basin. The depth of the basin is about 2 to 5 meters, 3 meters being preferred. The ratio of length of width is commonly between 3:1 and 5:1. Control of the outflow is generally secured by a weir attached to one or both sides of a single or multiple outlet trough.

2A.2 Advantages and Disadvantages

Rapid sand filter plants which include chemical treatment can effectively treat higher solids loadings and produce higher outputs than slow sand filters.

Raw waters contain colloids (particles stabilized by electrical charges which inhibit the agglomeration and subsequent removability of separate particles). These colloid systems may be destabilized (neutralization of the electrical forces) by adding chemical coagulants and supplying energy through mixing. Chemicals typically used include aluminum and iron salts and polyelectrolytes. Coagulation may be considered the charge neutralization stage and flocculation, the floc-building or agglomeration stage of the chemical treatment unit process. Properties of common coagulants are given in Table 2A.1 (55). Most polyelectrolytes are classified as nonionic, cationic, or anionic depending on the primary operative neutralization mechanism available for the given molecule. The cationic types typically have molecular weights below 100,000, and are available

TABLE 2A.1

PROPERTIES OF COMMON COAGULANTS

Common name	Formula	Equiv. weight	pH at 1%	Availability
Alum	$Al_2(S)_4)_3 \cdot 14H_2O$	100	3.4	Lump-17% Al_2O_3 Liquid-8.5% AL_2O_3
Lime	$CA(OH)_2$	40	12	Lump-asCaO Powder-93-95% Slurry-15-20%
Ferric chloride	$FeCl_3 \cdot 6H_2O$	91	3-4	Lump-20% Fe Liquid-20% Fe
Ferric sulfate	$Fe_2(SO_4)_4 \cdot 3H_2O$	51.5	3-4	Granular-18.5% Fe
Copperas	$FeSO_4 \cdot 7H_2O$	139	3-4	Granular-20% Fe
Soidum aluminate	$Na_2Al_1O_4$	100	11-12	Flake-46% Al_2O_3

Source: Reference 55

as aqueous solutions. The others have weights above 1,000,000, and are generally available as powders and/or emulsions (55).

2A.3 Costs

The design criteria used for the development of cost estimates are given on Table 2A.2. The costs are given on Figure 2A.1.

2A.4 Availability

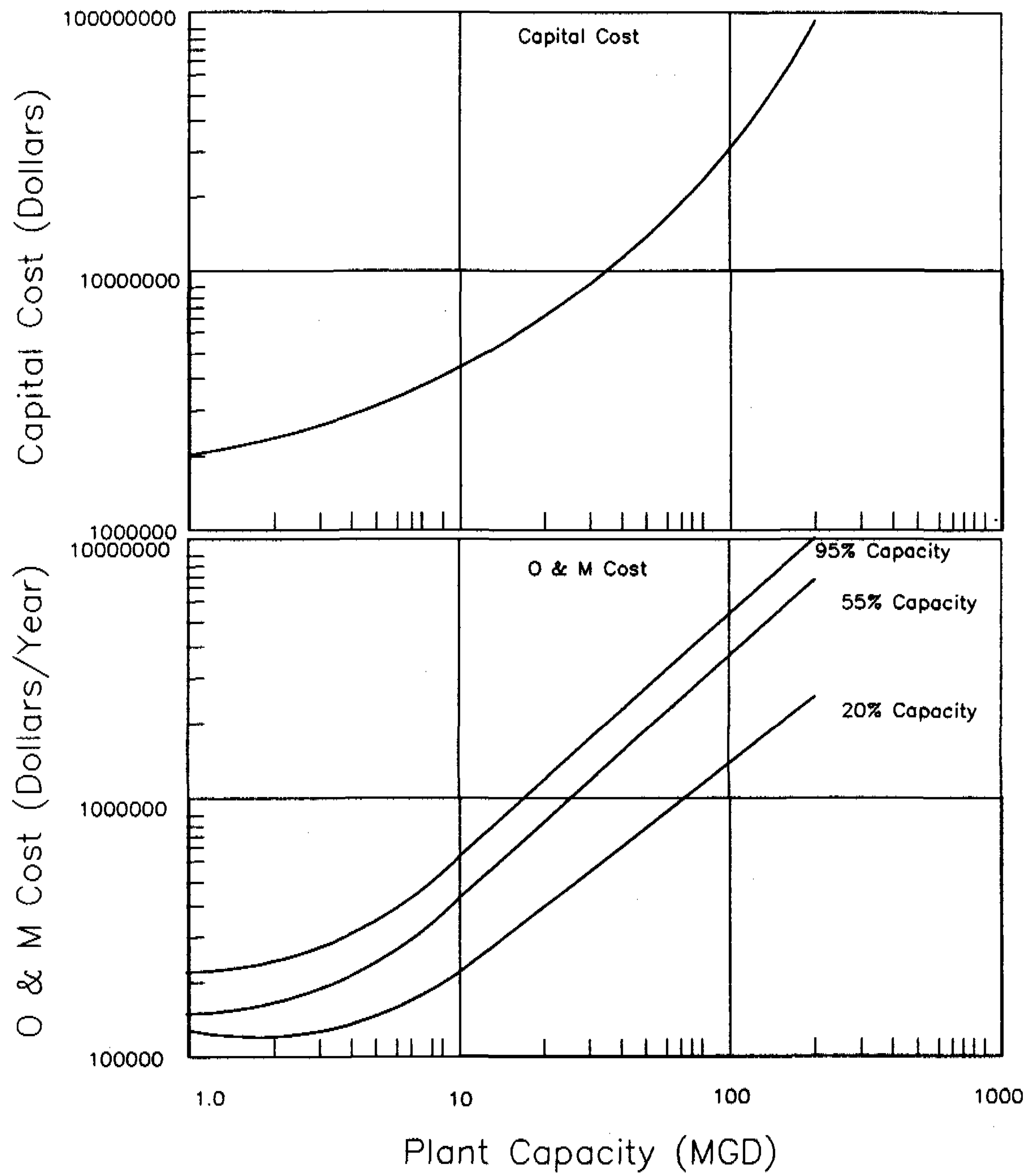
Most of the chemicals required are widely available with the exception of some polyelectrolytes. Availability should be examined in detail before a plant is designed with the planned use of chemicals.

2A.5 Operation and Maintenance

As untreated water flows through the various units, color, turbidity, tastes, odors and bacteria are removed from the surface water supplies. Additional precautions may include bar racks and course screens if floating debris and fish are a problem, aeration has been shown to be economical and beneficial for treatment of tastes and odors; plain sedimentation if the water is highly turbid; and softening if the water is high in hardness.

The cleaning of the tanks can be carried out either mechanically (i.e., a sludge removal device) or manually. Where manual labor is readily available, manual clean out should be considered. In manual cleaned basins, the time lapse between cleanings varies from a few weeks to a year or more. The sludge accumulated may be sluiced by a fire hose after the tank has been taken out of service and dewatered.

Figure 2A.1: Conventional Alum Coagulation Treatment Costs



(Source: Ref. 48)

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TABLE 2A.2

CONVENTIONAL ALUM COAGULATION TREATMENT

Process	Design Criteria
Raw water pumping	100 ft (30.5 m) TDH
Alum feed-liquid	50 mg/l design; 30 mg/l operating
Polymer feed	1 mg/l design; 0.2 mg/l operating
Rapid mix	60 sec detention; G = 900
Flocculation	30 min detention; G=80
Clarifers-rectangular	900 gpd/sq ft (1.525 m/h)
Gravity filtration-mixed media	5 gpm/sq ft (12.2 m/h)
Chlorine feed-gas chlorine	5 mg/l design; 2 mg/l operating
Product water pumping	300 ft (91.44 m) TDH

Source: Reference 48

2A.6 Control

Optimum floc formation requires that for alum the pH be within the range of 5.0-7.0. Sufficient alkalinity must be present for reaction with the coagulant. If sufficient alkalinity is not present in the water, lime is generally added. While they are not equally effective in all waters, some polyelectolytes, when used in conjunction with the common metal coagulants, yield large dense floc, which settles rapidly.

2A.7 Special Factors

Iron salts can operate over a wider pH range than alum and are generally more effective in removing color from water, but are usually more costly. Coagulants and dosages should be chosen on the basis of jar tests.

In the flocculation basins, when the floc particles have grown in size, they become weaker and more subject to being torn apart. So, tapered flocculation is frequently used. The around-the-end type of basin is commonly applied to plants with capacities below 76,000 m3/day, and the over-and-under type, with the advantage of more continuous turbulence, is applied where sufficient water head is available and land is limited.

If a circular settling tank is used, the diameter may be as large as 70 meters, but they are generally held to 30 meters or less in diameter to reduce wind effects.

Settled coagulation and backwash water have been disposed of without treatment, but these residues may promote buildup of deposits of "sludge banks" in the backwaters of slowly moving portions of streams. This discharge is not only aesthetically objectionable but also contains polluting concentrations of chemicals used in processing, and removed solids. Possible disposal/management procedures include: direct discharge to a receiving stream with adequate flow rate or drainage system, lagoons or sludge drying beds, hauling away for surface land applications, discharge into a municipal sewerage system, dewatering of sludge, and reclamation of alum or other useful constituents. Sludge production depends on coagulant dose, quantity of solids removed, and character of the water being treated (pH, salts, etc.). Sludge quantities are determined by testing.

3. DUAL MEDIA FILTRATION

3.1 Description

Gravity dual media (coal-sand) filtration is one of the most economical forms of granular media filtration. Granular media filtration involves the passage of water through a bed of heterogeneous filter media with resulting removal largely by straining, as with other filtration processes.

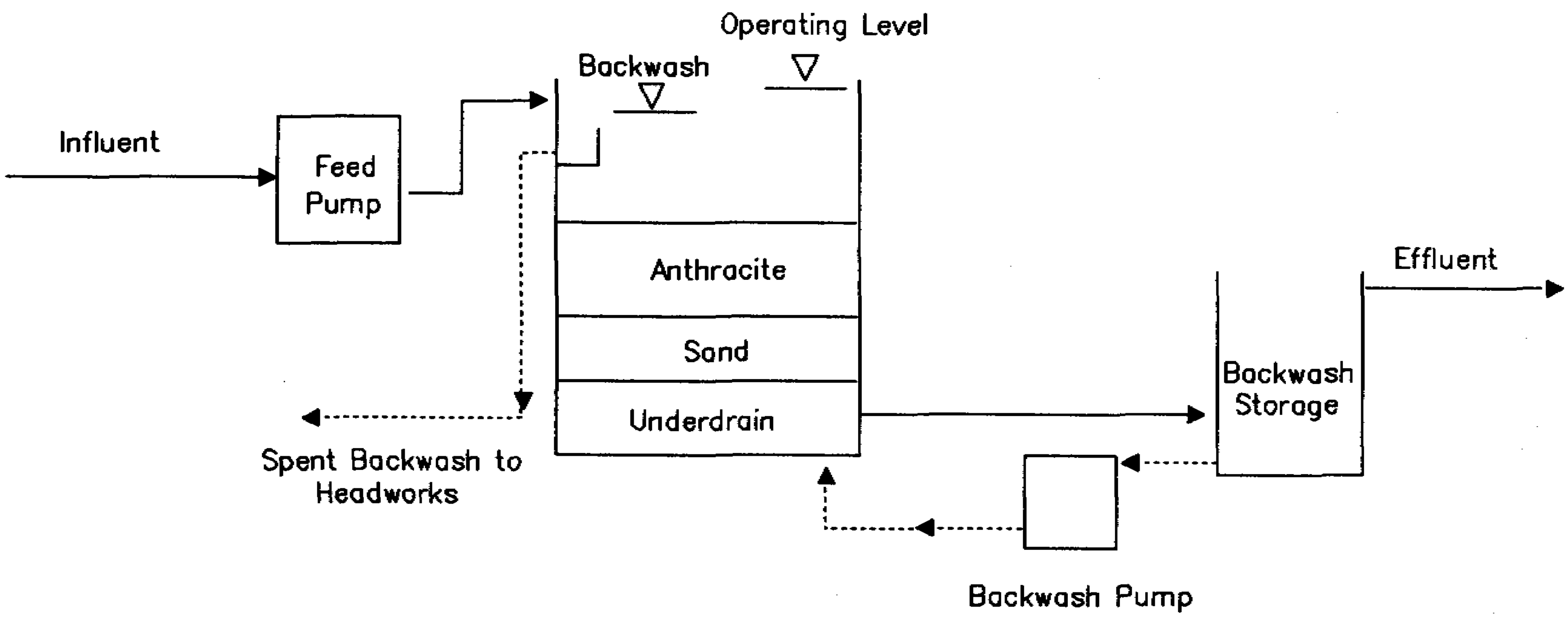
Dual media filtration involves the use of both sand and anthracite (coal) as filter media, with the anthracite being placed on top of the sand (See Figure 3.1) (2). Typical media sizes are shown on Table 3.1. It is used for the removal of residual biological floc in settled effluents from secondary wastewater treatment, and removal of residual chemical-biological floc after alum, iron or lime precipitation in water treatment plants. It is also used for tertiary or independent physical-chemical waste treatment in the U. S. and other countries.

The dual media filter consists of two layers - a top layer of anthracite and a bottom layer of sand. Gravity filters operate by either using the available head from the previous treatment unit, or by pumping to a flow split box after which the wastewater flows by gravity to the filter cells. Pressure filters utilize pumping to increase the available head.

A filter unit generally consists of a containing vessel, the filter media, structures to support the media, distribution and collection devices for influent, effluent and backwash water flows, supplemental cleaning devices, and necessary controls for flows, water levels and backwash sequencing.

Design criteria are as follows: filtration rate 2 to 8 gpm/ft^2 (5 to 20 $m^3/hr/m^2$); bed depth 24 to 48 inches (61 - 122 cm.), depth ratios of 1:1-4:1 sand to anthracite; 15 to 25

FIGURE 3.1 FLOW DIAGRAM OF DUAL MEDIA FILTRATION



(Source: Ref 2)

TABLE 3.1

TYPICAL COAL AND SAND DISTRIBUTION
BY SIEVE SIZE IN DUAL MEDIA BED

Coal Distribution by Sieve Size	
U.S. Sieve No.	Percent Passing Seive
4	99-100
6	95-100
14	60-100
16	30-100
18	0-50
20	0-5

Sand Distribution by Sieve Size	
U.S. Sieve No.	Percent Passing Seive
20	96-100
30	70-90
40	0-10
50	0-5

Source: Reference 48

gpm/ft^2 (37 to 62 $m^3/hr/m^2$); filter run length, 8 to 48 hours; and terminal head loss 6 to 15 ft.

3.2 Limitations

The economics are highly dependent on consistent pretreatment quality and smoothing flow modulations. Increasing suspended solids loading will reduce run lengths, and large flow variations will adversely effect effluent quality. Chemical pretreatment is often required.

There are a number of problems associated with dual and sand filters which can be alleviated or eliminated with mixed media. Furthermore, rapid rate filter systems can be easily converted to mixed media, using the same equipment and filter galleries. Operational changes may be desirable or required (see Section 2). Since mixed media operation will successfully remove and store solids from high turbidity waters, it is often not necessary to add settling basin capacity when plants are being expanded. There is no fixed distribution of grain sizes for mixed media operation. Table 3.2 presents sizes and applications for mixed media.

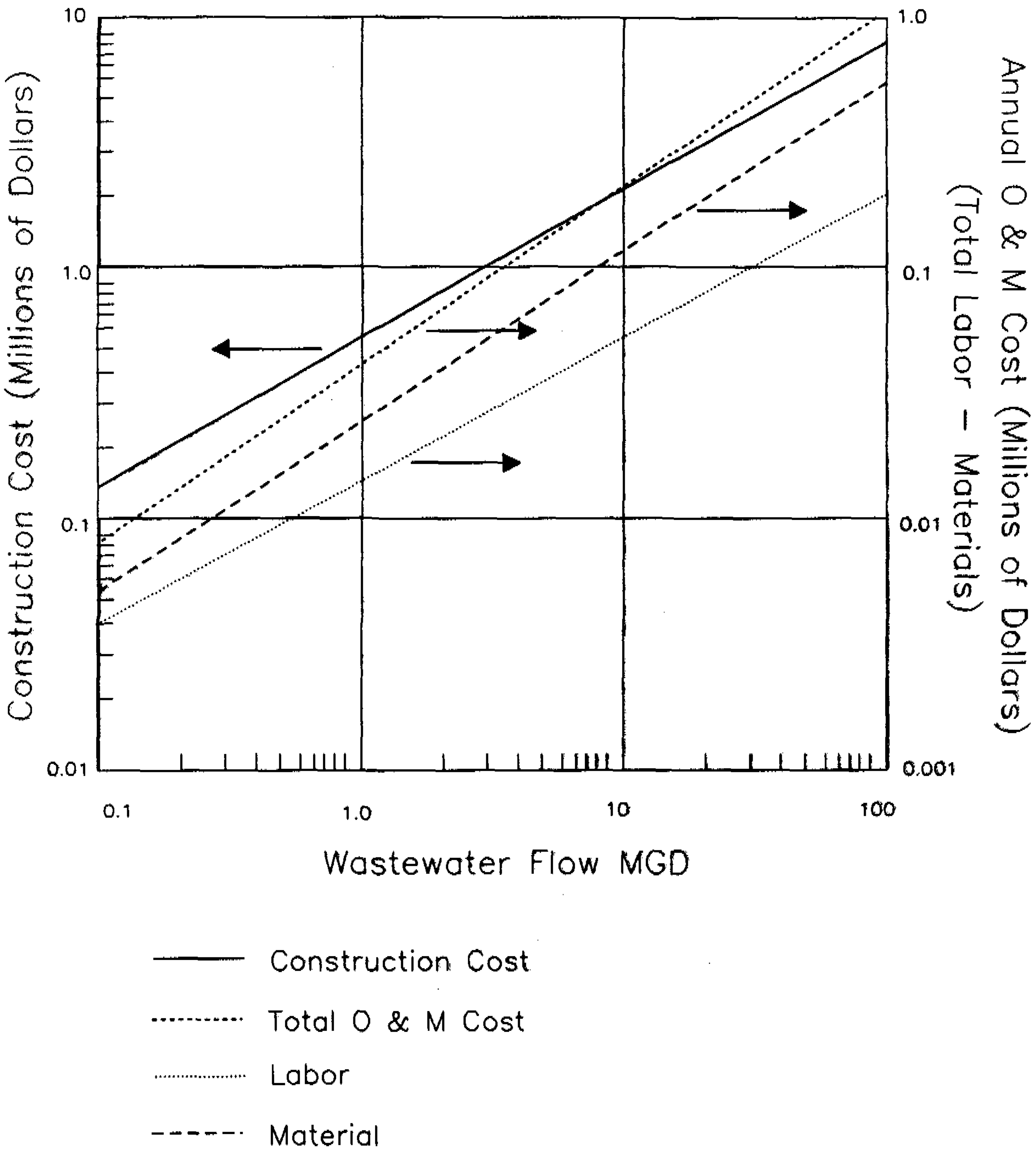
3.3 Costs

Construction cost includes facilities for backwash storage, all feed and backwash pumps, piping, and building (see Figure 3.3) (2,11). Operation and maintenance costs are shown in Figure 3.3).

3.4 Availability

This method will be limited to developing countries that can acquire anthracite cheaply. Also the higher skill and energy requirements for all high rate systems may limit applications.

Figure 3.3: Construction, Operation & Maintenance Cost for Dual Media Filtration.



(File: Martin46)

TABLE 3.2

ILLUSTRATIONS OF VARYING MEDIA DESIGN FOR VARIOUS TYPES OF FLOC REMOVAL

	Garnet		Silica Sand		Coal	
Type of Application	Size	Depth, Inches (MM)	Size	Depth, Inches (MM)	Size	Depth, Inches (MM)
Very heavy loading of fragile floc	-40 + 80*	8(203)	-20 + 40	12(305)	-10 + 20	22(559)
Moderate loading of very strong floc	-20 + 40	3(76)	-10 + 20	12(305)	-10 + 16	15(381)
Moderate loading of	-20 + 80	3(76)	-20 + 40	9(229)	-10 + 20	8(205)

* -40 + 80 = passing No. 4 and retained on No. 80 U.S. sieves.

Source: Reference 48

3.5 Operation and Maintenance

In dual-media and mixed-media beds, floc is stored throughout the bed depth to within a few inches of the bottom of the fine media.

Rapid sand, (see Section 2) and dual-media filters are cleaned by hydraulic backwashing (upflow) with potable water. Thorough cleaning of the bed makes it advisable in the case of single-medium filters and mandatory in the case of dual- or mixed-media filters to use auxiliary scour or so-called surface wash devices before or during the backwash cycle. Backwash flow rates of 15 to 20 gpm/sq ft (36.6 to 48.8 m/h) should be provided. A 20 to 50 percent expansion of the filter bed is usually adequate to suspend the bottom grains. The optimum rate of washwater application is a direct function of water temperature, as expansion of the bed varies inversely with viscosity of the washwater. For example, a backwash rate of 18 gpm/sq ft (43.9 m/h) at 68°F (20°C) equates to 15.7 gpm/sq ft (38.3 m/h) at 41°F (5°C) and 20 gpm/sq ft (48.8 m/h) at 95°F (35°C). The time required for complete washing varies from 3 to 15 minutes.

Following the washing process, water should be filtered to waste until the turbidity drops to an acceptable value. Filter-to-waste outlets should be through an air-gap-to-waste drain, which may require from 2 to 20 minutes, depending on pretreatment and type of filter. This practice was discontinued for many years, but modern recording turbidimeters have shown that this operation is valuable in the production of a high-quality water. Operating the washed filter at a slow rate at the start of a filter run may accomplish the same purpose. A recording turbidimeter for continuous monitoring of the effluent from each individual filter unit is of great value in controlling this operation at the start of a run, as well as in predicting or detecting filter breakthrough at the end of a run.

As backwashing begins, the sand grains do not move apart quickly and uniformly throughout the bed. Time is required for the sand to equilibrate at its expanded spacing in the upward flow of washwater. If the backwash is turned on suddenly, it lifts the sand bed bodily above the gravel layer, forming an open space between the sand and gravel. The sand bed then breaks at one or more points, causing sand boils and subsequent upsetting of the supporting gravel layers, so that the gravel section must be rebuilt. It is essential that the backwash valve open slowly.

The time from start to full backwash flow should be at least 30 seconds and perhaps longer, and should be restricted by devices built into the plant. This is frequently done by means of an automatically regulated master wash valve, controlled hydraulically or electrically and designed so that it cannot open too fast. Alternatively, a speed controller could be installed on the operator of each washwater valve.

Filters can be seriously damaged by slugs of air introduced during filter backwashing. The supporting gravel can be overturned and mixed with the fine media, which requires removal and replacement of all media for proper repair. Air can be unintentionally introduced to the bottom of the filter in a number of ways. If a vertical pump is used for the backwash supply, air may collect in the vertical pump column between backwashings. The air can be eliminated without harm by starting the pump against a closed discharge valve and bleeding the air out from behind the valve through a pressure air release valve. The pressure air release valve must have sufficient capacity to discharge the accumulated air in a few seconds.

Washwater may also be supplied by gravity flow from a storage tank located above the top of the filter boxes. Washwater supply tanks usually have a minimum capacity equal to a 7-minute wash for one filter unit, but may be larger. The bottom of the tank must be high enough above the filter wash troughs to supply water at the rate required for backwashing as determined by a hydraulic analysis of the washwater system. This distance is usually at least 10 feet (3.05 m), but more often is 25 feet (7.6 m) or greater. Washwater tanks should be equipped with an overflow line, and a vent for release and admission of air above the high-water level.

3.6 Special Factors

Normally filter systems include multiple filter compartments. This allows for the filtration system to continue to operate while one compartment is being backwashed. No less than four units should probably be designed into a typical system. In systems with two filters for example, the flow rate would be doubled when a filter is being backwashed.

Filtration systems can be constructed out of concrete or steel, with single or multiple compartment units. Systems can be manually or automatically operated.

Backwash sequences can include air scour and/or surface wash steps. Backwash water can be stored separately or in chambers that are integral parts of the filter unit. Backwash water can be pumped through the unit or can be supplied through gravity head tanks.

3.7 Recommendations

Dual or mixed media filtration should be considered for applications where high turbidities are common. Also, these systems are preferred when operating problems are consistent and can not be solved in other ways.

4. SLUDGE VACUUM FILTRATION

4.1 Description

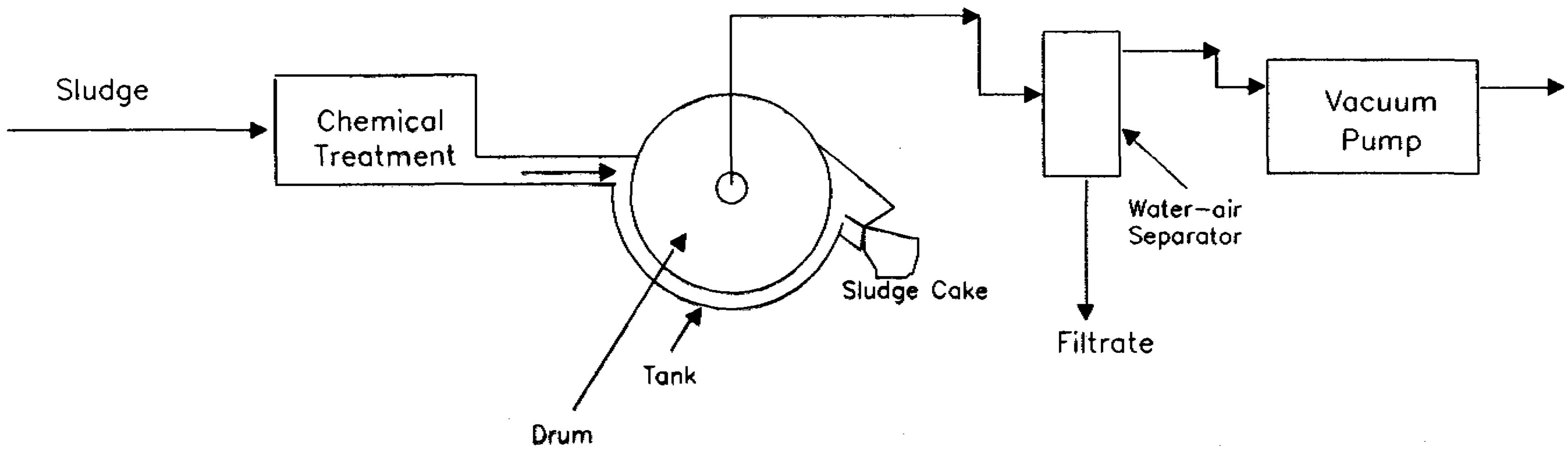
Vacuum filters are used to dewater sludges so as to produce a cake having the physical handling characteristics and moisture content required for subsequent processing (see Figure 4.1).

Solids capture ranges from 85 to 99.5% and cake moisture is usually 60 to 90% depending on feed type, solids concentration, chemical conditioning, machine operation and management. Dewatered cake is suitable for landfill, heat drying, incineration or land spreading.

This technology contains a rotary vacuum filter which consists of a cylindrical drum rotating partially submerged in a vat or pan of conditioned sludge. The drum is divided radially into a number of different sections which are connected through internal piping to ports in a valve body (plate) at the hub. This plate rotates in contact with a fixed valve plate with similar ports, which are connected to a vacuum supply, a compressed air supply and an atmospheric vent. As the drum rotates each section is thus connected to the appropriate service. Various operating zones are thus encountered during a complete revolution of the drum. In the pickup or form section, vacuum is applied to draw liquid through the filter covering (media) and form a cake or partially dewatered sludge. As the drum rotates the cake emerges from the liquid sludge pool, while suction is still maintained to promote further dewatering. A lower level of vacuum often exists in the cake drying zone. If the cake tends to adhere to the media, a scraper blade may be provided to assist removal.

The three principal types of rotary vacuum filters are the drum type, coil type and belt type. The filters differ primarily in the type of covering used and the cake discharge mechanism

FIGURE 4.1 GENERALIZED DIAGRAM OF SLUDGE VACUUM FILTRATION



(Source: Ref 2)

(File:Martin03)

employed. Cloth media are used on drum and belt types while stainless steel springs are used on the coil type. Infrequently, metal media is used on belt types. The drum filter also differs from the other two in that the cloth covering does not leave the drum but is washed in place, when necessary. The design of the drum filter provides considerable latitude in the amount of cycle time devoted to cake formation, washing and dewatering; while it minimizes inactive time.

The coil type vacuum filter uses two layers of stainless steel coils arranged in corduroy fashion around the drum. After a dewatering cycle, the two layers of springs leave the drum and are separated from each other so that the cake is lifted off the lower layer of springs and discharged from the upper layer. Cake release is essentially free of problems. The coils are then washed and reapplied to the drum.

Media on the belt type filter leave the drum surface at the end of the drying zone and pass over a small diameter discharge roll to facilitate cake discharge. Washing of the media then occurs before it returns to the drum and to the vat for another cycle.

Design criteria for vacuum filtration are as follows: typical loadings in pounds dry solids/h/ft^2 are 7 to 15 (34 to 74 $kg/m^2/hr$) for raw primary sludges, and 3.5 to 5 (17 to 24.5 $kg/m^2/hr$) for mixed digested sludges. The loading is a function of feed solids concentration, subsequent processing requirements and chemical preconditioning.

4.2 Limitations

Vacuum filters are generally used in larger facilities where space is limited, or when incineration is necessary for maximum volume reduction (see Figure 4.1).

Chemical conditioning costs can sometimes be large if a sludge is hard to dewater.

4.3 Costs

The typical construction cost of sludge vacuum filtration lime and biological sludges options are shown in Figure 4.2. (2,11). Operation and maintenance of both options are shown in Figures 4.3 and 4.4 (2,11).

4.4 Availability

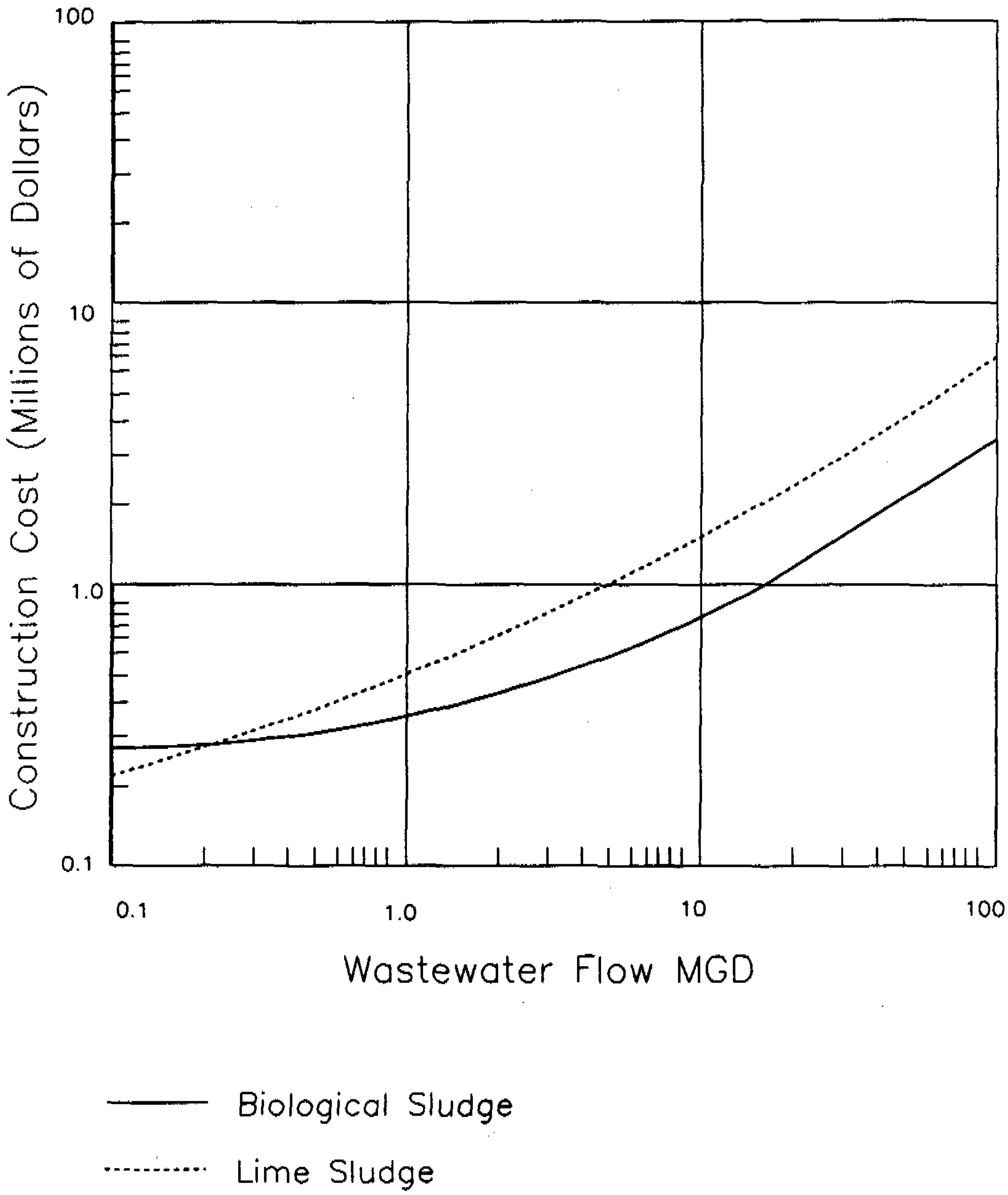
Vacuum filters possess many moving parts. Available services for maintenance from factory representatives and availability of spare parts should be carefully checked in the area of interest before selecting this technology. Frequent replacements of filter media, cloth, wire, etc., is required and these must be kept in adequate supply for as many replacements as necessary between shipments. If shipments are expected to be especially uncertain, it may be necessary to keep two such supplies.

4.5 Operation and Maintenance

Sludge vacuum filters require high operating skill and would be appropriate in areas of high population and technical skill in developing countries. Operation is sensitive to type of sludge and conditioning procedures. As raw sludge ages (3 or 4 hours) after thickening, vacuum filter performance decreases. Poor release of the filter cake from the belt is occasionally encountered.

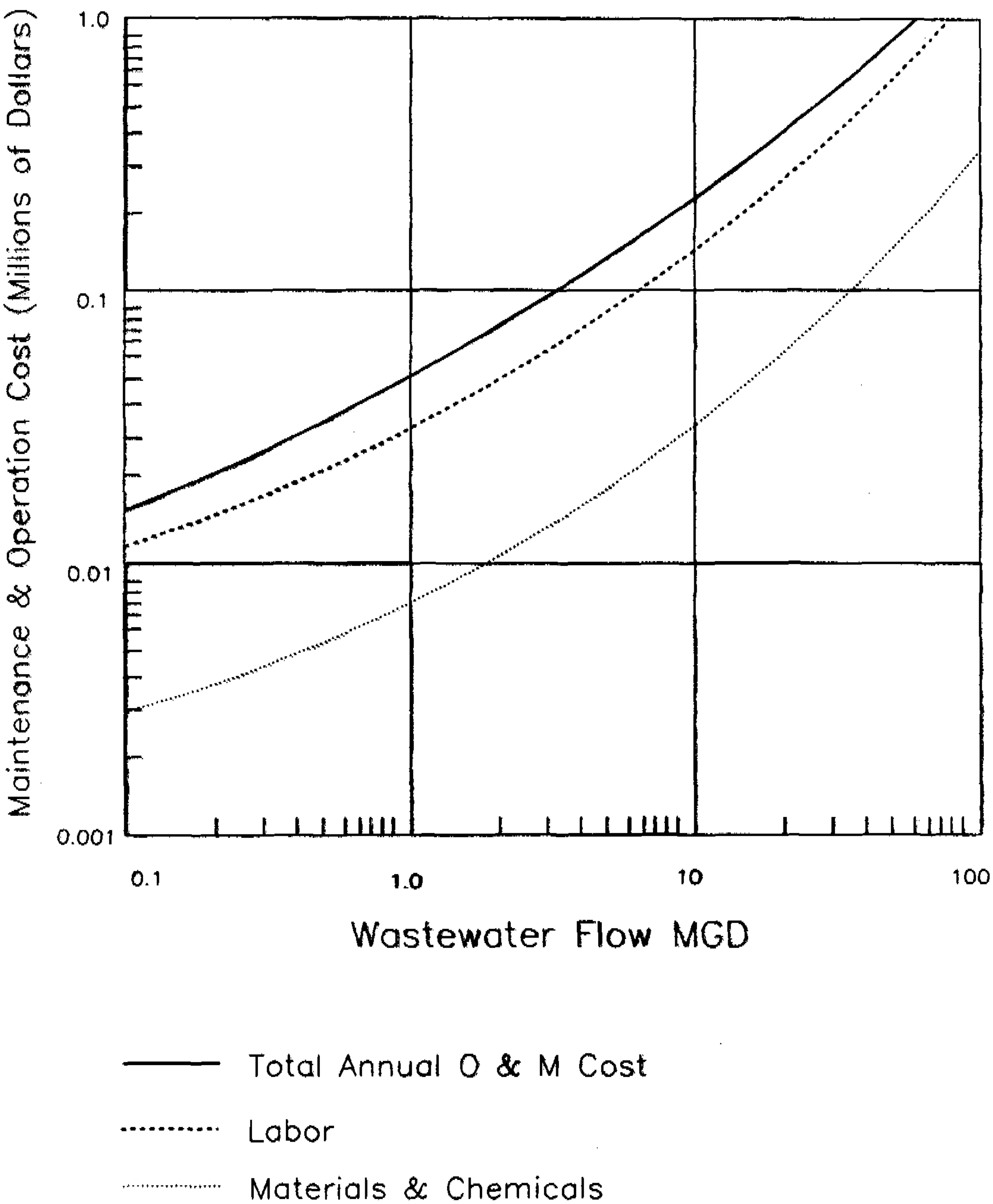
When sludge is difficult to filter conditioning may be required. Information on the types of chemicals which may be used with specific media may be obtained from manufacturers. Dosages of such chemicals are usually determined be testing at

Figure 4.2: Construction Cost of Sludge Vacuum Filtration (Lime and Biological Sludges.)



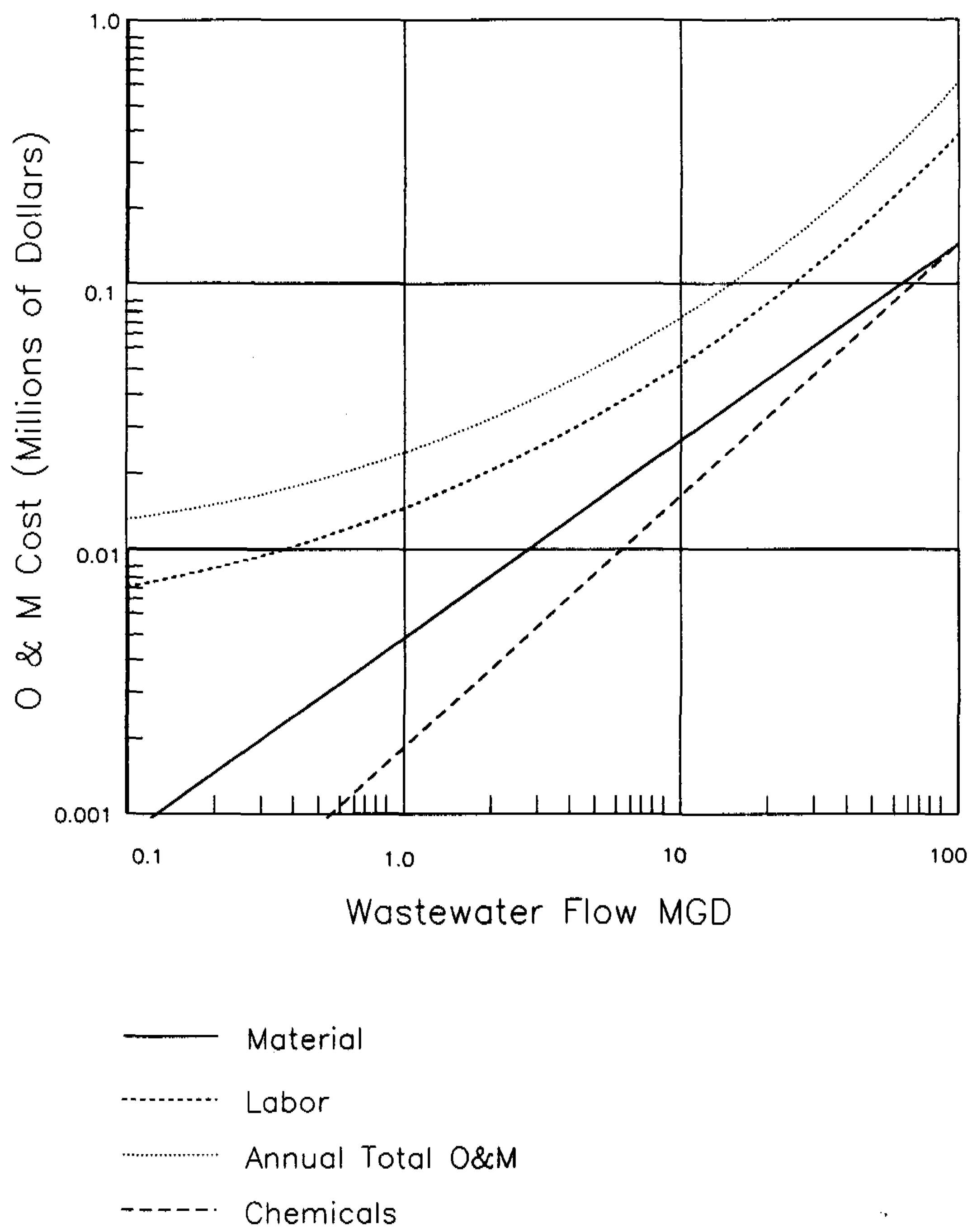
(File: Martin47)

Figure 4.3: Annual Operation & Maintenance Cost for Sludge Vacuum Filtration (Lime Sludge)



(File: Martin48)

Figure 4.4: Annual Operation & Maintenance Cost for Sludge Vacuum Filtration (Biological Sludge).



(File: Martin22)

the location with the actual sludge being produced. Unsatisfactorily treated quantities produced during testing may be recirculated to the plant for additional treatment.

Vacuum pumps, chain drives, media scraping mechanisms, and the media itself require frequent maintenance. A preventive maintenance program is required.

4.6 Control

Large doses of lime, ferric chloride (and even polyelectrolytes in some cases) may be required for good sludge yields from the filters. Frequent washings of drum filter media may be required. Remedial measures are frequently required to obtain operable cake releases from belt filters. Operating training is required to maintain a high level of reliability.

4.7 Special Factors

A great many types of filter media are available for the felt and drum filters. There is some question whether increases in yield due to operating vacuums greater than 15 inches of mercury are justifiable. The cost of a greater filter area must be balanced against the higher power costs for higher vacuums. An increase from 15 to 20 inches however, about 25%, (38 to 51 cm) of vacuum is reported to have provided about 10% greater yield in 3 full-scale installations.

Vacuum filters, because of their large energy (10,000 to 40,000 kWh/yr/MGD - 230,000 to 910,000 kWh/yr/m^3/sec) and high user skill requirements, may have limited applicability in most areas of developing countries. On the other hand, processed sludge usually filtered to reduce costs of hauling, can be used successfully for soil conditioning.

Chemical conditioning is often employed to agglomerate a large number of small particles. It is almost universally applied with mixed sludges.

4.8 Recommendations

This is the most common method of sludge dewatering in the United States. It should be applied to chemical and biological sludges in those cases where recovery of chemicals for reuse, or soil conditioning potential is desired. Its applicability to developing countries is to areas of high population and technical expertise in order for the technology to perform reliably. Vacuum filtration would be used instead of drying beds or other less complex technology when high volumes of sludge are being produced.

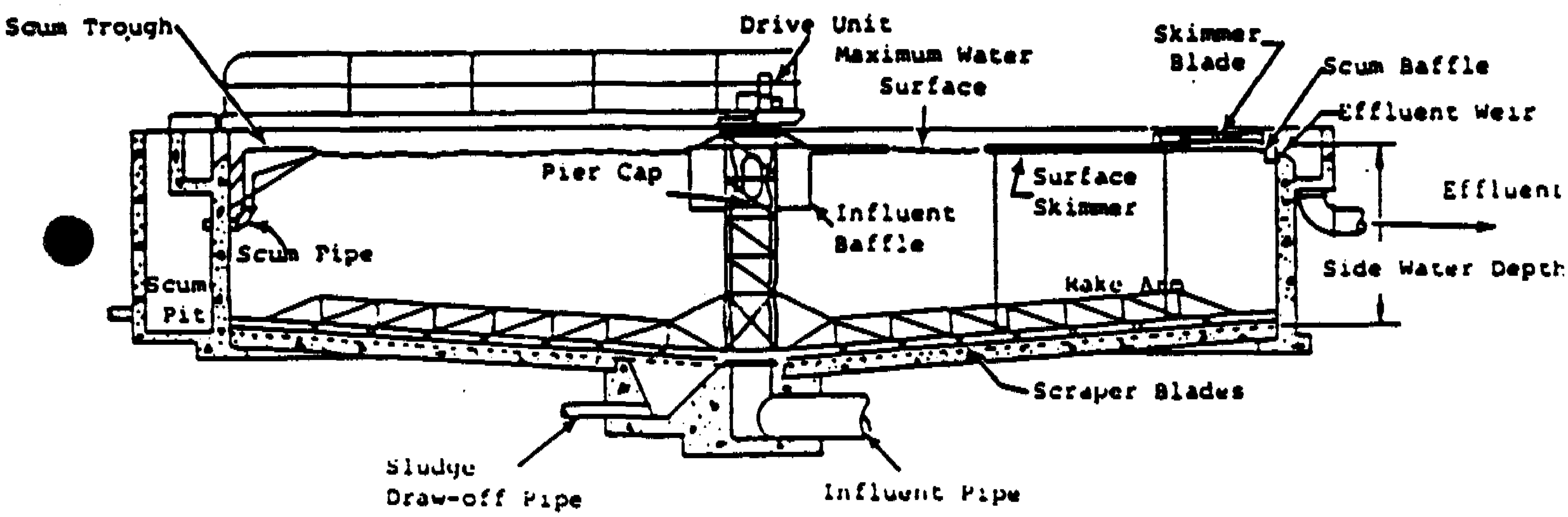
5. SEDIMENTATION - CIRCULAR PRIMARY CLARIFIER

5.1 Description

Clarifiers with the criteria suggested in this Section are used for solids removal from waters contaminated to the degree of raw sewage, or highly contaminated raw water supplies. These units provide removal of readily settleable solids and floating material to reduce suspended solids content. They can accept high solids loading and are generally employed as a preliminary step to further processing, or following treatment by softening or coagulation and flocculation. A circular clarifier cross section is shown in Figure 5.1.

A conical bottom (about 1 inch per ft slope; about 8%) is equipped with a rotating mechanical scraper (see Figure 5.1) that plows sludge to a central hopper. An influent feed in the center distributes the influent radially near the top, and a peripheral weir overflow system carries the effluent. Floating scum is trapped inside a peripheral scum baffle and squeegeed into a scum discharge box. The unit contains a center motor-driven turntable drive supported by a bridge spanning the top of the tank, or supported by a vertical steel center pier. The turntable gear rotates a vertical cage or torque tube, which in turn rotates the truss arms. The truss arms carry multiple flights (plows) on the bottom chord which are set at a 30 degree angle and literally "plow" heavy fractions of sludges and grit along the bottom slope toward the center blowdown hopper. An inner diffusion chamber receives influent flow and distributes this flow inside of the large diameter feedwell skirt. Approximately 3% of the clarifier surface area is used for the feed well. The depth of the feed wells are generally about one-half of the tank depth. The center sludge hopper should be less than 2 ft deep and less than 4 ft in cross section.

Figure 5.1: A Primary Circular Clarifier



(Source: Ref. 2)

Design criteria include: Surface loading rates from 500 to 1200 gal/d/ft^2 (21 to 50 $m^3/d/m^2$) for untreated wastewater; 360 to 600 (15 to 25) for alum floc; 540 to 800 (22 to 33) for iron floc; 540 to 1200 (22 to 50) for lime floc. Detention times are usually between 1.5 to 3 hours. Weir loadings are 10,000 to 30,000 gal/d/lineal ft (120 to 360 $m^3/d/m$). Sludge collector tip speed is 10 to 15 ft/min (3 to 4.6 m/min). Heads of 2-3 ft (0.61 to 0.9 m) of water are required to overcome losses at the inlet and effluent controls and in connecting pipes. Forward or radical velocity should be less than 9-15 times the particle settling velocity to avoid scour. Scum handling equipment should be sized for 6 ft3/Mgal (45 m^3/Mm^3). Sludge pumping rates range between 2,500 and 20,000 gal/d/Mgal ($m^3/d/Mm^3$) depending on chemical addition and service (2).

5.2 Limitations

The maximum diameter of a circular clarifier is 200 ft (61 m). Larger tanks are subject to unbalanced radial diffusion and wind action, both of which can reduce efficiency. Horizontal velocities in the clarifier must be limited to prevent "scouring" of settled solids from the sludge bed and eventual escape in the effluent.

Circular clarifiers have a larger land use requirement than rectangular clarifiers.

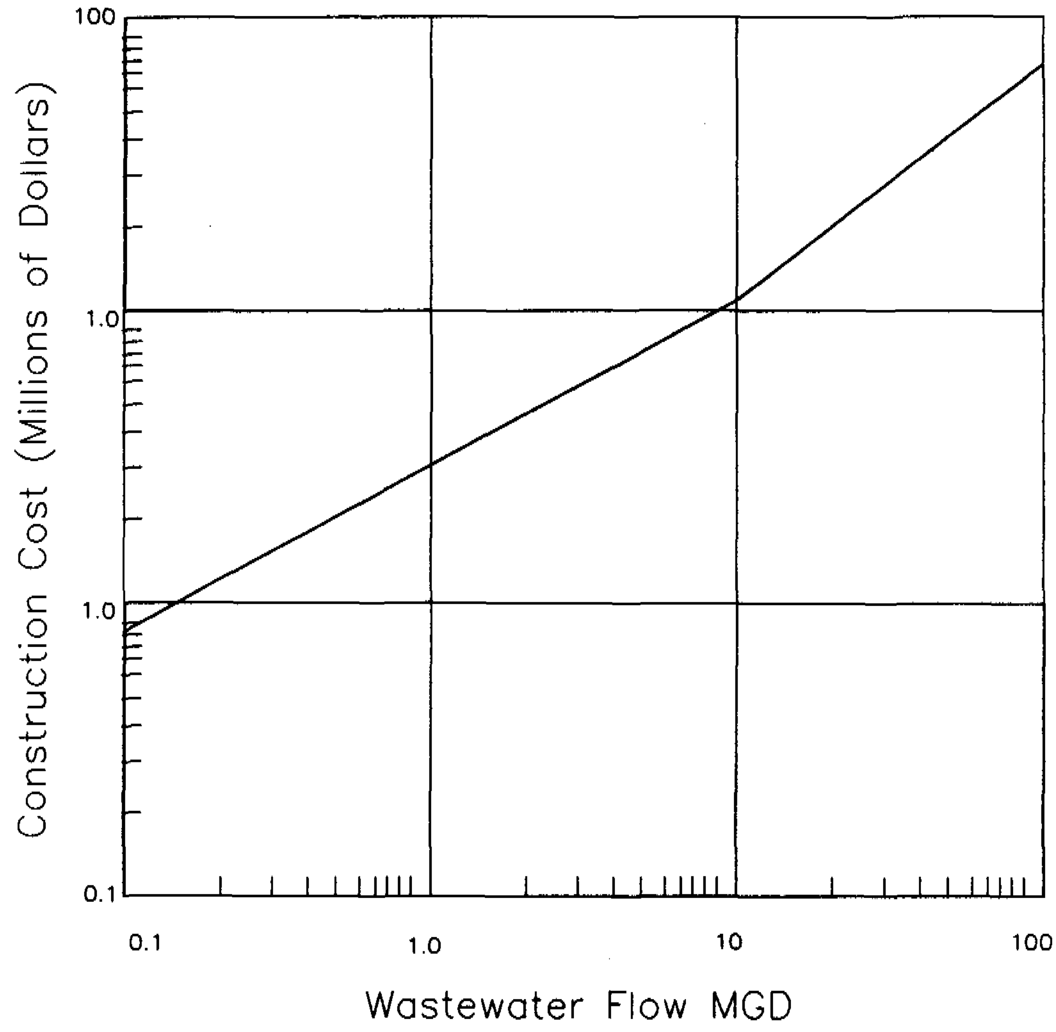
5.3 Costs

Figures 5.2 and 5.3 show the construction and operation and maintenance costs for circular clarifiers (2,11).

5.4 Availability

This technology is widely used in the United States and throughout the world. It can be applied to treatment systems of

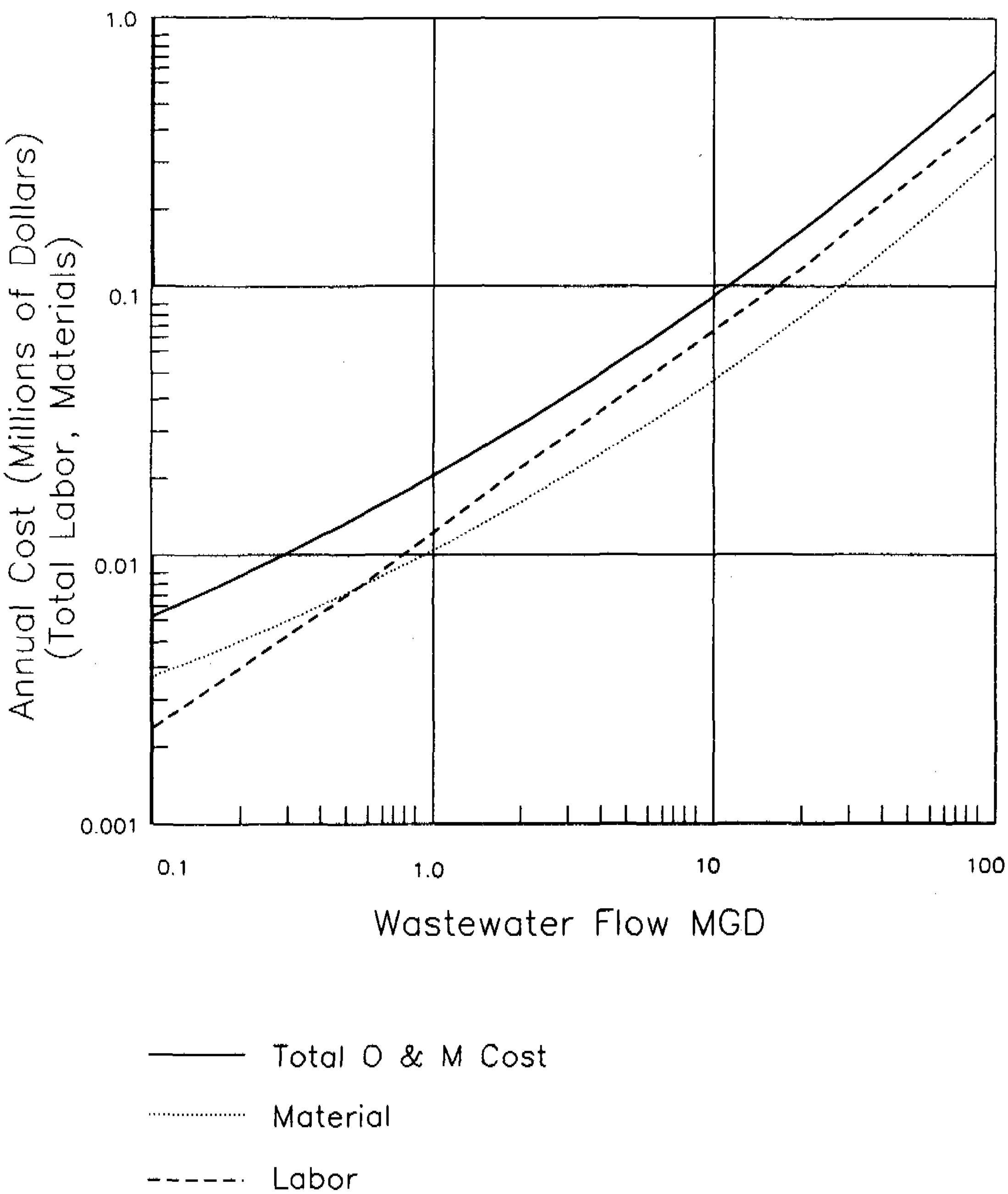
Figure 5.2: Construction Cost for Primary Clarifier
(Circular with Pump)



(Source: Ref. 2 & 5)

(File: Martin50)

Figure 5.3: Operation & Maintenance Cost for Primary Clarifier—Circular



(File: Martin51)

any size in developing countries. There is a variety of mechanical parts required, but the same is true for rectangular clarifiers. Good, reliable performance would rest on the availability of spare parts in developing countries. The alternative is intermittent removal of sludge (most of the mechanism is for removal of accumulated solids). A spare parts inventory should be kept for all mechanical processes at any plant.

5.5 Operation and Maintenance

Primary clarification involves a period of quiescence (15 to 45 min) in a basin (depths of 10-15 ft; 3 to 4.6 m) where most of the settleable solids fall out of suspension by gravity; a chemical coagulant may be added to enhance settling. The solids are mechanically collected on the bottom and pumped as a sludge underflow.

The most important aspect of operation is regular, frequent removal of accumulated sludge solids. Typically continuous removal mechanisms provide for better process performance than manual systems. Mechanical removal systems require regular maintenance for the chain drives, rakes, collectors and pumps. Scum (floating solids) which is carried out of the sedimentation tanks will significantly impact adversely the effluent quality. Scum collecting mechanisms require very frequent attention (in cases of high solids, perhaps hourly), especially to remove accumulated deposits at overflows and other collecting points usually near mechanical parts. Accumulating floating solids may jam mechanisms and even cause failure of parts and drives.

5.6 Reliability

Generally, reliability for circular primary clarifiers is high. However, clarification of solids into a packed central mass may cause collector arm stoppages. Attention to the design of the center area bottom slope, number of arms and center area scraper blade design is required to prevent such problems.

Sludge must be removed regularly. Often the failure of clarifiers to meet effluent expectations is due in turn to the failure of sludge removal equipment.

5.7 Special Factors

Two short auxiliary scraper arms are added perpendicular to the two long arms on large tanks. This makes practical the use of deep spiral flights which aid in center region plowing where ordinary shallow straight plows are nearly useless. Peripheral feed systems are sometimes used instead of central feed. Also, central effluent weirs are used sometimes. Flocculating feed wells may also be provided if coagulants are to be added to assist sedimentation.

Coagulants such as alum, ferrous sulfate and lime may be added to aid sedimentation. The dosage may be determined using jar tests.

5.8 Recommendations

This technology is widely used with high success in the United States and can be easily used in cities in developing countries.

Circular clarifiers require more land area, and some feel that they are more reliable. Rectangular tanks may be constructed in less space, and may be designed in "common wall" fashion with other plant processes. They are especially effective for small plants which must be built in confined spaces, say for industrial waste treatment. Equipment for circular tanks is more readily available since many manufacturers produce such hardware. Replacement parts and manufacturer service programs may be more readily available. Package plants are often designed with the circular tank as the basis (together with chemical treatment, for example).

6. RECTANGULAR PRIMARY CLARIFIER

6.1 Description

Rectangular clarifiers are used for the removal of settleable solids and floating material to reduce TSS and BOD and to treat raw water with high turbidity. High solids loadings is generally employed as a preliminary step to further may be processed. Rectangular tanks also lend themselves to "nesting" (common wall construction) with preaeration tanks in water treatment plants and aeration tanks in activated sludge plants. A cross section of a rectangular clarifier is shown in Figure 6.1.

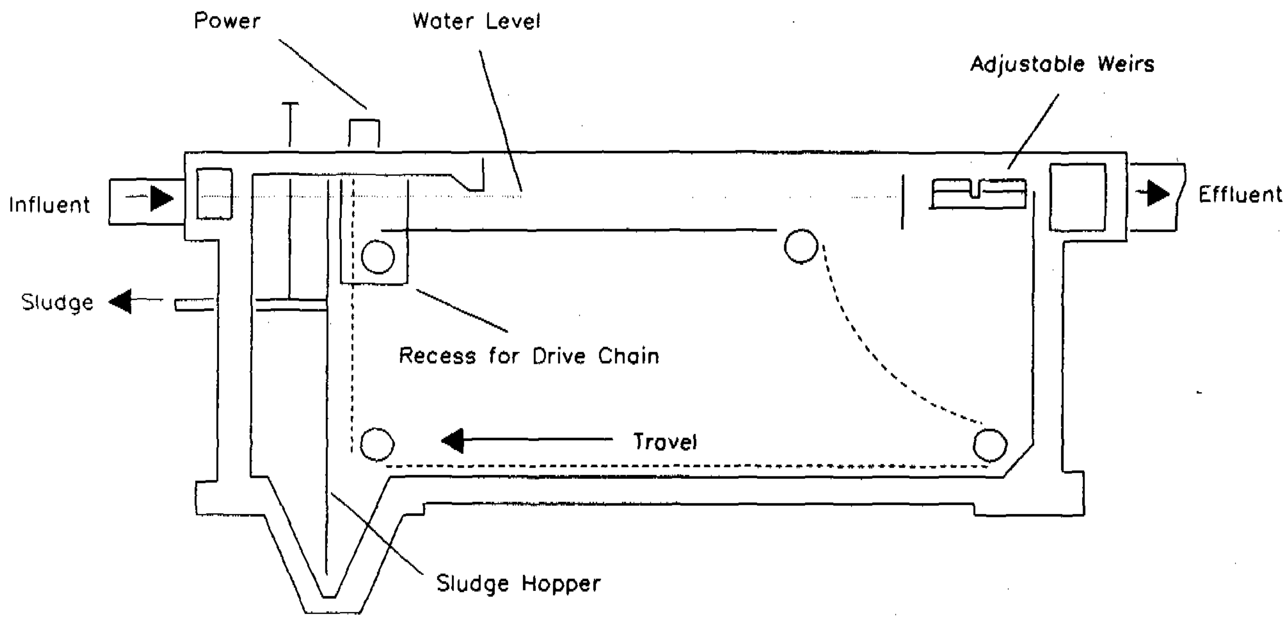
Efficiently designed and operated primary clarifiers should remove 50 to 65% of the TSS and 25 to 40% of the BOD while producing a sludge solids concentration of about 1 to 5%. Skimmings volume rarely exceeds 1.0 ft^3/Mgal (7.5 m^3/Mm^3).

Detention times for clarification depend on surface loading rates (see Table 6.1). Involves a relatively long period of quiescence in a basin (depths of 10-15 ft; 3 to 4.6 m) where most of the settleable solids fall out of suspension by gravity; a chemical may also be added to the process influent. The solids are mechanically collected on the bottom and pumped as a sludge underflow.

The maximum length of rectangular tanks has been approximately 300 ft (about 90 m). Where widths of 20 ft (6 m) are required, multiple bays with individual cleaning equipment may be employed, thus permitting tank widths of up to 80 ft (24 m) or more. Influent channels and effluent channels should be located at opposite ends of the tank.

Sludge removal equipment usually consists of a pair of endless conveyor chains (See Figure 6.1). Attached to the chains at ten foot intervals are wooden cross pieces or flights,

FIGURE 6.1 TYPICAL RECTANGULAR PRIMARY CLARIFIER



(Source: Ref 2)

(File:Martin18)

TABLE 6.1

TYPICAL DESIGN INFORMATION FOR PRIMARY SEDIMENTATION TANKS[a]

Item	Value: Range	Value: Typical
Primary settling floolowed by secondary treatment:		
Detention time,h	1.5-2.5	2.0
Overflow rate, $m^3/m^2 \cdot d$		
Average flow	32-48	
Peak flow	80-120	100
Weir loading, m^3/md	125-500	250
Dimensions		
Primary settling with waste activated-sludge return:		
Detention time, h	1.5-2.5	2.0
Overflow rate, $m^3/m^2/m \cdot d$		
Average flow	24-32	
Peak flow	48-70	60
Weir loading, $m^3/m \cdot d$	125-500	250
Diemnsions		

*Comparable data for secondary clarifiers are presented in Table 10-7

Note: $m^3/m^2 \cdot d \times 24.5424 = gal/ft^2 \cdot d$
$m^3/m \cdot d \times 80.5196 = gal/ft \cdot d$

Source: Reference 4

extending the full width of the tank or bay. Linear conveyor speeds of 2 to 4 ft/min (0.61 to 1.2 m/min) are common. The settled solids are scraped to sludge hoppers in small tanks and to transverse troughs in large tanks. The troughs in turn are equipped with cross collectors, usually of the same type as the longitudinal collectors, which convey solids to one or more sludge hoppers. Screw conveyors have been used for the cross collectors.

Design criteria include: Surface loading rates equal to 600 to 1200 gal/d/ft^2 (25 to 49 m^3/day/m^2) for untreated wastewater; 360 to 600 (15 to 25) for alum floc; 540 to 800 (22 to 33) for iron floc; 540 to 1200 (22 to 49) for lime floc. Detention times are usually between 1.5 to 3 hours. Weir loadings are 10,000 to 30,000 gal/d/lineal ft (120 to 360 m^3/day/m). Individual bays of rectangular tanks should have a length to width ratio of at least 4. Forward velocities should be less than 9-15 times settling velocity to avoid scour. Scum handling equipment should be sized for 6 ft^3/Mgal (45 m^3/Mm^3) of free decanted water. Sludge pumping rates range from between 2,500 to 20,000 gal/d/Mgal (m^3/day/m^3), depending upon chemical addition and service. Other design information is given in Table 6.1.

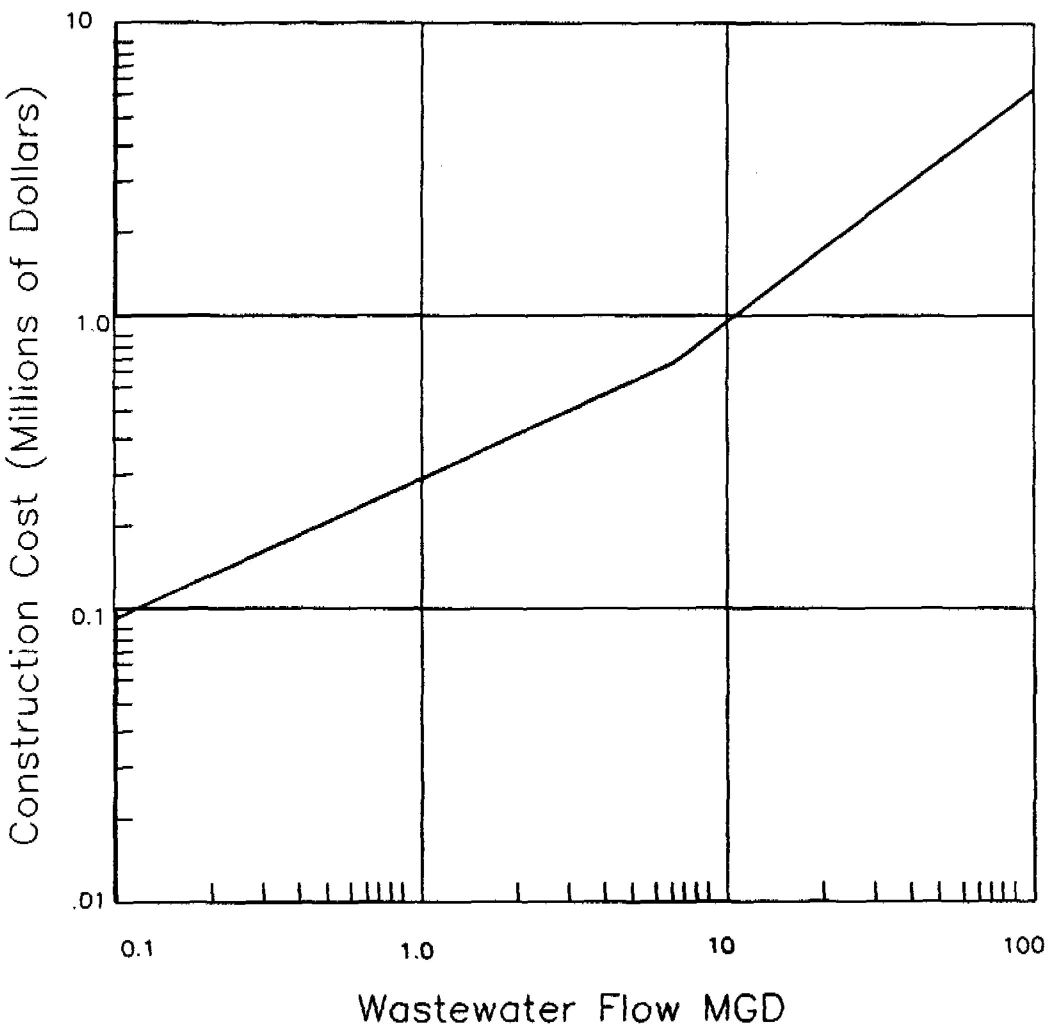
6.2 Limitations

Horizontal velocities in the clarifier must be limited to prevent "scouring" of settled solids from the sludge bed and their eventual escape in the effluent.

6.3 Costs

See Figures 6.2 and 6.3 for construction and operation and maintenance costs respectively (2,5,11).

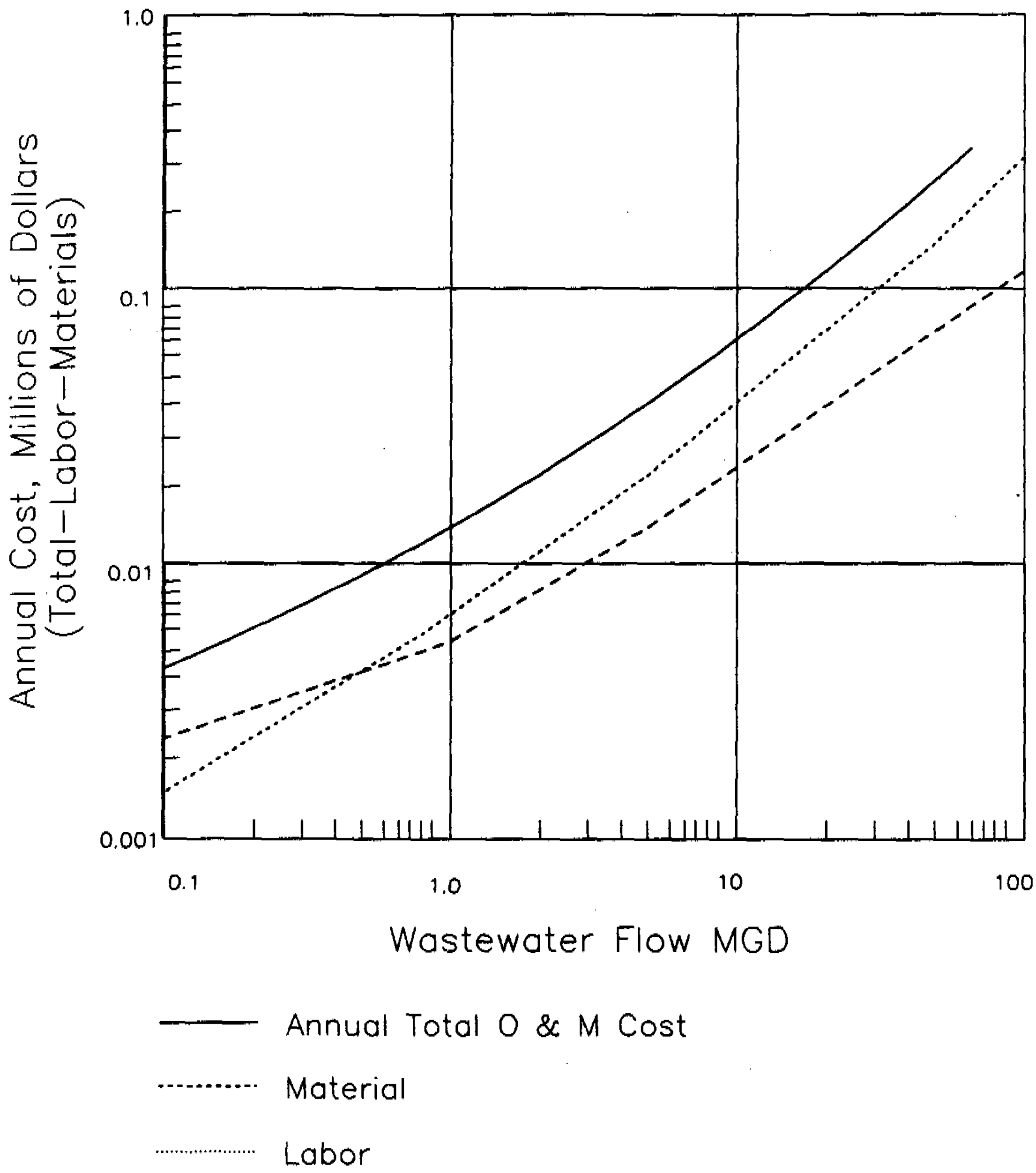
Figure 6.2: Construction Cost of Primary Clarifier Rectangular



(Source: Ref. 2 & 5)

(File: Martin54)

Figure 6.3: Operation & Maintenance Cost for Primary Clarifier (Rectangular with Pump)



(Source: Ref. 2 & 5)

(File: Martin55)

6.4 Availability

This technology is widely used in the United States and throughout the world. It can be applied to treatment systems in virtually any size town in Latin America and the Caribbean. This and many other processes require mechanical parts replacement on a fairly regular basis. It is necessary to keep a spare parts inventory to provide for emergency repairs.

6.5 Operation and Maintenance

Sedimentation systems, including rectangular clarifiers often fail in those cases where sludge collection systems are used. The sludge collection mechanisms require frequent cleaning and an active preventive maintenance program.

Scum is usually collected at the effluent end of the rectangular tanks by the flights returning at the liquid surface. The scum is moved by the flights to a point where it is trapped by baffles before removal or it can also be moved along the surface by water sprays. The scum is then scraped manually up an inclined apron, or it can be removed hydraulically or mechanically, and for this process a number of means have been developed (rotating slotted pipe, transverse rotating helical wiper, chain and flight collectors and scum rakes).

Tanks may also be cleaned by a bridge-type mechanism which travels up and down the tank on rails supported on the sidewalls. Scraper blades are suspended from the bridge and are lifted clear of the sludge on the return travel.

6.6 Control

Reliability for rectangular clarifiers is generally high. However, broken links in collector drive chains can cause outages. Plugging of sludge hoppers has also been a problem when cross collectors are not provided.

6.7 Special Factors

Coagulants such as alum, ferrous sulfate and lime may be added to aid sedimentation. The dosage is determined from jar tests.

Chemicals are almost always added with rapid mix systems. In the case of water treatment, flocculation is typically ahead of the settling process for effectiveness.

6.8 Recommendations

Multiple rectangular tanks require less area than multiple circular tanks and for this reason are used where ground area is at a premium. However, they require relatively large space for the level of treatment achieved. See recommendations for circular clarifers.

7. UPFLOW SOLIDS CONTACT CLARIFIER (FILTER)

7.1 Description

Upflow solids contact clarifiers combine mixing, coagulation and flocculation, liquid-solids separation and sludge removal into a single unit process.

These units eliminate the need for flocculators and settling tanks. The process should be applied when raw water with low turbidity (up to 50 JTU), has no more than 150 mg/liter of suspended solids.

Many plants, especially in Brazil, use this type of process. The technology came to Brazil from Russia (See Figure 7.1) (9). These filters are designed for rates of filtration between 120 and 150 $m^3/day/m^2$.

7.2 Limitations

If flow-through rates become too high, fine bed material will be lost with the treated effluent over the top of the upflow unit.

7.3 Costs

See Figures 7.2 and 7.3. (5).

7.4 Availability

Because of the simplicity and low cost of this technology, it should have widespread use throughout developing countries.

7.5 Operation and Maintenance

Upflow filters possess a major advantage in that the fine

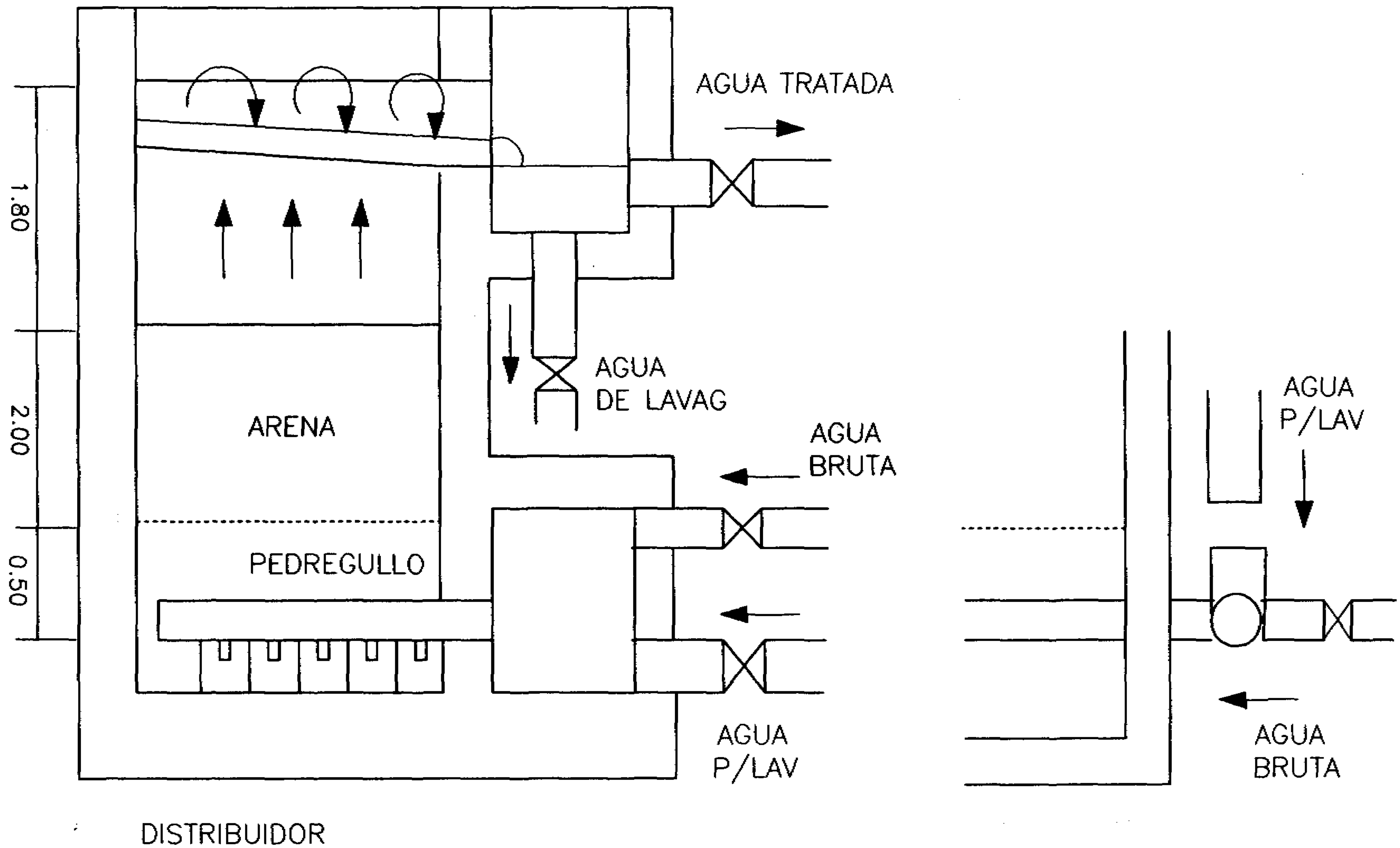
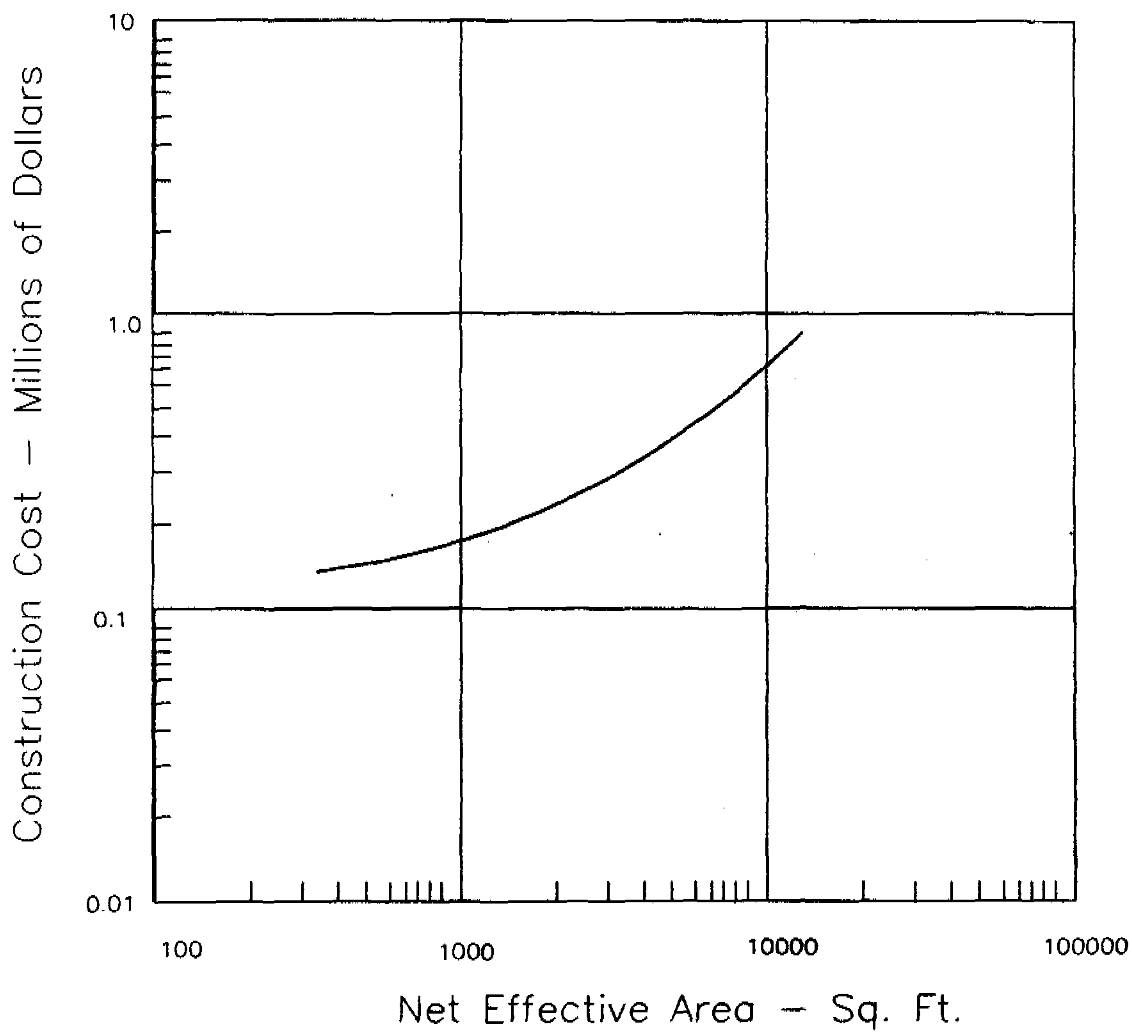


FIGURE 7.1: Generalized Diagram of Upflow Solids Contact Clarifier

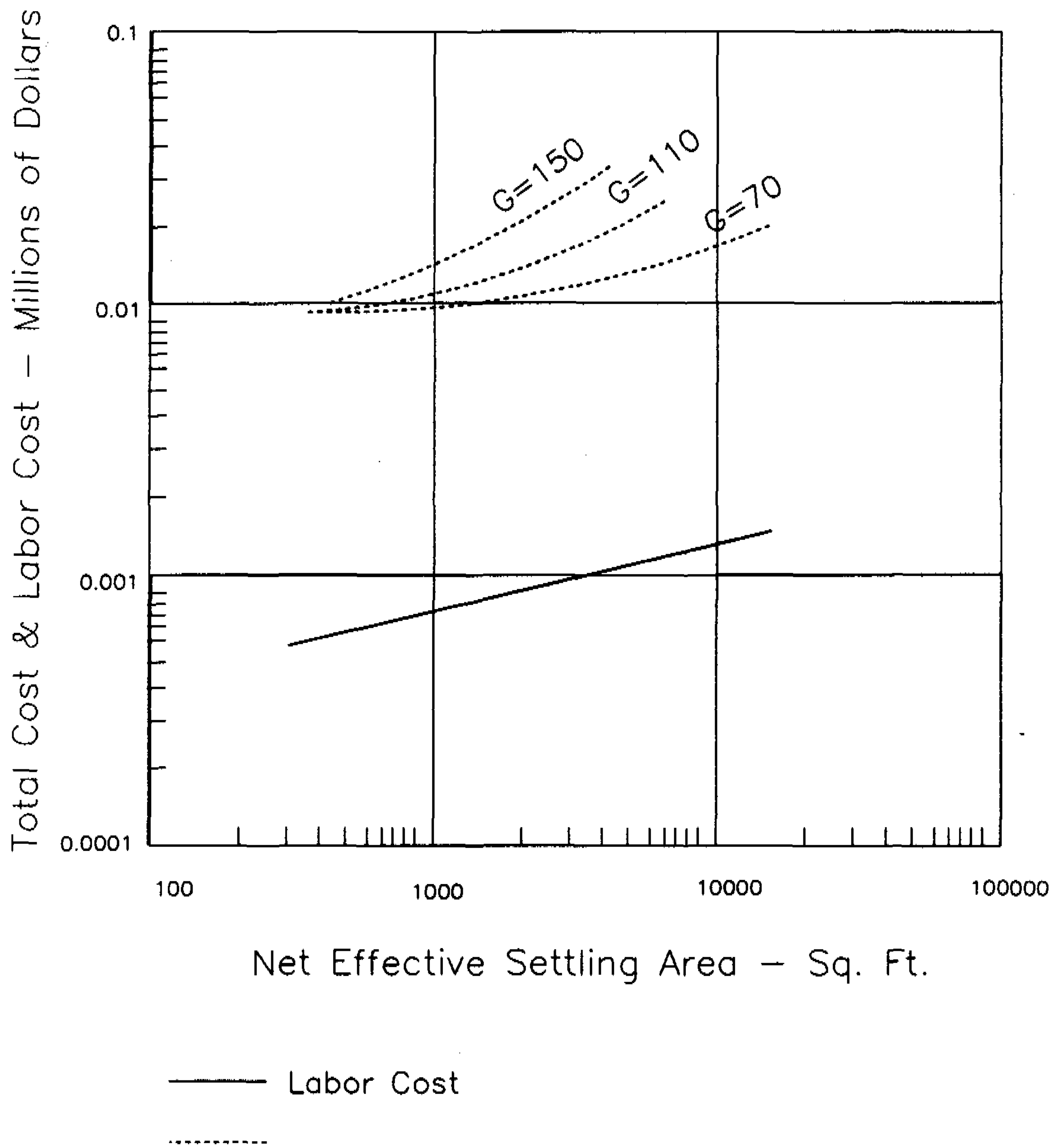
(File:Martin39)

Figure 7.2: Construction Cost for Upflow Solids Contact Clarifier



(File: Mart157)

Figure 7.3: Total Annual Labor for Upflow Solids Contact Clarifier



Note: G–values are flash mix values and are in units of sec–1

(File: Mart158)

bed grains are at the top (with uniform density bed material), thus allowing the use of virtually the entire bed depth for bed solids storage. During filtration, the bed material is kept in place usually by a metal grid.

The same water which is being filtered is used for backwashing, and this is claimed as an advantage for this type of unit.

7.6 Special Factors

Coagulation and flocculation performed in a granular media (such as the layer of gravel under the sand bed), and in the presence of chemical compounds previously precipitated improves filtration results. Thus, there may be an economy in the use of chemicals. If more than one operation is expected in a single unit, the complication for the operator increases. More attention is required to flow rates and system head loss for successful operation. Even with careful operator attention, the results may be mixed.

7.7 Recommendations

Upflow solids and contact clarifiers are generally selected on the basis of a lower cost and the operational advantages of combining several processes into a single basin. These are advantages and disadvantages in operational complexity. Operator training is required for consistent process performance.

Upflow clarifiers with flocculation may be more appropriate for small systems since there are fewer moving parts. The sensitivity of successful operational performance to control the level of the process especially if used with plate settlers make it an attractive option for package plants.

8. FLOCCULATION - CHEMICAL TREATMENT

8.1 Description

The objective of flocculation is to provide for an increase in the number of contacts among coagulating particles suspended in water by gentle and prolonged agitation. Flocculation follows chemical addition. During agitation, particles collide, producing larger and more easily removed flocs.

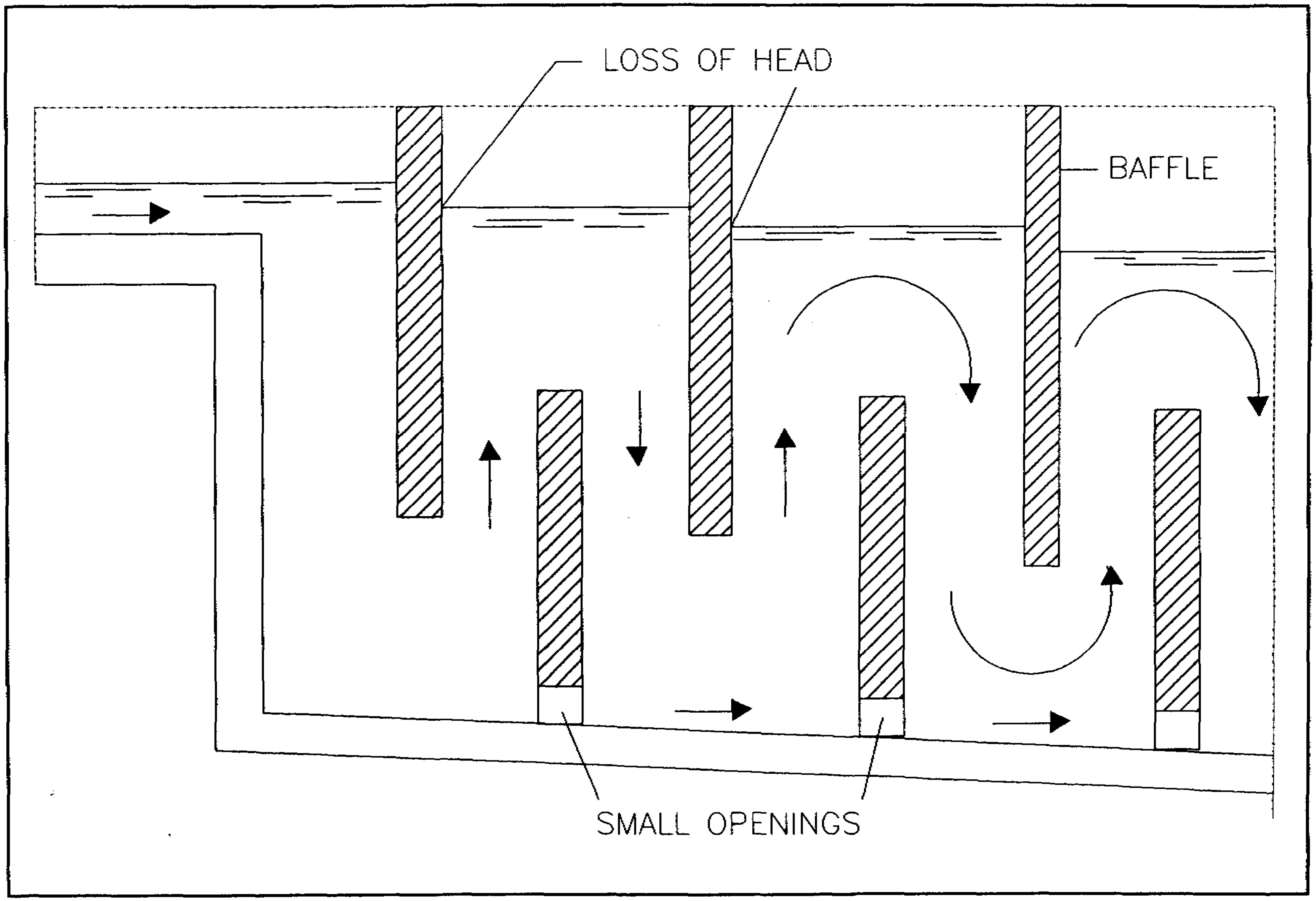
There are many different types of flocculators currently in use. The four discussed here are gravel, baffled, and horizontal and vertical mechanical flocculators. Gravel and baffled types may be used for smaller plants, and mechanical types for larger plants. There are no rigid size selection criteria.

In small and medium installations (up to 200 l/sec) one of the most common types of flocculation units in developing countries is the baffled (see Figure 8.1) (9, 25) or "Alabama," jet action flocculator (see Figure 8.2) (9, 25). The Alabama flocculator was introduced into Brazil during the Second World War by North American engineers. For small plants, the gravel flocculator (a bed of gravel used to create tortuous flow paths in which flocculation is enhanced).

The size and shape of a flocculation basin is generally determined by the type of flocculator selected and the type of sedimentation process employed. For example, if mechanical flocculators are paired with rectangular horizontal flow sedimentation basins, the width and depth of the flocculation basins should match the width and depth of the sedimentation basins. Similar dimensions enhance constructability and reduce overall project costs.

In all cases, the size of the flocculation basin is determined by the required reaction or detention time. Although

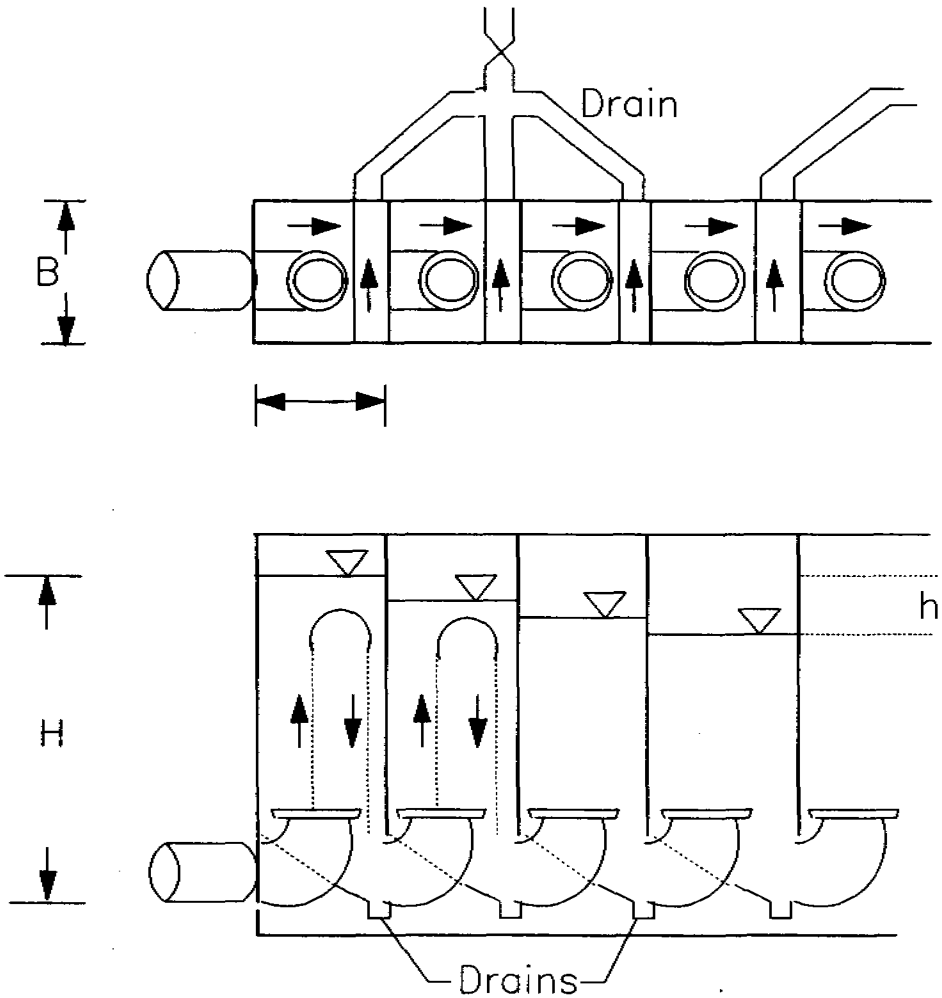
FIGURE 8.1: TYPICAL BAFFLED FLOCULATOR BASIN COMMONLY USED IN DEVELOPING COUNTRIES



(Source: Ref 9,25)

(File:Martin16)

FIGURE 8.2: GENERALIZED DIAGRAM OF FLOCCULATION BASIN COMMONLY USED IN BRAZIL



(Source Ref. 9)

(File:Martin17)

no theoretical relationship exists between basin area and water depth for optimal flocculation, the tank should be no deeper than 5 m. Basins with depths of greater than 5 m often display unstable flow patterns and poor flocculation.

Design Criteria (3, 8, 9, 25) - For baffled units with vertical flow: detention time, 15 to 25 min (warm climate); flow velocity, 0.10 to 0.20 m/sec; velocity gradient (G), 80-40 sec-1 (See Tables 8.1 and 8.2 for additional design data). For Alabama flocculators: detention time, 15-25 min; G, 50-40 sec-1; loss of head, 0.30-0.50 m; applied flow per chamber, 25-50 1/sec per m2; velocity in curves or bends of baffle flocculators, 0.40-0.60 m/sec; useful depth 2.5-3.5 m (See Table 8.3 for additional design guidance for flocculation chambers). For the gravel flocculator: depth of stone bed, 3 meters. For vertical shaft mechanical flocculators: minimum no. of successive chambers, 3; detention time: 20-30 min; G, 70-20 sec-1; max. tip speed 2 m/sec; approximately 5m x 5m to 10m x 10m basin surface area per unit. For horizontal shaft mechanical flocculators: G value, up to 50 sec-1; max. tip speed of 1 m/sec, paddle area should not exceed 20% of tank section area

The velocity gradient (G) for flocculators is determined from the equations:

$$G = (Q\ p\ g\ h_l/uV)^{1/2} = (p\ g\ h_l/ut)^{1/2}$$

for hydraulic flocculation, and

$$G = [P/(uV)]^{1/2}$$

for mechanical flocculation,

where

G = velocity gradients (s^{-1})

p = density of water (kg/m^3)

h = head loss (m)

u = dynamic viscosity (kg/m·s)

t = detention time, Q/V, (s)

Q = flow (m^3/s)

P = power, Qpgh (watts, kgm^2/s^3)

TALE 8.1 DESIGN DATA FOR FLOCCULATION

	V = 0,10 m/s		V = 0,12 m/s		V = 0,14 m/s		V = 0,16 m/s	
	15 min	20 min	15 min	20 min	15 min	20 min	15 min	20 min
Velocidade de escoamento, m/min	6	6	7,2	7,2	8,4	8,4	9,6	9,6
Energia cinética $\frac{v^2}{2g}$	0,0050	0,0050	0,0073	0,0073	0,0100	0,0100	0,0130	0,0130
Perda de carga por compartimento, m	0,0125	0,0125	0,0173	0,0173	0,0250	0,0250	0,0326	0,0326
Desenvolvimento (extensão) dos canais, m	90	120	108	144	126	168	144	192
Número de compartimentos para H = 4,50	20	26	24	32	28	37	32	42
Perda de carga total, m (considerados todos os compartimentos iguais)	0,25	0,32	0,42	0,55	0,70	0,93	1,04	1,37
Valor aproximado de G (todos os compartimentos iguais)	40	40	50	50	70	70	80	80

(Source: Ref 9)

TABLE 8.2 DESIGN DATA FOR FLOCCULATION

	30 l/s	40 l/s	50 l/s	75 l/s	100 l/s	150 l/s
V = 0,10 m/s	0,50 x 0,60	0,55 x 0,75	0,60 x 0,85	0,75 x 1,00	0,85 x 1,20	1,00 x 1,50
V = 0,12 m/s	0,45 x 0,60	0,55 x 0,65	0,55 x 0,75	0,70 x 0,90	0,80 x 1,05	0,90 x 1,40
V = 0,14 m/s	0,50 x 0,50	0,50 x 0,60	0,55 x 0,65	0,60 x 0,90	0,70 x 1,00	0,85 x 1,20

(Source: Ref 9)

TABLE 8.3

TYPICAL DESIGN PARAMETERS
FOR FLOCCULATION UNIT

Flow Rate Q 1/sec	Width B (m)	Length L (m)	Diameter D (mm)	Unit Chamber area (m^2)	Unit Chamber volume (m^3)
10	0.60	0.60	150	0.35	1.1
20	0.60	0.75	250	0.45	1.3
30	0.70	0.85	300	0.6	1.8
40	0.80	1.00	350	0.8	2.4
50	0.90	1.10	350	1.80	3.0
60	1.00	1.20	400	1.2	3.6
70	1.05	1.35	450	1.4	4.2
80	1.15	1.40	450	1.6	4.8
90	1.20	1.50	500	1.8	5.4
100	1.25	1.60	500	2.0	6.0

V = volume of unit (m^3)

g = gravitational constant (9.81 m/s^2)

In the design of flocculation systems, the total number of particle collisions, and thus the floc formation action, is indicated as a function of the product of the velocity gradient and the detention time, Gt. The range of velocity gradient (g) and Gt values given in Table 8.4 have been shown in practice to be the most effective for plants using mechanical flocculators. Nonetheless, in order to obtain appropriate values for particular designs and water characteristics to provide for the optimal formation of flocs, laboratory jar testing or pilot-plant studies should be conducted on the water to be treated.

TABLE 8A

RECOMMENDED G AND GT VALUES FOR FLOCCULATORS

Type	Velocity Gradient G (s^{-1})	GT
Turbidity or color removal (without solids recirculation)	20 to 100	20,000 to 150,000
Turbidity or color removal (with solids recirculation)	75 to 175	125,000 to 200,000
Softeners (solids contact reactors)	130 to 200	200,000 to 250,000

Source: 25

In baffled channel flocculation, mixing is accomplished by reversing the flow of water through channels formed by around-the-end or over-and-under baffles. Baffled channel flocculators are limited to relatively large treatment plants (greater than 10,000 m^3/day capacity) where the flowrates can maintain sufficient head losses in the channels for slow mixing without requiring that baffles be spaced too close together (which would make cleaning difficult). A distinct advantage of such

TABLE 8.4

FLOCCULATION CAPITAL COST
U.S. DOLLARS

	TOTAL BASIN VOLUME (ft^3)					
	1,800	10,000	25,000	100,000	500,000	1,000,000
Horizontal Paddle System						
G = 20	49,000	163,000	310,000	385,000	1,260,000	2,500,000
G = 50	50,000	163,000	313,000	425,000	1,400,000	2,600,000
G = 80	49,000	172,000	228,000	515,000	1,900,000	-

	TOTAL BASIN VOLUME (ft^3)				
	1,800	10,000	25,000		
	G=20 G=50 G=80	G=20 G=50 G=80	G=20	G=50	G=80
Vertical Turbine	43,400	144,000	209,000	209,000	218,400

flocculators is that they operate under plug-flow conditions that free them from short-circuiting problems.

Horizontal-flow flocculators with around-the-end baffles are sometimes preferred over vertical-flow flocculators with over-and-under baffles because they are easier to drain and clean; also, the head loss, which governs the degree of mixing, can be changed more easily by installing additional baffles or removing portions of existing ones. however, vertical-flow units have been used successfully in Brazil and in the Untied States and are appropriate for specific applications, such as, for example, where a scarcity of land prohibits the use of larger horizontal-flow flocculators.

The water depth in the channels of vertical flow units can be as high as 3 m, and therefore less surface area is required than with horizontal units. The major problem with such flocculators is the accumulation of settled material on the chamber floors and the difficulty in removing it. To mitigate this problem, the Brazilian designs have included small openings (weep holes) in the base of the lower baffles of a size equivalent to 5% of the flow area of each chamber. The purpose is to allow the major portion of the flow of water to follow the over-and-under path created by the baffles, whereas a smaller portion flows through the hole, creating some additional turbulence and preventing the accumulation of material (Arboleda, 1973). Weep holes also facilitate manual cleaning of the over-and-under flocculator.

For design purposes, the head loss in the bend of a baffle flocculator is approximated by the following formula:

$$h = k)v^2/2g) \qquad (6.3)$$

where

- h = head loss (m)
- v = the fluid velocity (m/s)
- g = the gravitational constant (9.81 m/s^2)
- k = empirical constant (varies from 2.5 to 4)

The value of k cannot be determined precisely in advance; therefore it is better to design for a low k value, because boards can always be added to the baffles if additional head loss is needed.

The number of baffles needed to achieve a desired velocity gradient for both horizontal and vertical flow units can be calculated from equations below,

$$n = \{[(2ut)/p(1.44 + f)]\ [(HLG)/Q]^2\}^{1/3}$$

for horizontal units, and

$$n = \{[(2ut)/p(1.44 + f)]\ [(HLG)/Q]^2\}^{1/3}$$

for vertical units,

where

- n = number of baffles in the basin
- H = depth of water in the basin (m)
- L = length of the basin (m)
- G = velocity gradients (s^{-1})
- Q = flow rate (m^3/s)
- t = time of flocculation (s)
- u = dynamic viscosity ($kg/m \cdot s$)
- p = density of water (kg/m^3)
- f = coefficient of friction of the baffles
- W = width of the basin (m)

The water velocity in both horizontal-flow and vertical-flow units generally varies from 0.3 to 0.1 m/sec. Detention time varies from 15 to 30 min. In general, velocity gradients for both types of baffled channel flocculators should vary between 100 to 10 s^{-1}. In addition to the foregoing design criteria, the practical criteria given in the Table should be followed:

Guidelines for the Design and Construction of
Baffled Channel Flocculators (25)

A. Around-The-End (Horizontal Flow)

1. Distance between baffles should not be less than 45 cm to permit cleaning.

2. Clear distance between the end of each baffle and the wall is about 1 1/2 times the distance between the baffles; should not be less than 60 cm.

3. Depth of water should not be less than 1.0 m.

4. Decay-resistant timber should be used for baffles; wood construction is preferred over metal parts.

5. Avoid using asbestos-cement baffles because they corrode at the pH of alum coagulation.

B. Over-and-Under (Vertical Flow)

1. Distance between baffles should not be less than 45 cm.

2. Depth should be two to three time the distance between baffles.

3. Clear space between the upper edge of a baffle and the water surface, or the lower edge of a baffle and the basin bottom, should be about 1 1/2 times the distance between baffles.

4. Material for baffles is the same as in around-the-end units.

5. Weep holes should be provided for drainage.

The Alabama-type flocculator is illustrated in Figure 8.2. The jet action is provided in each chamber via a cast iron pipe with its outlet turned upward. For effective flocculation, the outlet should be placed at a depth of about 2.5 m below the water level. Common design criteria are listed below:

Rated capacity per unit chamber	25 to 50 l/s per m^2
Velocity at turns	0.40 to 0.60 m/s
Length of unit chamber (L)	0.75 to 1.50 m
Width (W)	0.50 to 1.25 m
Depth (H)	2.50 to 3.0 m
Detention time (t)	15 to 25 min

The head loss with this type of flocculator is estimated at two velocity heads per chamber, generally about 0.35 to 0.50 m of head loss for the entire unit. Velocity gradients range from 40 to 50 s^{-1}. Arrangements should be made for draining each chamber, since material tends to collect at the bottom and must be removed.

The gravel-bed flocculator provides a simple and inexpensive design for flocculation in small water treatment plants (less than 5000 m^3/day capacity). It has been tested experimentally and employed successfully in several upflow-downflow plants in India and in package plants in Parana, Brazil. The packed bed of gravel provides ideal conditions for the formation of compact settleable flocs because of continuous recontacts provided by the sinuous flow of water through the interstices formed by the grave. The velocity gradients that are introduced into the bed are a function of (1) the size of the gravel, (2) rate of flow, (3) cross-sectional area of the bed, and (4) the head loss across the bed. The direction of flow can be either upward or downward, and is usually determined from the design and hydraulic requirements of other process units in the plant.

8.2 Advantages and Disadvantages

The major shortcomings of hydraulic flocculators are: 1. little flexibility to respond to changes in raw water quality, 2. hydraulic parameters are a function of flow and cannot be changed within the process, 3. head loss can be significant, 4. cleaning may be difficult unless design incorporates cleaning provisions.

There is a paradox for the application of these various types. While simpler hydraulic types would be desirable for small plant applications where operator skill is likely to be less, high potential head losses argue for appplication at large plants.

Baffled flocculators: a lack of flexibility for mixing intensity; a high head loss if over-and-under baffles are used; some plant flowrates may vary in the range of 1:4 within a single day so achieving good mixing in the entire flow range may be difficult.

Vertical shaft mechanical flocculators: many units are required in a large plant; high capital cost for variable speed reducers and support slabs.

Horizontal shaft mechanical flocculators: precise installation and maintenance is necessary; difficult to increase energy input; problems with leakage and shaft alignment.

The main problem with gravel bed flocculators is fouling, either by intercepted floc or by biological growth on the media, or both.

8.3 Costs

See Table 8.5 (5) for costs. These costs are for mechanical units only, at different G values. Costs for the full range, and for operation and maintenance are given on Figures 8.3 and 8.4.

Many examples of baffle flocculations exist all over the world. No cost data are available for plants visited.

8.4 Availability

All of these systems are used throughout developing countries as well as the U.S. Most materials (even the mechanical

TABLE 8.5

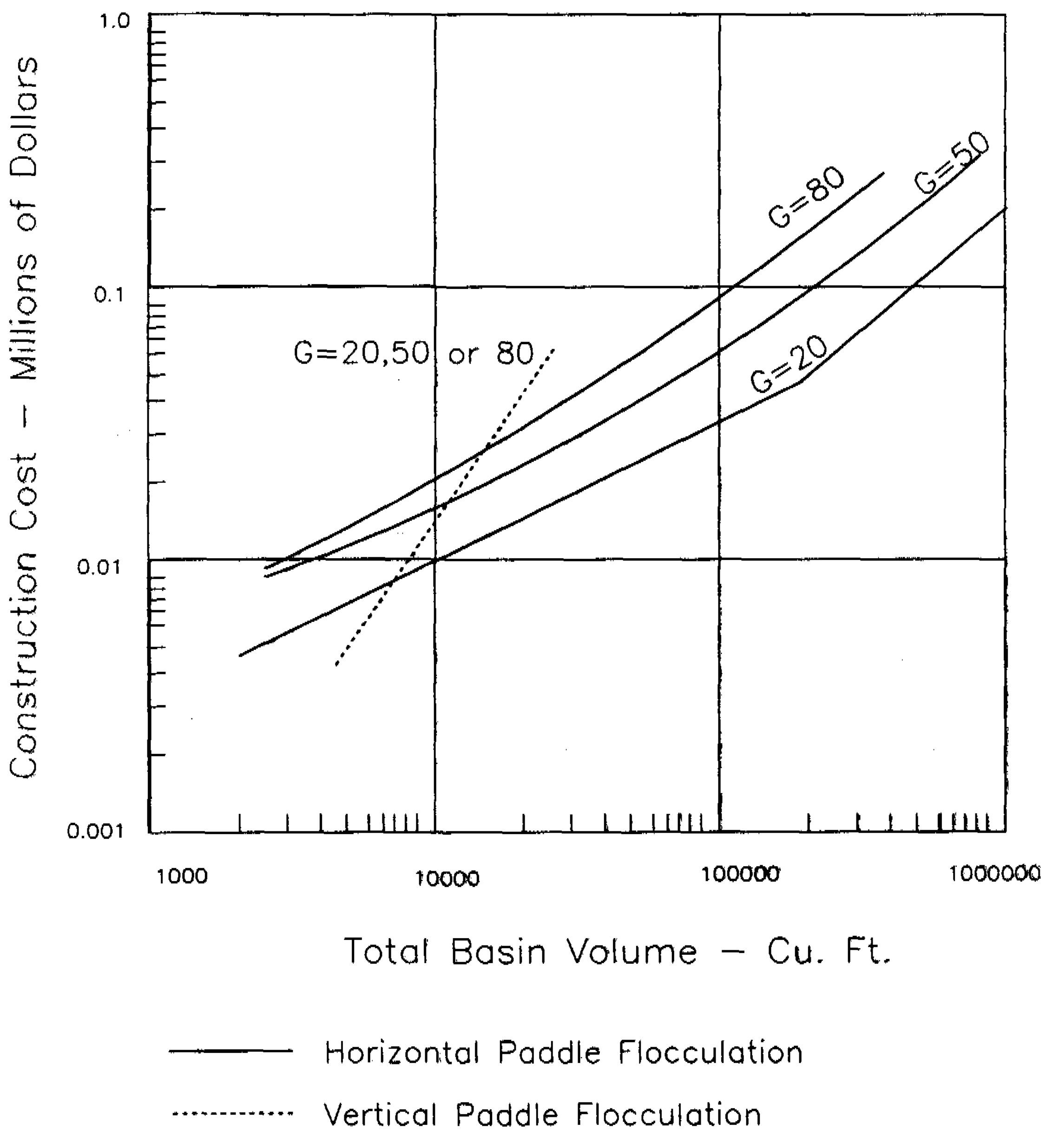
OPERATION AND MAINTENANCE SUMMARY
FLOCCULATION - HORIZONTAL PADDLE SYSTEM

Total Basin Volume (ft^3)	Energy (kw-hr/yr) G=20	G=50	G=80	Maintenace Material ($/yr)	Labor (hr/yr)	Total Cost ($/yr)* G=20	G=50	G=80
1,800	330	2,070	6,100	615				
99	2,100	2,200	2,400					
10,000	1,960	11,870	33,660	1,600				
199	4,650	5,100	6,100					
25,000	4,900	29,630	84,080	1,600	199	4,800	5,900	8,300
100,000	19,600	118,720	336,550	6,000	397	12,900	17,300	27,000
500,000	98,020	593,590	1,682,750	21,600	496	33,500	56,000	105,000
1,000,000	198,230	1,188,300	--	43,000	990	67,000	112,000	--

* Calculated using $0.03/kw-hr and $10.00/hr of labor

NOTE: G values are flash mix values and are in units of SEC^{-1}

Figure 8.3: Construction Cost for Flocculation Horizontal Paddle System and Vertical Turbine Flocculation



Note: G-values are flash mix values and are in units of sec-1

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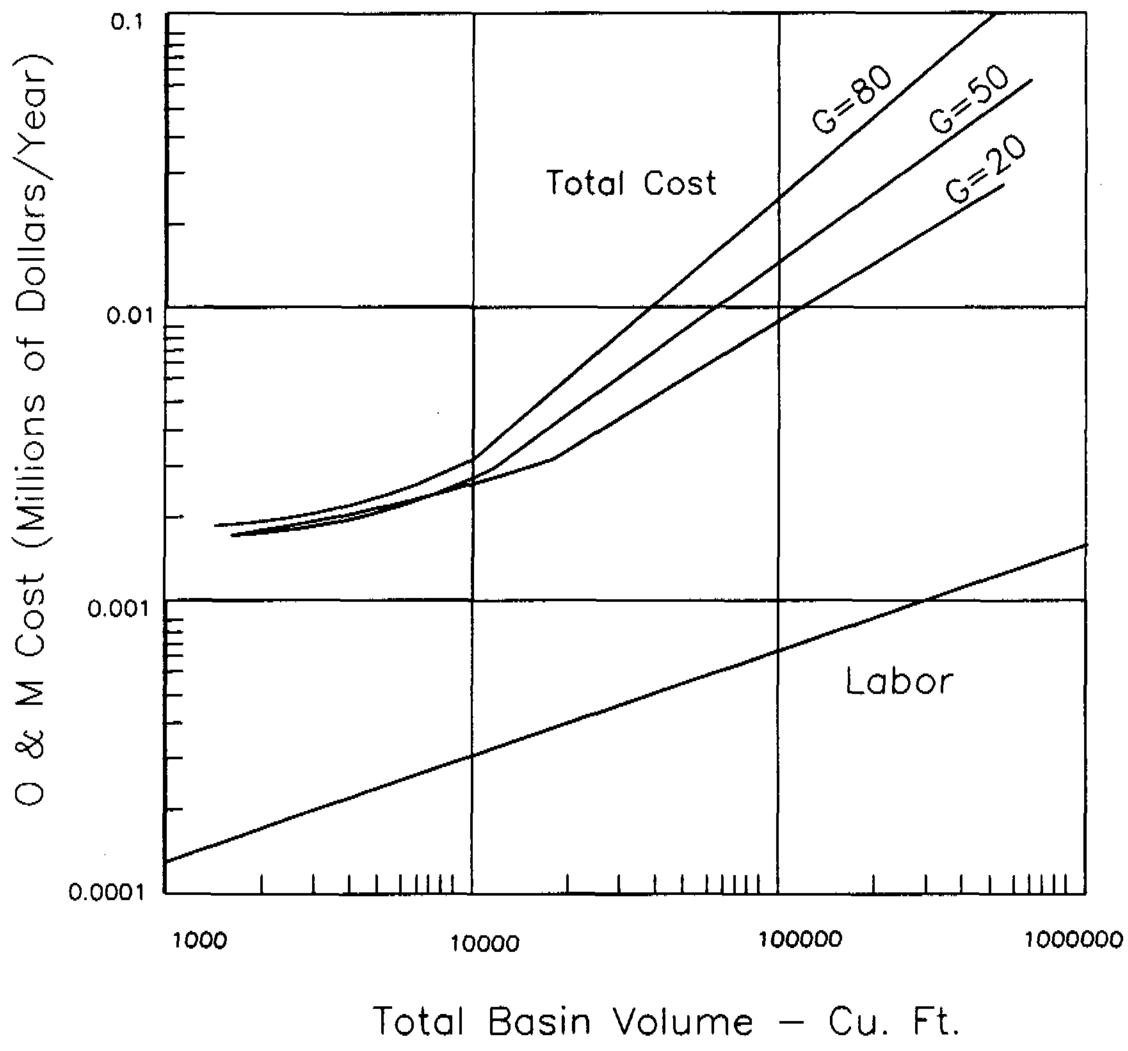
Figure 8.4: Operation & Maintenance Requirements
for Horizontal Paddle Flocculation



Note: G Values are flash mix values and are in units of sec−1

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flocculator parts) can be purchased in many countries. Purchases can be made in Brazil, for example.

8.5 Operation and Maintenance

Baffled: This type provides plug-flow mixing conditions and is an effective flocculation system. Since tapered mixing is effective in forming large settleable flocs, the baffles should be properly arranged to reduce the mixing intensity and floc shearing force. It performs well if the plant flow rate is reasonably constant. Removal of accumulations of material in chambers is required. Little maintenance is required because of few moving parts.

Gravel: This flocculator is composed of a concrete chamber full of small stones or pebbles where water is introduced by a distribution system at the bottom so that it flows upward through the bed of stones. With this process, little maintenance is required. Frequent cleaning however, is required.

Vertical mechanical: This system is suitable for high energy input, direct filtration and conventional treatment. All mechanical systems require more maintenance than systems with few or no moving parts.

Horizontal mechanical: This method produces a large size floc, has a simple mixing unit and is suitable for conventional treatment. More data and experience is available generally than for vertical mixing types.

8.6 Control

Mechanical systems require more intensive maintenance programs than non mechanical systems. Thus, the gravel and baffled types are more appropriate for remote systems, and those where operator training is not expected to be available.

8.7 Special Factors

Baffled channel flocculators should be designed so that the cross section will provide the proper flow velocity. Adjustable baffles should be included to provide the desired degree of turbulence. Vertical-shaft flocculators should be situated to cover a square or circular mixing zone for maximum efficiency.

If a decision is to be made about one of the two mechanical flocculator systems then the decision will be mainly dependent on the type of filtration system used. Horizontal shaft flocculators are generally used if conventional rapid sand filtration is employed. Conventional rapid sand filtration requires a high degree of solids removal by the sedimentation basin before filtration. The horizontal shaft flocculators generally are suited to produce large and easily settleable floc with alum flocculation. However, they usually require more maintenance and expense mainly because bearings and packings are typically submerged. High energy, vertical shaft flocculators are the unit of choice for high-rate filtration systems. Since high rate filters allow floc penetration into the filter bed, the desired type of floc for these filters is small in size but physically strong to resist high shear forces in the filter bed.

8.8 Recommendations

All systems can be used with efficiency in developing countries if the system selection is correctly matched to the size of the plant and expected operator skill. For instance, for small installations gravel could be used. For small and medium size installations baffled or Alabama flocculators could be used (provided available head allows). Alabama flocculators have been proven in Brazil; gravel bed units have been used in India for small plants. Finally, for larger installations the mechanical types are best. The vertical shaft flocculator is used with some frequency in South America.

9. GRAVITY SEWERS

9.1 Description

Gravity sewers are used for the transport of sanitary and industrial wastewater, stormwater and any combination of wastewaters. Slope design is important and results in flow due to gravity. Access to a gravity sewer is by manholes spaced about every 300 to 500 feet or at changes in slope or direction.

Design Criteria (2): Size: dependent on flow, minimum 6" inside diameter for all laterals in collection systems. Slope: dependent on size and flow. Velocity: minimum of 2.0 ft/sec at full depth. Materials: must meet service application requirements. Additional Requirements: adequate ground cover, minimum scouring (self-cleaning) velocity; infiltration should not exceed 200 gal/d/inch diameter/mile. Small diameter gravity sewers for transport of septic tank effluent, have a minimum diameter of 4 in and are designed for 1/2-peak flow (corresponding to a gradient of 0.67 percent for a 4 in sewer.)

See Table 9.1 (2) for design criteria for the sizing of collector and interceptor sewers.

9.2 Limitations

High capital cost in rural areas, in areas requiring removal of ledge rock and where depths greater than 15 feet are required. Possible explosive hazards can occur due to production of gas or improper hydraulic design of a sewer. Blockage is also a possibility because of grease, sedimentation, tree root development and, in the case of combined sewers, debris. Excessive infiltration and inflow are the most common problems for both old and new systems.

TABLE 9.1

SIZING OF COLLECTOR AND INTERCEPTOR SEWERS

Pipe Diameter	Minimum Slope for Pipe Velocity of 2 ft/sec	Minimum Slope for Pipe Velocity of 8 ft/sec	Design Wastewater flow (Mgal/d) with Pipe Flowing Full, Velocity, ft/sec 2	3	4	8
6	0.0060	0.075	0.26	0.36	0.47	0.91
8	0.0038	0.45	0.48	0.69	0.91	1.76
10	0.0030	0.035	0.72	1.04	1.37	2.54
12	0.0022	0.026	1.04	1.46	1.94	3.51
15	0.0015	0.020	1.69	2.47	3.25	5.84
18	0.0012	0.016	2.41	3.45	4.42	9.43
21	0.0010	0.013	3.38	4.94	6.37	12.35
24	0.00078	0.011	4.10	6.24	8.13	15.28
27	0.00065	0.0095	5.20	7.80	10.08	18.85
30	0.00058	0.0080	6.50	9.75	13.00	24.05
36	0.00045	0.0060	9.75	14.63	18.20	37.05
42	0.00038	0.0050	13.00	19.50	25.36	48.10
48	0.00032	0.0045	16.25	24.70	31.85	59.80
54	0.00026	0.0039	20.80	31.85	39.65	84.50

9.3 Costs

See Figures 9.1 and 9.2 (2,11).

9.4 Availability

Gravity sewers are the oldest and most common wastewater transport system existing. Use of water for waste carriage results in contamination of the water and often in sources used for downstream supply, or groundwater.

Materials for sewer construction - cement, asbestos cement, reinforced concrete are widely available. Materials such as PVC may have improved erodibility characteristics and should be considered also (see Section on Steep Slope Sewers). Also, possible health considerations for asbestos cement pipe should be taken into account.

9.5 Operation and Maintenance

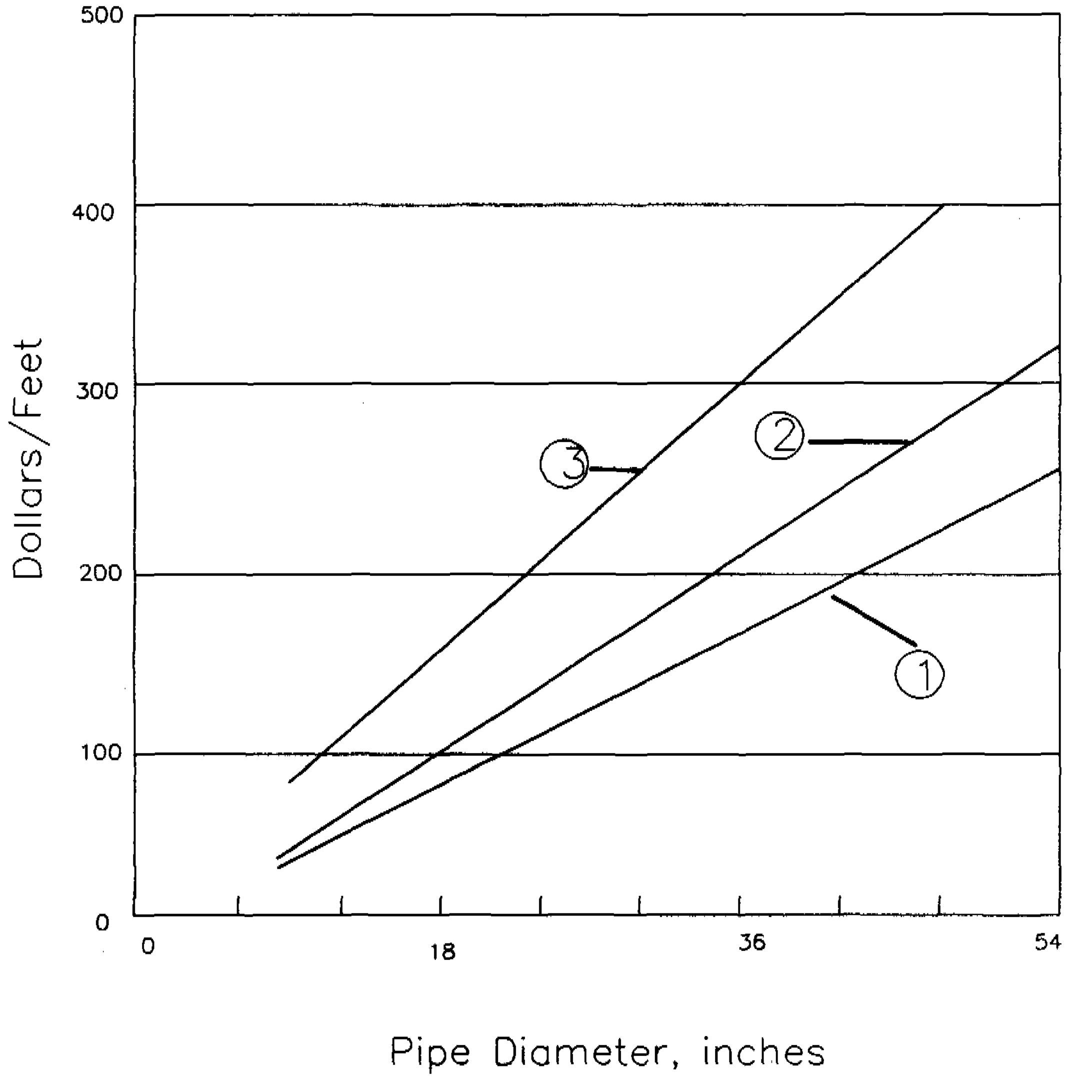
Pipe material with special considerations are as follows:

Asbestos Cement: advantages include light weight, ease of handling, long laying lengths and tight joints; disadvantages include corrosion where acids and hydrogen sulfide are present along with its known carcinogenic characteristics; diameters available from 4 to 42 in.

Clay Pipe: advantages include a resistance to corrosion from acids and alkalies and also a resistance to erosion and scour; disadvantages are that clay pipe has a limited range of sizes and strengths, it is also brittle with short pipe lengths and many joints; diameters available from 4 to 42 in.

Concrete (both reinforced and non-reinforced): advantages are its strength, availability of sizes and wide spread use;

Figure 9.1: Capital Cost of Gravity Sewer.



1. Depth < 8 ft
2. Depth = 8–15 ft
3. Depth > 15 ft

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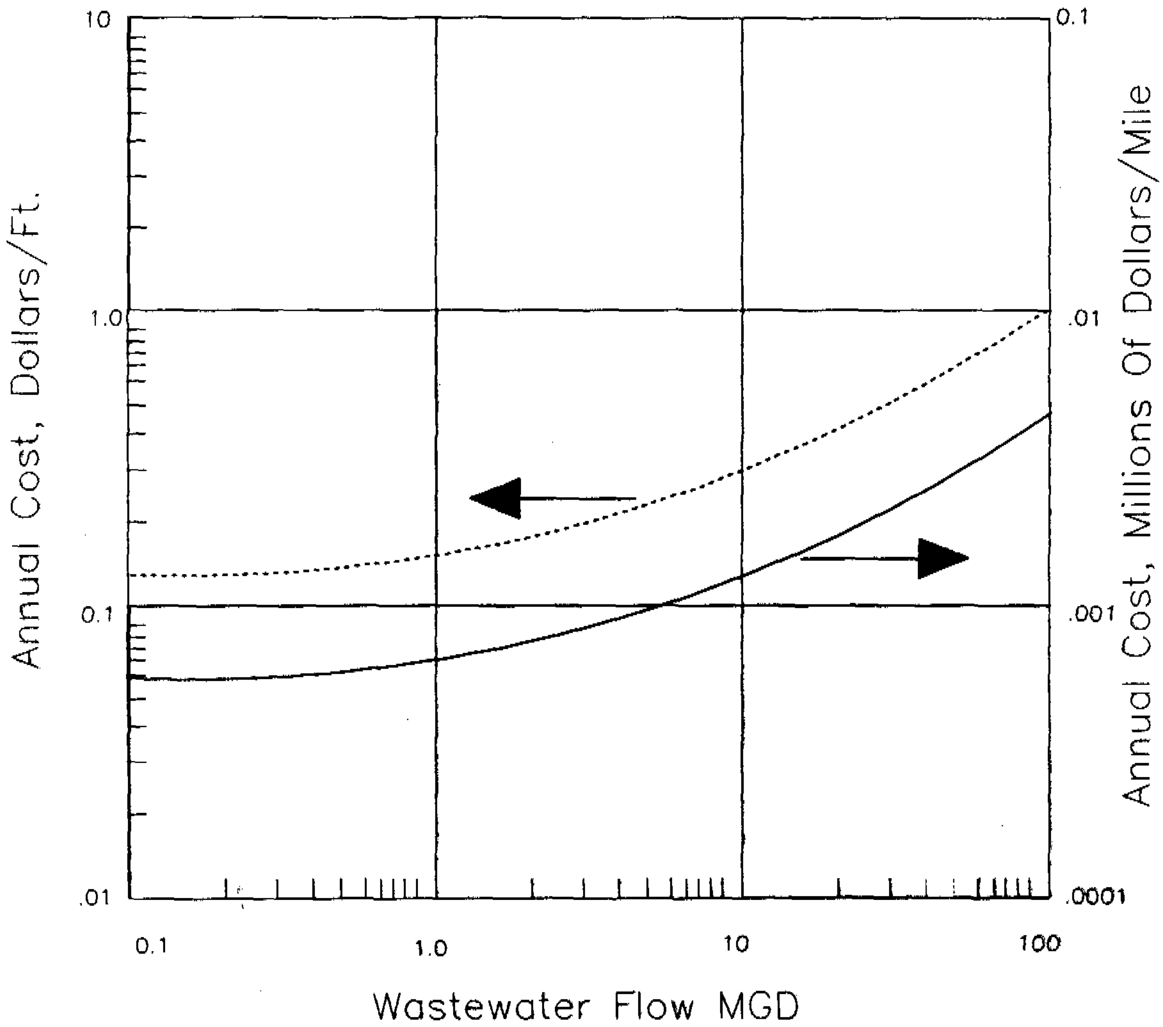
Figure 9.2: Operation & Maintenance Cost for Gravity Sewers.



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disadvantages include a corrosion tendency when acid and hydrogen sulfide are present. Short pipe lengths and many joints are required; diameters available from 12 to 144 in for reinforced and 4 to 24 in for non-reinforced concrete pipe.

Cast Iron: advantages include long laying lengths and tight joints along with an ability to withstand high external loads and a corrosive resistant nature in neutral solids; disadvantages include corrosion by acid, septic wastewater or corrosive soils; diameters available from 2 to 48 in.

Plastic Pipe: advantages include light weight, tight joints, long laying lengths and in some cases, corrosion and erosion resistance; disadvantages include thin walls, susceptibility to sunlight and low temperature, which affect shape and strength; diameters are available to 12 in for solid wall and from 8 to 15 inch for plastic or truss pipe.

Highly reliable, with a long life expectancy. System is not dependent on moving parts.

9.6 Special Factors

Common modifications include addition of corrosion protection coatings (coal based tar, PVC based tar) chemical grouting and slip-in liners (for pipes with diameters of less than 12 in) for rehabilitation of in-place sewers, inverted syphons, lift stations for hilly or excessively flat terrain and diversion regulators for combined sewers.

In rural communities where topography is favorable, small diameter gravity sewers which transport septic effluent to central treatment works have been employed in Australia and, to a limited extent in the U.S. Generally, these sewers have a minimum pipe diameter of 4 in. All installations to date have employed PVC pipe, owing to its light weight, long lengths and ease of laying. Curvilinear alignments in the vertical and horizontal planes are allowable, and manholes and meter boxes (depending on line depth) may be kept to a minimum (400 to 600 feet spacing).

There is a low environmental impact on air and water. However, there is considerable impact on land during installation. The installation of sewers in roadways adjacent to vacant properties leads to an increase in the rate of development of the land. Small diameter gravity sewers in rural areas would result in a reduction in the magnitude of the land and secondary development impacts for conventional gravity sewers. They may also reduce the land requirements for subsequent treatment processes where organic loading is the principal design parameter.

9.7 Recommendations

Gravity sewers are commonly used for the transportation of wastewater wherever gravity flow is cost effective.

10. PRESSURE SEWERS

10.1 Description

Pressure sewers are operated with pumping instead of gravity. They may result in lower construction costs relative to gravity sewer systems in less populated areas. Pressure sewers may be placed independent of slope. Pressure sewers have been largely offered in order to reduce the high costs of sewer systems which have been designed in accordance with generally accepted design parameters for slope and velocity (generally to maintain a minimum of 2 ft/sec; 0.61 m/sec). Generally there is a trade-off in more system operating complexity because of typically smaller diameters in pressure systems, and increased operating costs due to power requirements. The pressure sewer system requires a number of pressurizing inlet points and a single outlet to a treatment facility or to a gravity sewer, depending on the application.

The two major types of pressure sewer systems are the grinder pump (GP) system (See Figure 10.1) and the septic tank effluent pump (STEP) system (See Figure 10.2). The major difference between the two systems are in the on-site equipment and layout. Neither pressure sewer system alternative requires any modification of household plumbing.

10.2 Advantages and Disadvantages

High operation and maintenance costs are likely because of the use of mechanical equipment at each point of entry to the system. In GP systems, the wastewater conveyed to the treatment facility may be more concentrated than normal wastewater. In STEP systems, a weaker, more septic waste is generated. Therefore, both systems require special care in system design and in treatment facility design. It is more difficult to monitor and maintain many small pumps than a few larger ones in gravity systems.

Figure 10.1

PRESSURE SEWER CONNECTION

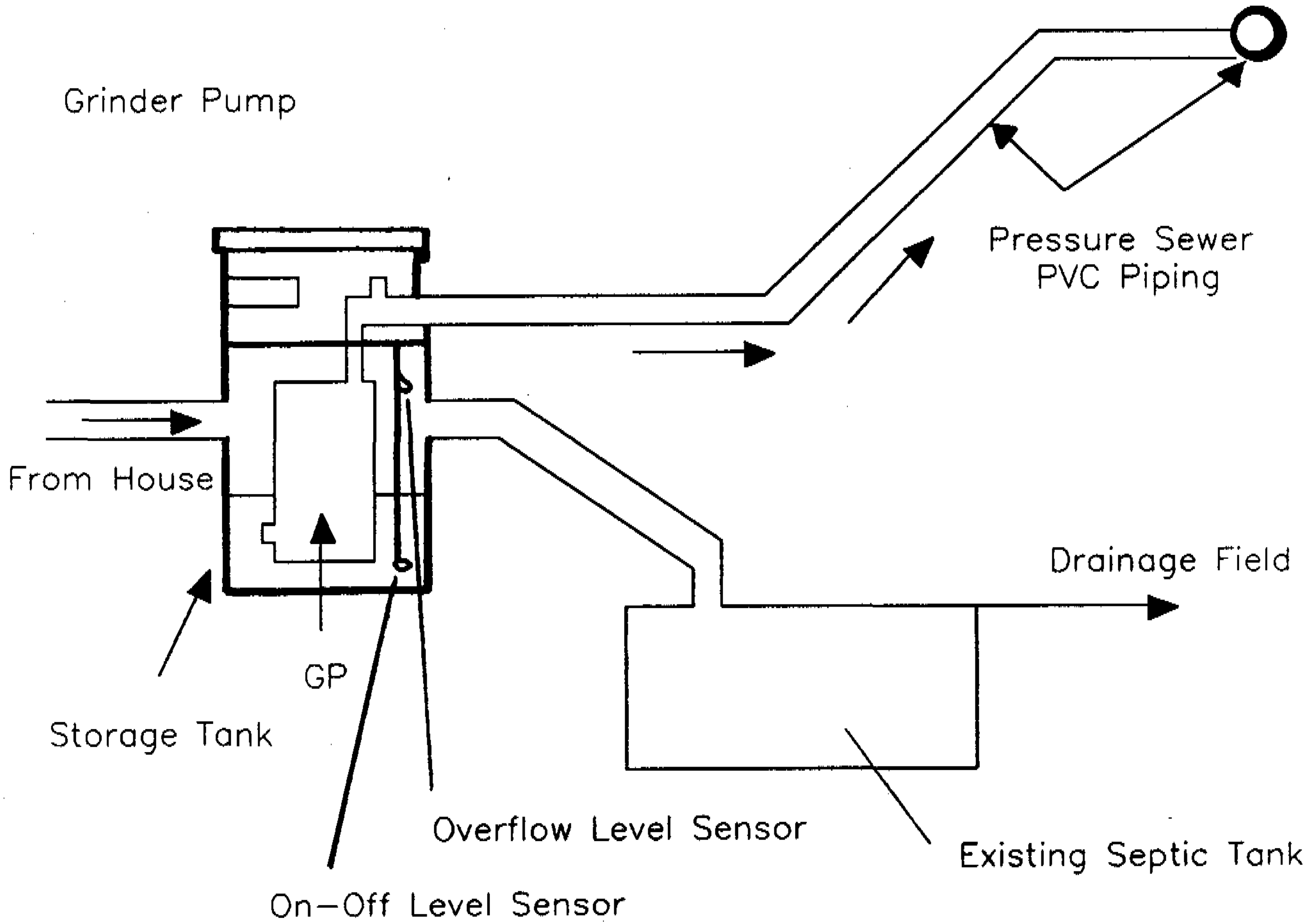

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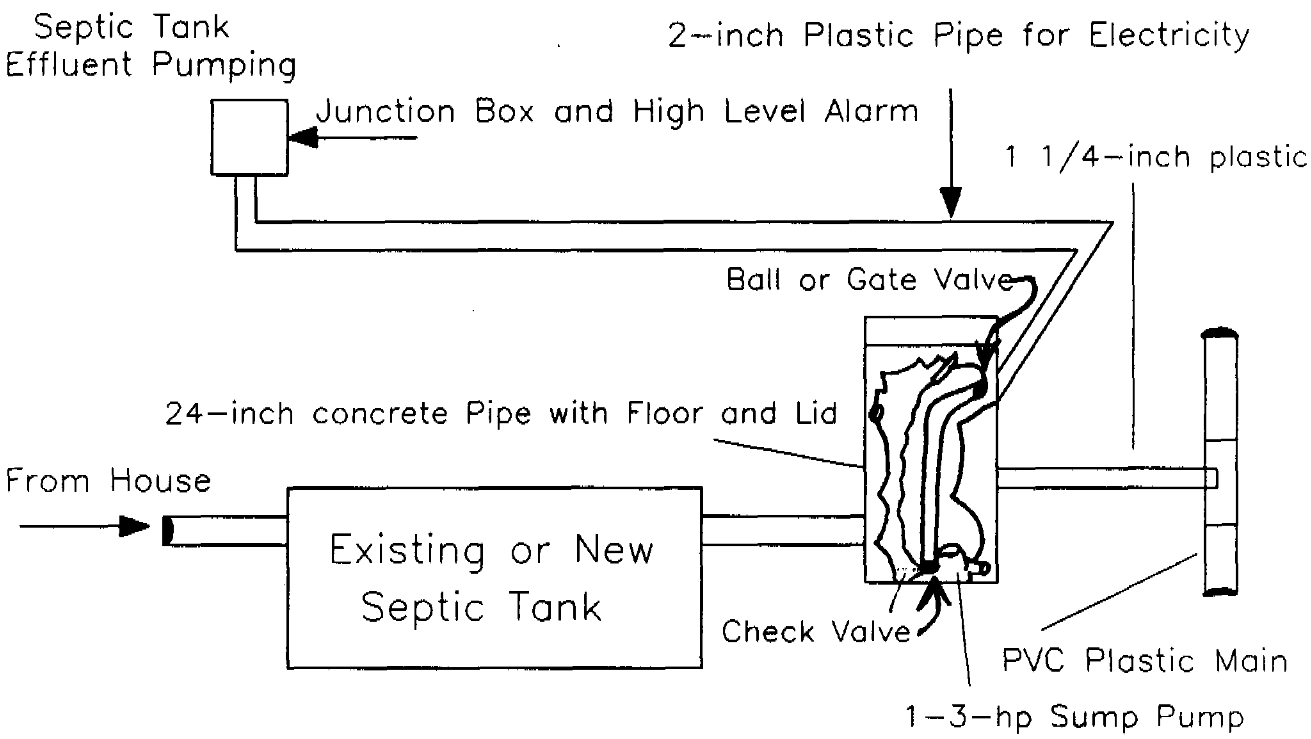
Figure: 10.2

PRESSURE SEWER SYSTEM



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PRESSURE SEWERS

ADVANTAGES	DISADVANTAGES
o Low construction cost o Adaptable to severe terrain conditions; no slope required o No infiltration/exfiltration o STEP systems reduce grit, grease, and solids present in wastewater flow o Shallow sewer depths o Cleanouts used instead of manholes	o Multiple pumping units o required o Relatively high operation and maintenance requirements o STEP systems require periodic cleaning of septic tanks and disposal of septage o Individual services susceptile to power/pump failures (unless overflow storage is provided o Potential odor/corrosion problems

Design criteria are as follows: Dendriform systems (irregular piping network) are generally used instead of rectangular grids. Pump requirements vary with the type of pump employed and its location in the system. Flushing provisions are necessary. Pipe design is based on Hazen-Williams friction coefficient of 130 to 140. For GP systems a minimum velocity of about 3 ft/sec at least one time per day is used to prevent deposition of solids. Meter boxes generally suffice in place of manholes.

Service connection lines between the pump and the pressure main are generally made of 1 to 2 in PVC pipe with PVC drain, waste and vent fittings. Pressure mains are generally 2 to 12 in diameter PVC pipe, depending on hydraulic requirements. Pipes must only be buried deep enough to avoid freezing. Head loss due to pipe friction generally is in the range 1 to 4 ft water/100 ft of pipe.

10.3 Costs

See Table 10.1 (2). Local economics, especially cement and other material costs, distance to manufacturing and distribution centers, climate, geology and slope, soil type, and many other

TABLE 10.1

SEWER COSTS

Components**	Construction Cost	Operation & Maintenance Cost
1. Mainline Piping (PVC)		
a. 1-3 in diameter	$5.61/lin ft	$187-374/year/mile
b. 4 in	$6.55/lin ft	
c. 6 in	$8.70/lin ft	
TOTAL		$187-374/year/mile
2. On-lot Septic Tank Effluent Pumping (STEP)		
a. Pump, controls, etc.	$1700-2800	$75/year
b. Service line (100 ft @ $2/ft)	374	
c. Corporation cocks, valves, etc.	94	
d. Septic tank (optional)	470	$19
e. Connection fee	94-187	
TOTAL	$ 2240-4,000	$94/year
3. On-lot Grinder Pump (GP)		
a. GP unit, controls, etc.	2400-3740	$140/year
b. Service line (100 ft @ $2/ft)	374	
c. Cocks, valves, etc.	94	
d. Connection fee	94-187	
TOTAL	$ 3000-4400	$140/year

factors make efforts to give realistic costs extremely difficult. The costs given on Table 10.1 should be used for gross estimating purposes only.

10.4 Availability

More than 100 pressure sewer systems have been operated in the United States to date.

10.5 Operation and Maintenance

In both designs household wastes are collected in the sanitary drain and conveyed by gravity to the pressurization facility. The on-lot piping arrangement includes at least one check valve and one gate valve to permit isolation of each pressurization system from the main sewer. GP's can be installed in the basement of a home to provide easier access for maintenance and greater protection from vandalism.

In STEP systems, wastewater receives intermediate treatment in a septic tank. The septic tank effluent then flows to a holding tank which houses the pressurization device, control sensors and valves required for a STEP system.

Normally, small centrifugal pumps are employed for the STEP systems. These pumps are submersible and range in size from 1/4 to 2 hp. Pump total head requirements generally range from 25 to 90 ft. Impellers can be made of plastic to reduce corrosion problems. Also included within the holding tank are level controls, valves and piping. The effluent holding tank can be made of properly cured precast or cast-in-place reinforced concrete, or they may be made of molded fiberglass or reinforced polyester resin. Tank size is based on accessibility for repairs and maintenance.

Pump control switches are either a pressure sensing type, or the mercury float type switch.

10.6 Control

Severe corrosion can cause mechanical and/or electrical problems. Accumulations of grease and fiber can cause reduction in pipe cross sectional area of GP systems during partial flow conditions. Estimated life of current pump designs exceeds ten years. Centralized maintenance is generally required for optimum service.

10.7 Special Factors

On GP systems an emergency (i.e., power failure, etc.) overflow tank may be provided. Measures such as standpipes and pressure control valves are sometimes used to maintain a positive pressure on the system. Air release valves are also provided to release gas pockets in the system. Polyethylene pipe, pneumatic ejectors and mainline check valves have been used in some designs.

10.8 Recommendations

Pressure sewers are most applicable where population density is low, where it is severely rocky, and where high ground water or unstable soils prevail and also where terrain slopes change frequently.

11. FACULTATIVE LAGOONS

11.1 Description

Facultative Lagoons are low-cost, highly efficient alternative of wastewater treatment in tropical and subtropical climates. These lagoons are intermediate depth (3 to 8 ft) ponds in which the wastewater is stratified into three zones (See Figure 11.1) (2). These zones consist of an anaerobic bottom layer, an aerobic surface layer and an intermediate zone. Stratification is a result of solids settling and temperature-water density variations. Oxygen in the surface stabilization zone is provided by reaeration and photosynthesis. This is in contrast to aerated lagoons in which mechanical aeration is used to create aerobic surface conditions. In general, the aerobic surface layer serves to reduce odors while providing treatment of soluble organic by-products of the anaerobic processes operating at the bottom.

Sludge at the bottom of the facultative lagoons will undergo anaerobic digestion producing carbon dioxide and methane. The photosynthetic activity in the aerobic lagoon surface produces oxygen diurnally, increasing the dissolved oxygen during daylight hours, while surface oxygen is depleted at night.

Design Criteria (also see Table 11.1): At least three cells in series. Parallel trains of cells may be used for larger systems. Detention time: 20 to 180 days. Depth, ft: 3 to 8 (0.9 to 2.4 m), although a portion of the anaerobic zone of the first cell may be up to 12 ft (3.66 m) deep to accommodate large initial solids deposition. The pH: 6.5 to 9.0. Water temperature range: 35 to 90 degrees F for municipal applications. Optimum water temperature: 68 degrees F. Organic loading: 10 to 100 lb BOD_5/acre/day, perhaps up to 300 lb/acre/day (approx. kg/ha/day) in tropical climates.

Figure 11.1: Typical Facultative (Aerobic–Anaerobic) Lagoon

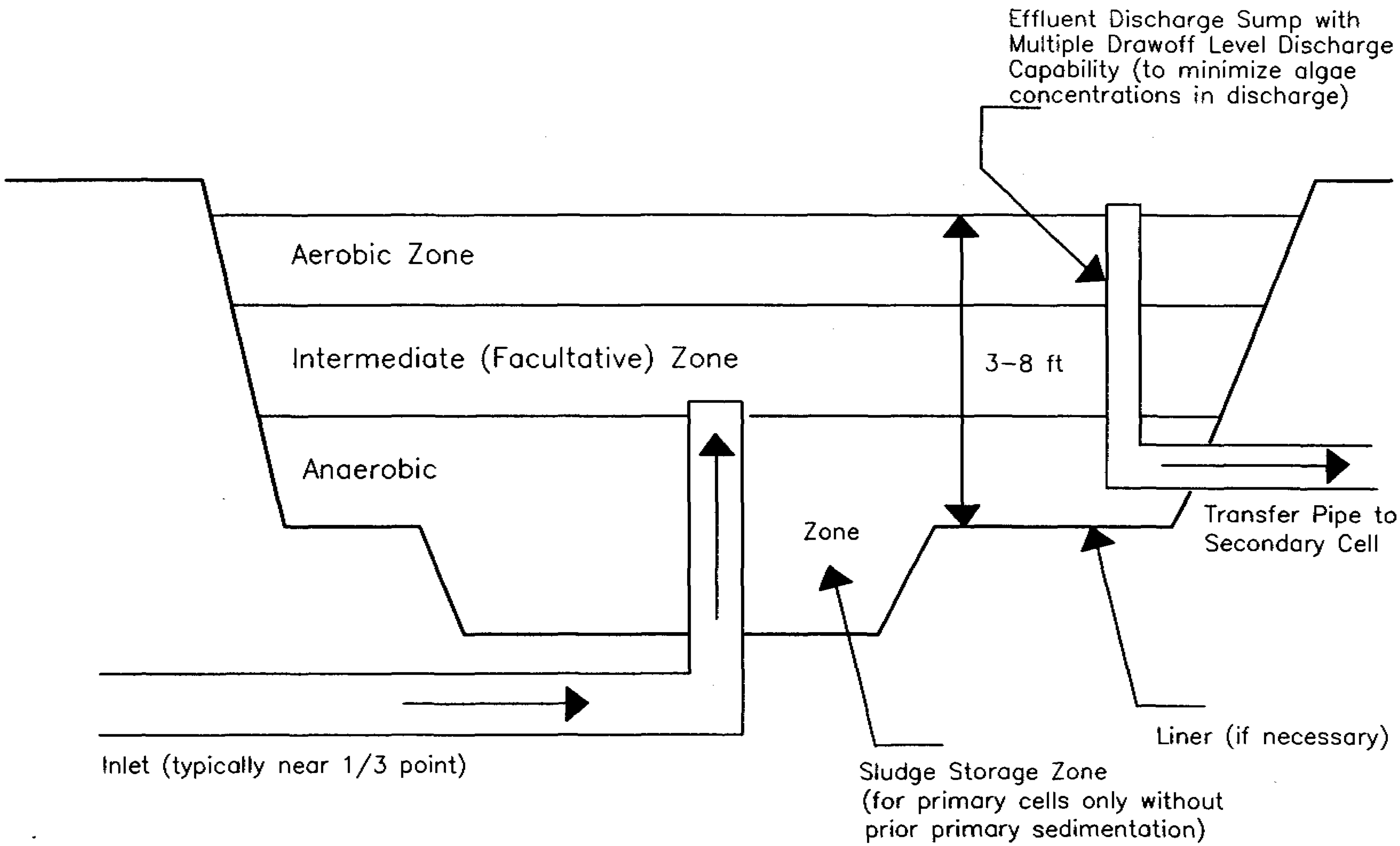

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TABLE 11.1

TYPICAL DESIGN PARAMETERS FOR ANAEROBIC AND FACULTATIVE STABILIZATION PONDS

Parameter	Aerobic-anaerobic (faculative) pond	Aerobic-anerobic (faculative) pond	Anaerobic pond	Aerated lagoons
Flow regime	....	Mixed surface layer	....	Completely mixed
Pond size, ha	1-4 multipies	1-4 multiples	0.2-1 multipies	1-4 multiples
Operation[a]	Series or parallel	Series or parallel	Series	Series or parallel
Detention time, d[a]	7-30	7-20	20-50	3-10
Depth, m	1-2	1-2.5	2.5-5	2-6
pH	6.5-9.0	6.5-8.5	6.8-7.2	6.5-8.0
Temperature range, °C	0-50	0-50	6-50	0-40
Optimum temperature, °	20	20	30	20
BOD_5 loading, kg/ha · d[b]	15-80	50-200	200-500	
BOD_5 conversion	80-95	80-95	50-85	80-95
Principal conversion products	Algae, CO_2, CH_4 bacterial cell tissue	Alge, Co_2, CH_4, bacterial cell tissue	CO_2, CH_4, bacterial cell tissue	CO_2, bacterial cell tissue
Algal concentration mg/L	20-80	5-20	0-5	
Effluent suspended solids, mg/L[c]	40-100	40-60	80-160	80-250

[a] Depends on climatic conditions

[b] Typical values (much higher values have been applied at various locations). Loading values are often specified by state control agencies.

[c] Includes algae, microorganisms, and residual influent suspended solids. Values are based on an influent soluble BOD_5 of 200 mg/L and, with the exception of the aerobic ponds, an invluent suspended-solids concentration of 200 mg/L.

Note: ha x 2.4711 = acre
m x 3.2808 = ft
kg/ha · d x 0.8922 = lb/acre · d
mg/L = g/m³

Source: Reference 4

Performance: BOD_5 reductions of 75 to 95% have been reported. Effluent suspended solids concentrations of 20 to 150 mg/l can be expected, depending on the degree of algae separation achieved in the last cell. Efficiencies are strongly related to pond depth, detention time and temperature.

Total containment ponds are a variant of facultative lagoons. In the case of total containment however, the design is based on the difference between evaporation and precipitation and the total expected flow. In areas where evaporation exceeds precipitation by a significant margin, and where wastewater flows are relatively small, this option may be used. Large land areas are required. Thus, the option is a good one in regions where land costs are low. Moisture deficit is equal to annual evaporation minus annual precipitation. For various values, the design criteria are as follows (58):

45 inch moisture deficit:

Flow (MGD)	0.25	0.50	1.00	1.25	1.50
Lagoon surface area (acres)	2	4	8	9	12

30 inch moisture deficit:

Flow (MGD)	same				
Lagoon surface area (acres)	3.5	6	12	15	18

15 inch moisture deficit:

Flow (MGD)	same				
Lagoon surface area (acres)	6	12	24	30	37

11.2 Limitations

In very cold climates, facultative lagoons may experience reduced biological activity and treatment efficiency. Ice formation can also hamper operations. In overloading situations, odors can be a problem.

Bacteria, parasite, and virus removal is effective in multiple cell (minimum of 3) wastewater stabilization lagoons, as long as detention times are sufficient (minimum of 20 days).

11.3 Costs

See Figure No. 11.2 (2,11).

11.4 Availability

Fully demonstrated and in moderate use especially for treatment of relatively weak municipal wastewater in areas where real estate costs are not a restricting factor. Such is the case in many developing countries.

11.5 Operation and Maintenance

Facultative lagoons are often operated in series. As a matter of fact, for optimum performance they should be operated this way. When three or more cells are linked, the effluent from either the second or third cell may be recirculated to the first. Recirculation rates of 0.5 to 2.0 times the plant flow have been used to improve overall performance.

Settled solids may require cleaning and removal once every 10 to 20 years.

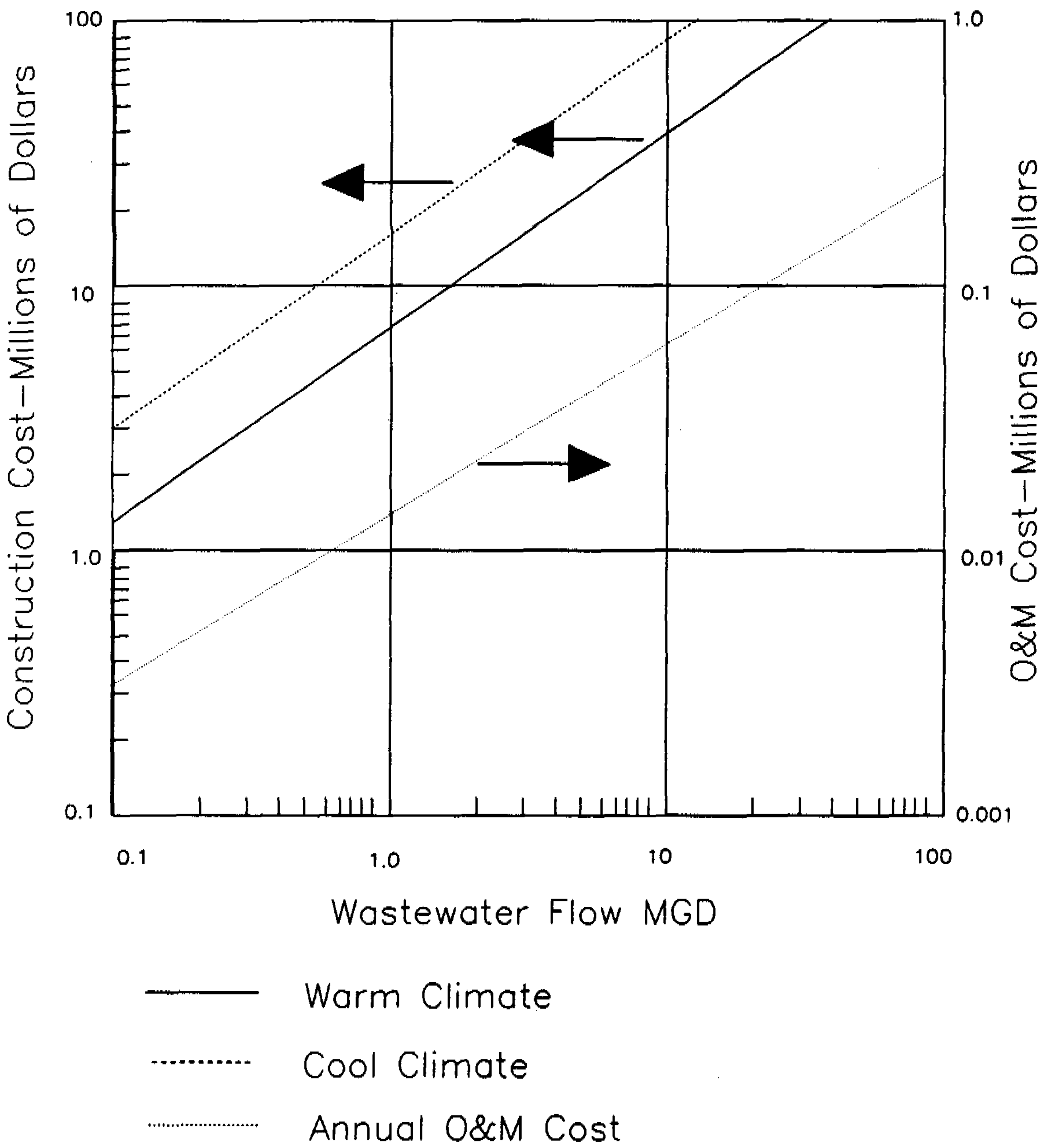
11.6 Control

The service life of a lagoon is estimated to be 50 years. Little operator expertise is required. Overall the system is highly reliable.

11.7 Special Factors

Facultative lagoons are customarily contained in earthen dikes. Depending on soil characteristics, lining with various

Figure 11.2: Construction, Operation & Maintenance Costs for Facultative Lagoons.



(File: Mart555)

impervious materials such as rubber, plastic or clay may be necessary. Use of supplemental top layer aeration can improve overall treatment capacity, particularly in high elevations or cold climates where icing might occur.

If wastewater is nutrient deficient, a source of supplemental nitrogen or phosphorous may be needed. No other chemicals are required.

There is potential for seepage of wastewater into the ground water unless the lagoon is lined. Compared to other secondary biological treatment processes, relatively small quantities of sludge are produced.

11.8 Recommendations

Used for treating raw, screened or primary settled domestic wastewaters and weak biodegradable industrial wastewaters. Most applicable when land costs are not of concern, and operation and maintenance costs are to be minimized. The technology is preferable to mechanical systems where climate is good, and is ideal for many locations in Latin America.

12. AQUATIC PLANT - AQUACULTURE SYSTEM

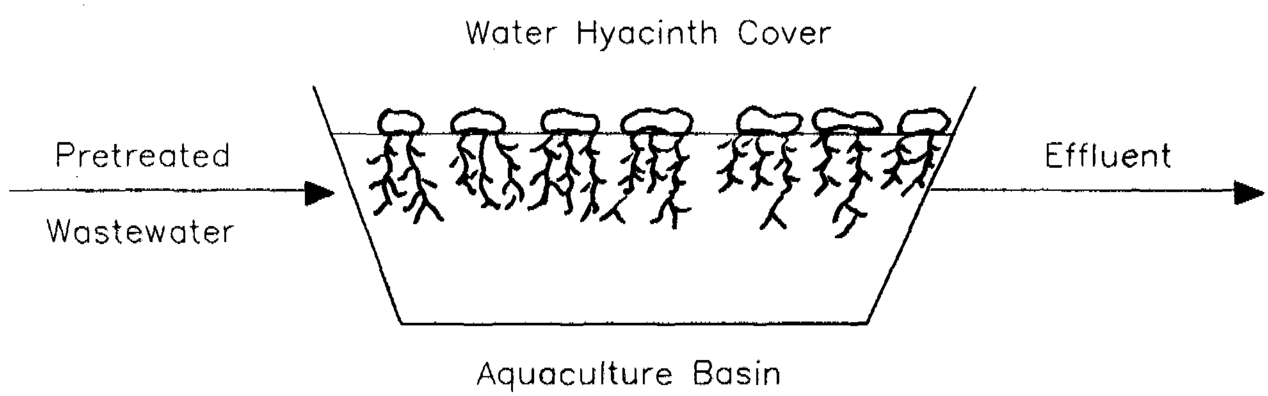
12.1 Description

Aquaculture, or the production of aquatic organisms (both flora and fauna) under controlled conditions, has been practiced for centuries primarily for the generation of food, fiber and fertilizer. The water hyacinth (Eichhornia crassipes) appears to be the most promising organism for wastewater treatment and has received the most attention. Other organisms however are also being studied. Among them are duckweed, seaweed, midge larvae, alligator weeds and a host of other organisms. Water hyacinths are large fast-growing floating aquatic plants with broad, glossy green leaves and light lavender flowers (See Figure 12.1)(2). Figure 12.2 (15) shows a plan and cross section with some design criteria. A native of South America, water hyacinths are found naturally in waterways, bayous and other backwaters. Insects and disease have little effect on the hyacinth, and they thrive in raw, as well as partially treated wastewater.

Wastewater treatment by water hyacinths is accomplished by passing the wastewater through a hyacinth-covered basin, where the plants remove nutrients, BOD_5, suspended solids, metals, etc. Batch treatment and flow-through systems, using single and multiple cell units, are possible. Hyacinths harvested from these systems have been investigated as a fertilizer/soil conditioner after composting, animal feed and as a source of methane when anaerobically digested.

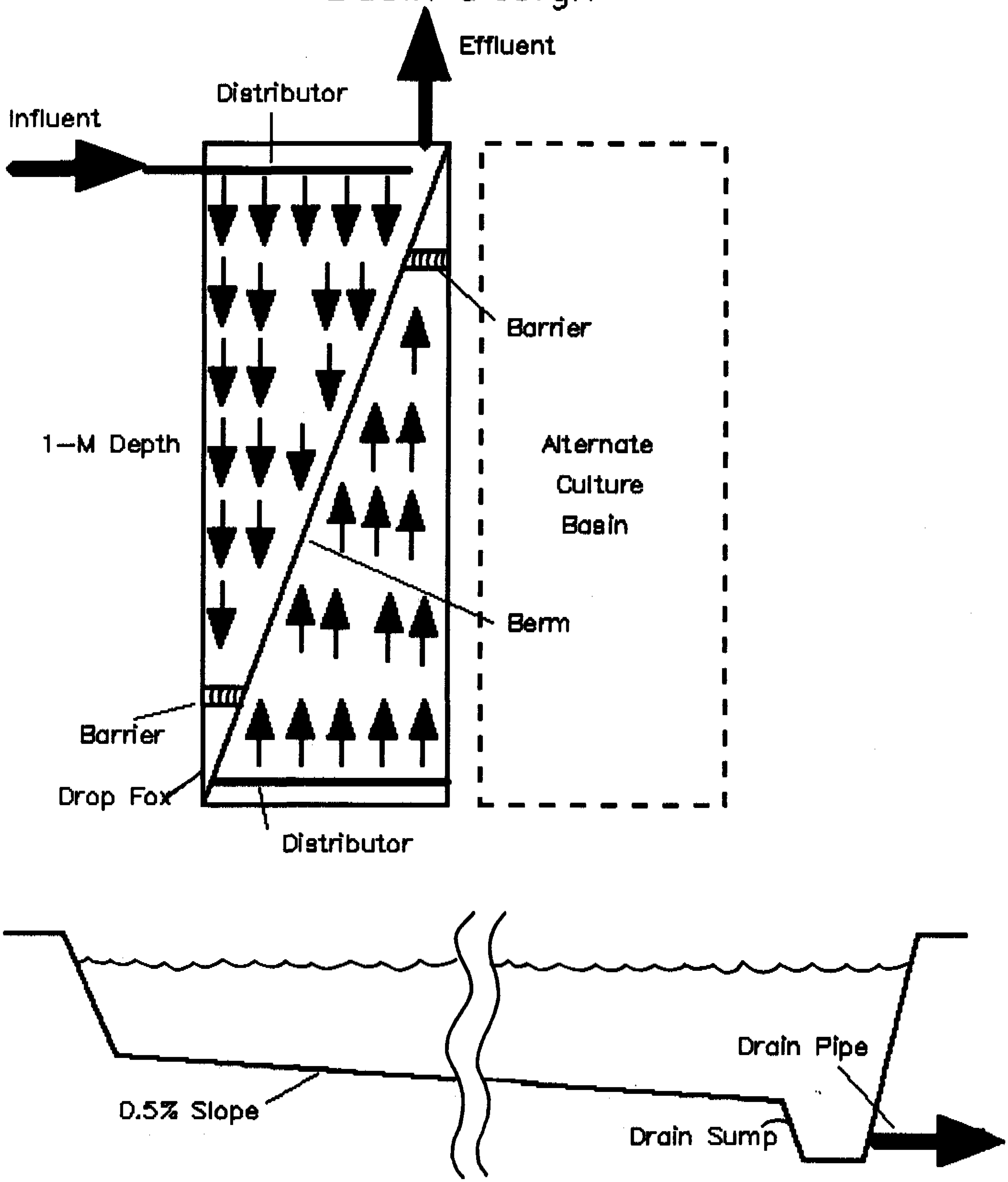
Design Criteria - Experimental data vary widely from different experiences. Ranges herein refer to hyacinth treatment as a tertiary process on secondary effluent. Depth should be sufficient to maximize plant rooting and plant absorption. Detention time: depends on effluent requirements and flow; 4-15 days average; phosphorous reduction: 10 to 75 percent; nitrogen reduction: 40 to 75 percent; land requirement: 2-15

Figure 12.1: Generalized Aquaculture Basin with Hyacinth Plants.



(Source: Ref. 2)

Figure 12.2 Suggested Basic Hyacinth Culture Basin Design



acres/Mgal/d (approx. m^2 per m^3/day). Tables 12.1 (15), and Table 12.2 (2) present design criteria and performance expectations.

Performance - The removal results from five different wastewater streams are given in Table 12.3 (2). There is also some evidence that coliform, heavy metals and organics are removed, as well as pH neutralization.

12.2 Limitations

Climate or climate control is the major limitation. Active growth begins when the water temperature rises above 10 degrees C. and flourishes when the water temperature is 21 degrees C. Plants die rapidly when the water temperature approaches the freezing point, therefore greenhouse structures are necessary in cooler climates. Water hyacinths are sensitive to high salinity. Removal of potassium and phosphorous is restricted to the active growth period of the plants.

Metals such as arsenic, chromium, copper, mercury, lead, nickel and zinc can accumulate in hyacinths and limit their suitability as a fertilizer or feed material. The hyacinths may also create small pools of stagnant surface water which can breed mosquitos. Mosquito problems can generally be avoided by maintaining mosquito fish in the system. The spread of the hyacinth plant itself must be controlled by barriers since the plant can spread and grow rapidly and clog affected waterways. Hyacinth treatment may prove impractical for large treatment plants due to land requirements. Removal must be at regular intervals to avoid heavy intertwined growth conditions. Evapotranspiration can be increased by 2 to 7 times greater than evaporation alone.

Probably the biggest limitation is disposing of the plant mass produced. Where metals are not a problem, the plant may be used for fertilizer and mixed with soil as conditioner. Harvest

TABLE 12.1

SUMMARY OF NUTRIENT LOADING RATES APPLIED TO WATER HYACINTHS WASTEWATER TREATMENT SYSTEMS

Location	Organic Loading Rate	Nutrient Loading Rates to First Unit		Nutrient Removal, %		Comments
	kg BOD_5/ha·day	kg TN/ha·day	kg TP/ha·day	TN	TP	
Williamson Creek, Texas						
Phase I (109 m^3/d) surface area =	43	15.3	-	70	-	Single Basin, 0.0585 ha
Phase II (109 m^3/d)	89	18.5	-	64	-	Single Basin, surface area = 0.0585 ha
Coral Springs, Florida	31	19.5	4.8	96	67	Five Basins in Series Total surface area = 0.52 ha
National Space Technology Labs	26	2.9	0.9	72	57	Single Basin Receiving Raw Wastewater, Surface area = 2 ha

Source: 15

TABLE 12.2

REMOVAL PERFORMANCE OF FIVE WASTEWATER STREAMS
BY AQUACULTURE TREATMENT SYSTEM

Performance - In tests on five different wastewater streams the following removals were reported:

Feed Source	BOD_5 Reduction	COD Reduction	TSS Reducation	N Reduction	Phosphate Reduction
Secondary Effluent	35%	-	-	44%	74%
Secondary Effluent	83%	61%	83%	72%	31%
Raw Wastewater	97%	-	75%	92%	60%
Secondary Effluent	60-79%	-	71%	47%	11%

Source: Reference 2

of the water hyacinth or duckweed plants is essential to maintain high levels of system performance. It is essential for high levels of nutrient removal. Equipment and procedures have been demonstrated for accomplishing these tasks. Disposal and/or reuse of the harvested materials is an important consideration. The water hyacinth plants have a moisture content similar to that of primary sludges. The amount of plant biomass produced (dry basis) in a water hyacinth pond system is about 4 times the quantity of waste sludge produced in conventional activated sludge secondary wastewater treatment. Composting, anaerobic digestion with methane production, and processing for animal feed are all technically feasible, however have not been proven.

12.3 Costs

See Figure 12.3 (2, 5).

12.4 Availability

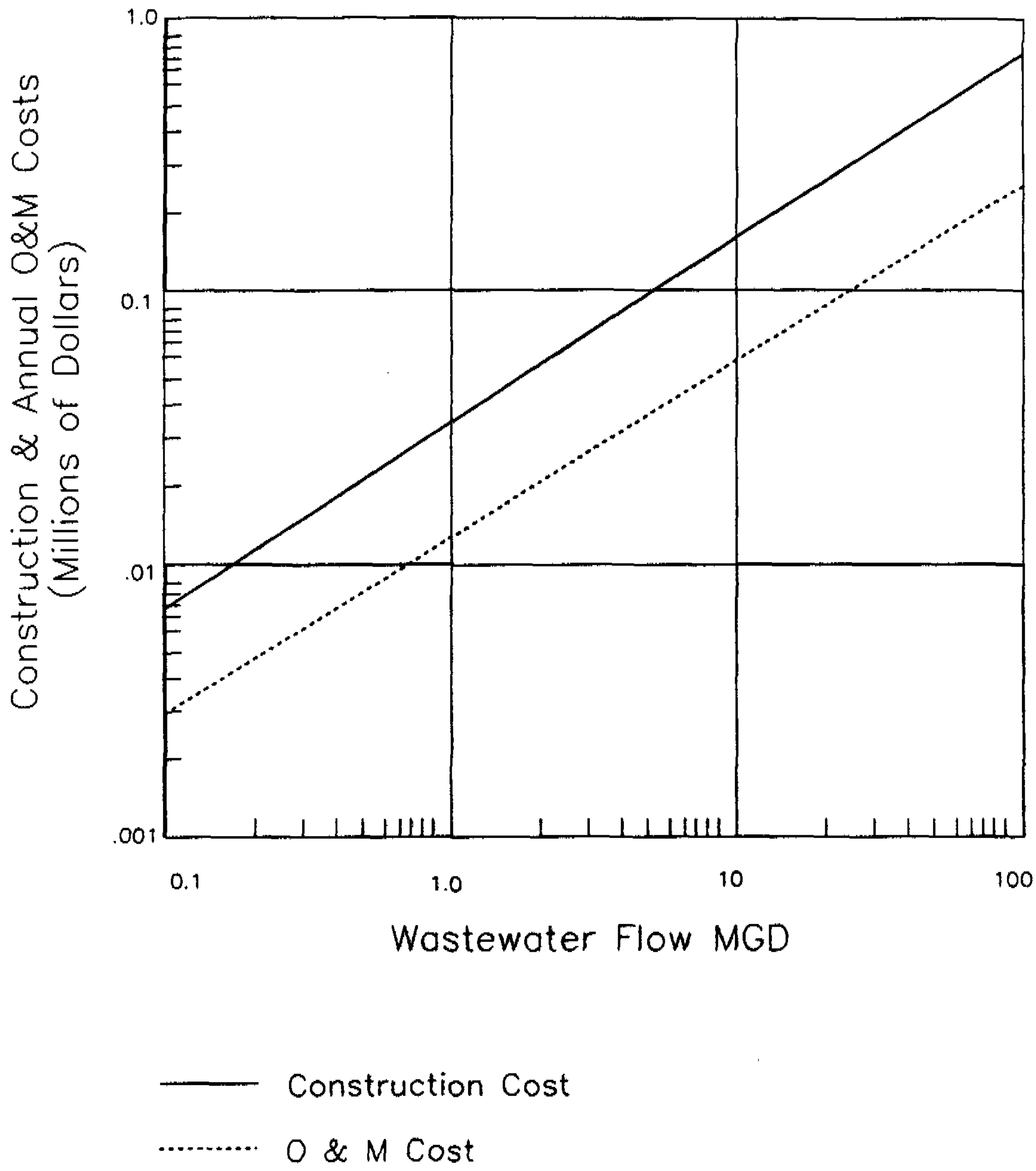
This technology is still in the developmental stage. A number of full-scale demonstration systems are in operation. Systems are in use in Mexico.

12.5 Operation and Maintenance

While the water hyacinth system can successfully cope with a variety of stresses, health of the plants must be maintained for most effective treatment. While the water hyacinth is a hardy, disease-resistant plant that thrives at all above freezing temperatures, its growth rate and nutrient uptake efficiency can be compromised.

Presence of a high chlorine residual definitely inhibits plant growth. If possible, effluent chlorination should be accomplished subsequent to hyacinth treatment. If local conditions dictate pre-hyacinth chlorination, care should be

Figure 12.3: Operation, Maintenance & Construction Costs for Aquatic Plant–Aquaculture System.



(Source: Ref. 2 & 5)

(File: Martin87)

taken that chlorine residual in the influent does not exceed 1 mg/l. Plant health is also adversely affected by chlorides.

Maintenance of nourishment is essential to plant health. Hyacinth has a voracious appetite, which if not satisfied also results in chlorosis and decreased uptake efficiency. Lease efficient performance of the system was obtained during periods of significantly reduced influent flow and during periods when influent nitrogen concentration dropped below 10 mg/l. Plant health is also adversely affected by overcrowding.

The best indication of plant health is an abundant growth of dark green leaves. Any appearance of stunted leaf growth with yellowish green leaves in immature plants or of leaf yellowing in mature plants should be investigated immediately.

Intense sun with temperatures in the mid-nineties may cause some leaf browning and wilting. This is not a serious condition if new growth is present, beneath the brown wilted leaves. Wilted-leaved plants may be removed during the normal harvest cycle by selective harvesting.

Healthiest plant condition and best system performance was obtained when ponds were maintained in a loosely packed condition by a four week harvest cycle. From 15 to 20% of the plants should be removed at each harvest. Uncovering more than 20% of pond surface area will result in an algae problem.

Operation is by gravity flow and requires no energy. Hyacinth growth energy is provided by sunlight.

Operation and maintenance is relatively simple. Maintenance is largely associated with harvesting the plants on a regular basis.

12.6 Control

Hyacinth harvesting may be continuous or intermittent. Studies indicate that average hyacinth production (including 95% water) is on the order of 1,000 to 10,000 lb/d/acre (approx. kg/ha/day). Basin cleaning at least once per year produces harvested hyacinths.

12.7 Special Factors

This technology is generally used in combination with other treatment, such as lagoons.

12.8 Recommendations

The process appears to be reliable from mechanical and process standpoints, but the system is subject to temperature constraints. This technology would be very useful in developing countries with hot climates and where land costs are small.

13. AQUACULTURE - WETLANDS

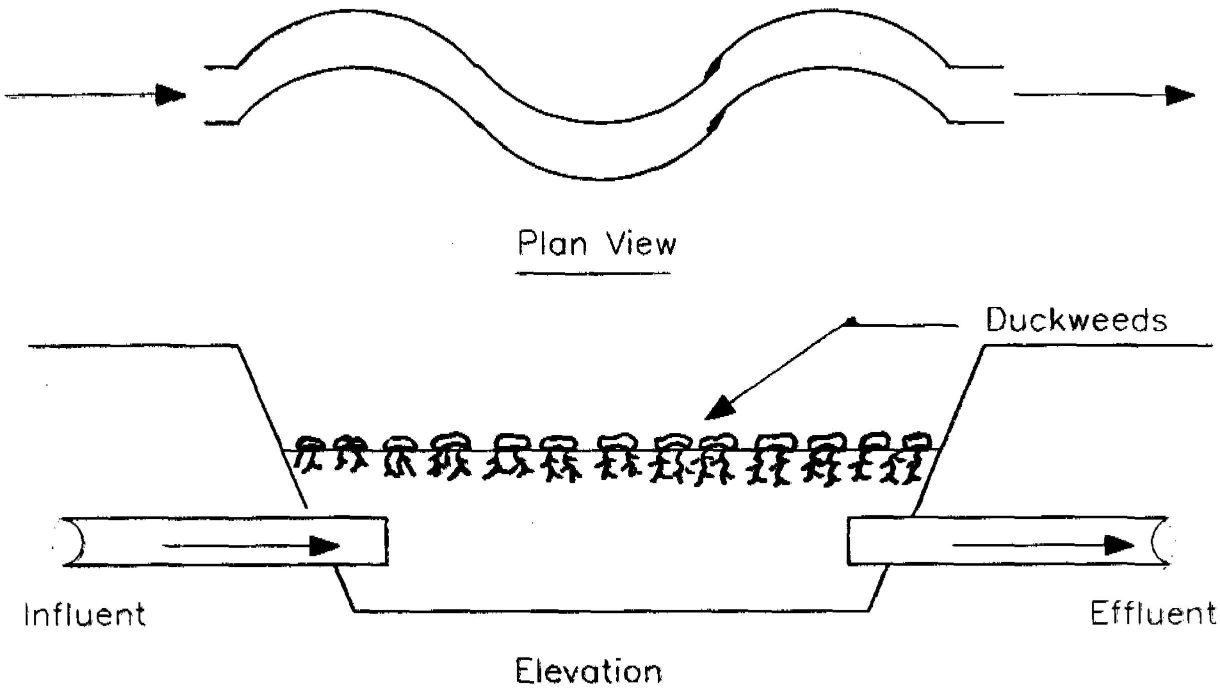
13.1 Description

Aquaculture systems for wastewater treatment include natural and artificial wetlands as well as other aquatic systems involving the production of algae and higher plants (both submergent and emergent), in vertebrates, fish and integrated polyculture food chain systems. Natural wetlands, both marine and freshwater, have inadvertently served as natural waste treatment systems for centuries; however, in recent years marshes, swamps, bogs and other wetland areas have been successfully utilized as managed natural "nutrient sinks" for polishing partially treated effluents under relatively controlled conditions. Constructed artificial wetlands can be designed to meet specific project conditions while providing new wetland areas that also improve available wildlife wetland habitats and the other numerous benefits of wetland areas. Managed plantings of reeds (e.g., Phragmites spp.) and rushes (Scirpus spp. and Schoenoplectus spp.) as well as managed natural and constructed marshes, swamps and bogs have been demonstrated to provide pH neutralization and some reduction of nutrients, heavy metals, organics, BOD_5, COD, TSS, fecal coliforms and pathogenic bacteria. The system is shown schematically in Figure 13.1 (2).

Wastewater by natural and constructed artificial wetland systems is generally accomplished by sprinkling or flood irrigating the wastewater into the wetland area or by passing the wastewater through a system of shallow ponds, channels, basins or other constructed areas where the emergent aquatic vegetation has been planted or naturally grows and is actively growing (See Figure 13.1).

In test units and operating artificial marsh facilities using various wastewater streams, the following removals have been reported for secondary effluent treatment (10 day detention): BOD_5, 80-95%; TSS 29-87%; COD, 43-87%; nitrogen, 42-

Figure 13.1: Generalized Diagram of Aquaculture Wetland.



(Source: Ref. 2)

94%; total Phosphate, 94% (higher levels possible with warm climates and harvesting); coliforms, 86-99%; heavy metals, highly variable depending on the species. There is also evidence of reductions in wastewater concentrations of chlorinated organics and pathogens, as well as pH neutralization without causing detectable harm to the wetland ecosystem.

Design parameters can be found in Table 13.1.

13.2 Limitations

Temperature (climate) is a major limitation since effective treatment is linked to the active growth phase of mergent (surface and above) vegetation. Herbicides and other materials toxic to the plants can affect their health and lead to poor treatment. Duckweeds are prized for food of waterfowl and fish and can be seriously depleted by these species. Winds may blow duckweeds to the shore if wind screens of deep trenches are not employed. Small pools of stagnant surface water which can allow mosquitos to breed can develop, but problems can generally be avoided by maintaining mosquito fish or a healthy mix of aquatic flora and fauna in the system. Wetland systems may prove impractical for large treatment plants due to the large land requirements. Evapotranspiration increases are likely.

13.3 Costs

The generalized construction cost and operation and maintenance costs are shown in Figure 13.2 (2). The case study examples of actual construction costs and operation and maintenance costs for wetland treatment systems at Vermontville and Houghton Lake, Michigan are shown in Table 13.1 (16).

Figure 13.2 and Table 13.2 also show the operation and maintenance costs.

Figure 13.2: Construction, Operation & Maintenance Costs for Aquaculture - Wetlands.

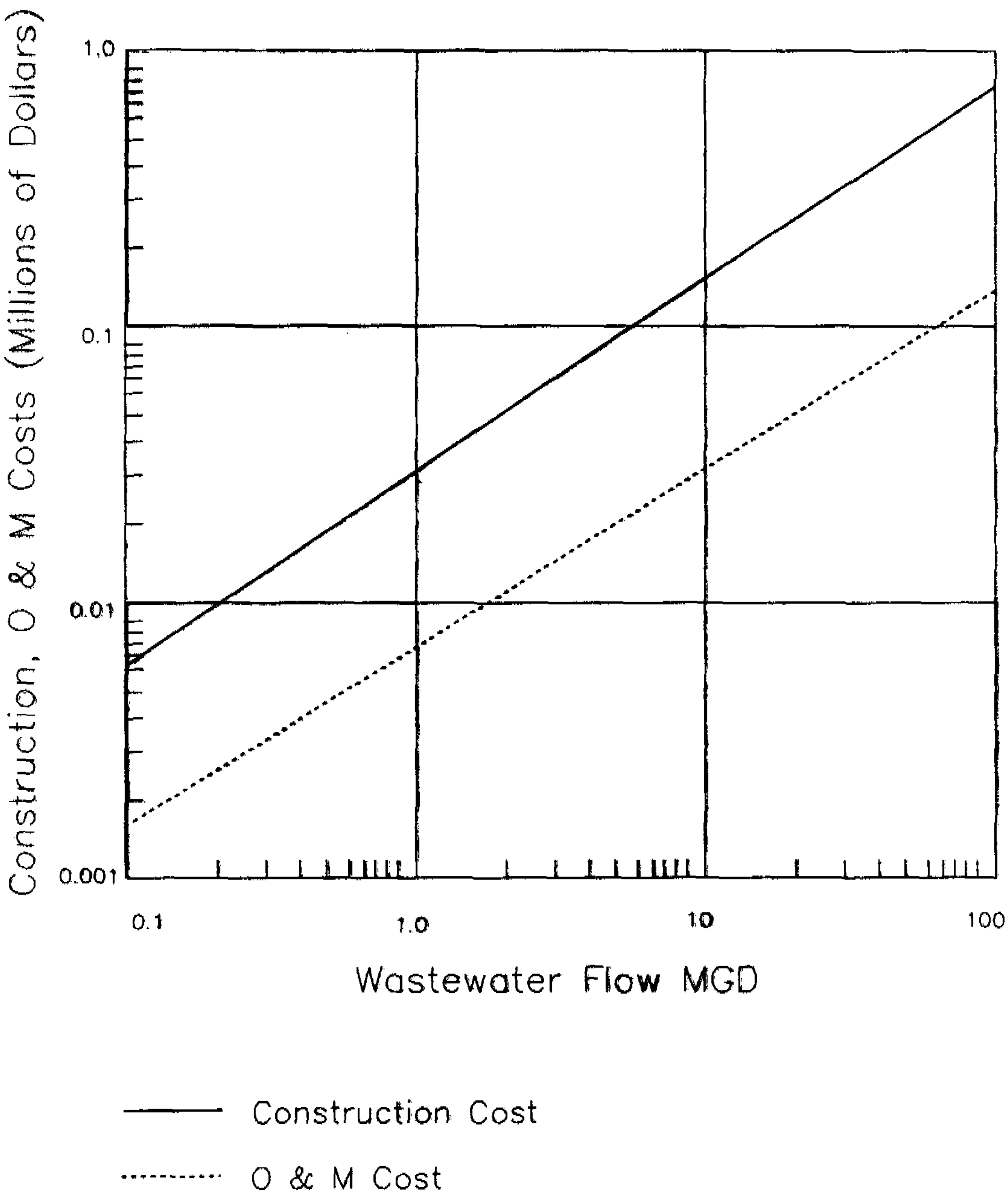


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TABLE 13.1

PRELIMINARY DESIGN PARAMETERS FOR PLANNING ARTIFICIAL WETLAND WASTEWATER TREATMENT SYSTEMS[a]

Type of system	Flow regime[b]	Characteristic/design parameter					
		Detention time, d		Depth of flow, ft (m)		Loading rate g/ft² d (cm/d)	
		Range	Typ.	Range	Typ.	Range	Typ.
Trench (with reeds or rushes)	PF	6-15	10	1.0-1.5 (0.3-0.5)	1.3 (0.4)	0.8-2.0 (3.25-8.0)	1.0 (4.0)
Marsh (reeds rushes, other)	AF	8-20	10	0.5-2.0 (0.15-0.6)	0.75 0.25	0.2-2.0 (0.8-8.0)	0.6 (2.5)
Marsh-pond							
1. Marsh	AF	4-12	6	0.5-2.0 (0.15-0.6)	0.75 (0.25)	0.3-3.8 (0.8-15.5)	1.0 (4.0)
2. Pond	AF	6-12	8	1.5-3.0 (0.5-1.0)	2.0 (0.6)	0.9-2.0 (4.2-18.0)	1.8 (7.5)
Lined trench	PF	4-20 (hr.)	6 (hr.)	-	-	5-15 (20-60)	12 (50)

[a] Based on the application of primary or secondary effluent.

[b] PF = plug flow, AF = arbitrary flow.

Source: Reference 59

TABLE 13.2

WETLANDS COST SUMMARY

Category	Vermontville, MI 11-Acre Constructed Wetland (0.07 MGD)		Houghton Lake, MI Natural Wetland (1.1 MGD Summer/ 0.42 MGD Winter)	
	Cost	% of Total	Cost	% of Total
Labor (including Overhead/Administration)	$ 1,900	56.1	$ 4,750	53.9
Electrical Energy	372	11.2	2,400	27.4
Equipment Use (including Fuel)	1,100	31.7	660	7.5
Repair/Replacement	33	1.0	165	1.9
New Equipment	0	0	275	9.4
Totals	$ 3,300	100%	$ 8,800	100%
Cost Breakdown for Million Gallons Treated	$ 128		$ 54	

Source: Reference 14

13.4 Availability

This technology is in the developmental stage. Several full scale, demonstration, experimental systems are in operation or under construction.

13.5 Operation and Maintenance

Vegetation is the main form of erosion control and works quite well once established. A minimum of one spring and summer are needed before the vegetation can become established, without specific planting and cultivation. Vegetation is not sufficient around weirs, gates and pipes. These areas must be fortified with riprap. The District is fortunate in this respect because it is located en route to the local landfill and gets all its riprap free of charge.

By dividing the total area designated for the wetlands into plots more habitat goals may be achieved. When a multiple plot system is created flow variation is facilitated. This allows one plot to be isolated from the system in case of major maintenance needs. Multiple plots also allow depth variation. Depth is a key factor in habitat design: it will determine whether or not emergent vegetation will be present and will affect temperature and dissolved oxygen values. Plot shapes may vary but small, constricted areas should be avoided as they would promote stagnation and vector problems. Deciding which groups of organisms are desired in the wetlands and knowing what conditions these organisms normally live under will determine the fundamental components of the design.

Clostridum botulinum is the cause of avian botulism and will not cause botulism in humans. It is, however, deadly to waterfowl and certain measures may be taken to avoid its occurrence. Avoiding anaerobic conditions by keeping the water circulating and maintaining the depth under 3' is an important factor in botulism avoidance. Removal of floating organic debris which collects behind weirs and in corners is regularly done. Steep-sided levees, adjustable broad crested weirs for controlling water levels, conveying water by pipeline, and ability to shunt a plot out of service for draining, are also factors in the botulism avoidance program.

Mosquitoes lay eggs in water and the larva grow there undergoing metamorphosis to the adult form. To breathe the larva must hang from the surface film of the water, piercing it with their respiratory tube to obtain oxygen. This knowledge of the mosquito life cycle and habitat needs helps the wetlands manager avoid mosquito breeding problems. Open water areas, subject to wind action and providing easy access for predators, limit mosquito production. Maintaining good circulation in vegetated areas provides for predator access and lessens mosquito production.

The vegetation produced as a result of the system's operation may or may not be removed and can be utilized for various purposes (e.g., composted for use as a fertilizer/soil conditioner, dried or otherwise processed for use as animal feed supplements, or digested to produce methane).

13.6 Control

Low operator attention is required if properly designed.

13.7 Special Factors

Tie-ins with cooling water from power plants to recover waste heat have potential for extending growing seasons in colder climates. Enclosed and covered systems are possible for small flows.

13.8 Recommendations

This technology is useful for polishing treated effluents. It has potential as a low cost, low energy consuming alternative or addition to conventional treatment systems, especially for small flows. It has been successfully used in combination with chemical addition and overland flow land treatment systems. Wetland systems may also be suitable for seasonal use in treating wastewaters from recreational facilities, some agricultural operations, or other waste-producing units where the necessary land area is available. Finally, it also has potential application as an alternative to lengthy outfalls extended into rivers, etc. and as a method of pretreatment of surface waters for domestic supply, storm water treatment and other purposes.

14. PRELIMINARY TREATMENT

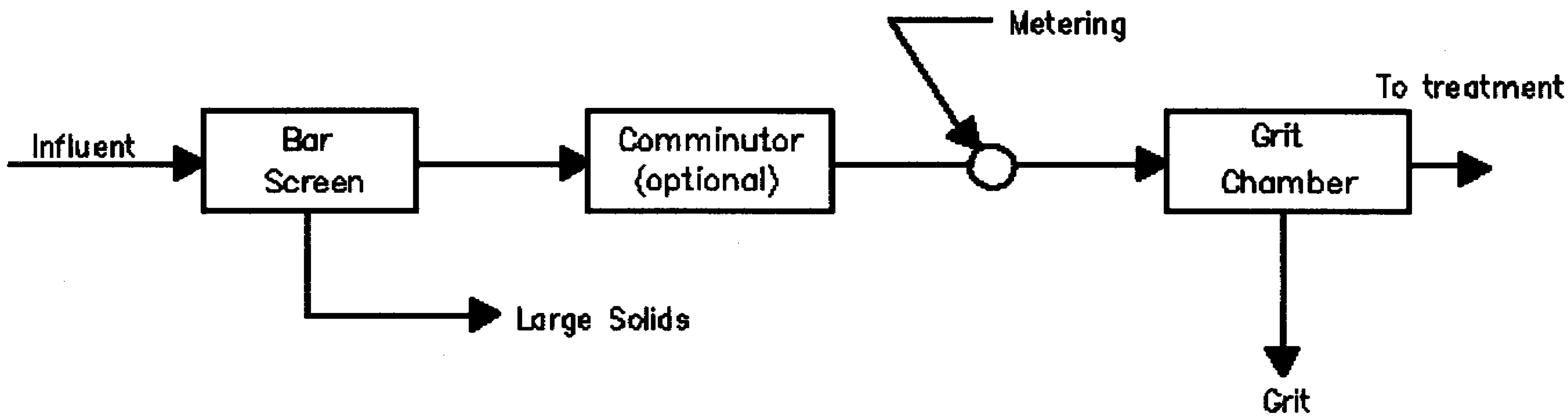
14.1 Description

Preliminary treatment is used to remove grit, heavy solids and floatable material from municipal wastewater by using coarse screening (bar racks), medium screening, and comminution/grinding. In conventional wastewater treatment or raw water supply treatment, the preliminary treatment system is used to protect pumps, valves, pipelines, and other appurtenances from damage or clogging by large solids or high density materials. Preliminary treatment will also remove large particulate material, thus reducing loadings on following processes.

Preliminary treatment typically consists of bar screens and grit chambers. These two appurtenances are available in varying sizes with several maintenance options, such as hand-cleaned, and/or mechanically cleaned depending on the size of treatment plants. A typical flow diagram of preliminary treatment is shown in Figure 14.1 (2).

Design Criteria - the arrangement of preliminary treatment units varies depending on sewage wastewater characteristics and/or subsequent treatment processes. The preliminary system may also include flow measurement devices such as flumes. Also, low lift pumping may be included to adjust for operating head losses in the subsequent treatment processes. A few general design rules are followed (17): Bar Screen - Bar size 1/4 to 5/8 inch (0.6 to 1.6 cm) width by 1 to 3 inch (2.54 to 7.2 cm) depth; spacing 0.75 to 3 inch (1.9 to 7.2 cm). Slope - from vertical to 45°, velocity 1.5 to 3 ft/s (0.5 to 0.9 m/s). The typical grit removal chambers; horizontal velocities of 0.5 to 1.25 ft/s (0.15 to 0.4 m/s). Sufficiently long retention times should be provided in the grit chambers to settle the lightest and smallest grit particles. This may be between 10 and 30 minutes.

Figure 14.1: Flow Diagram of Preliminary Treatment



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14.2 Limitations

Bars and screens require regular cleaning. If the cleaning is by mechanical means, preventive maintenance is required on a regular basis, especially in those cases where solids are heavy and solids concentrations are high. This may be during periods of rainfall for both water and wastewater treatment systems. Operational problems have been experienced if comminutors are used in certain installations with a heavy influx of plastic or high density objects.

14.3 Costs

The construction costs are shown on Figure 14.2 (2,11), for flow channels and superstructures, bar screen (mechanical), horizontal grit chamber with mechanical grit handling equipment, Parshall flume and flow recording equipment. Figure 14.2 also shows the operation and maintenance cost (cost for grit disposal not included).

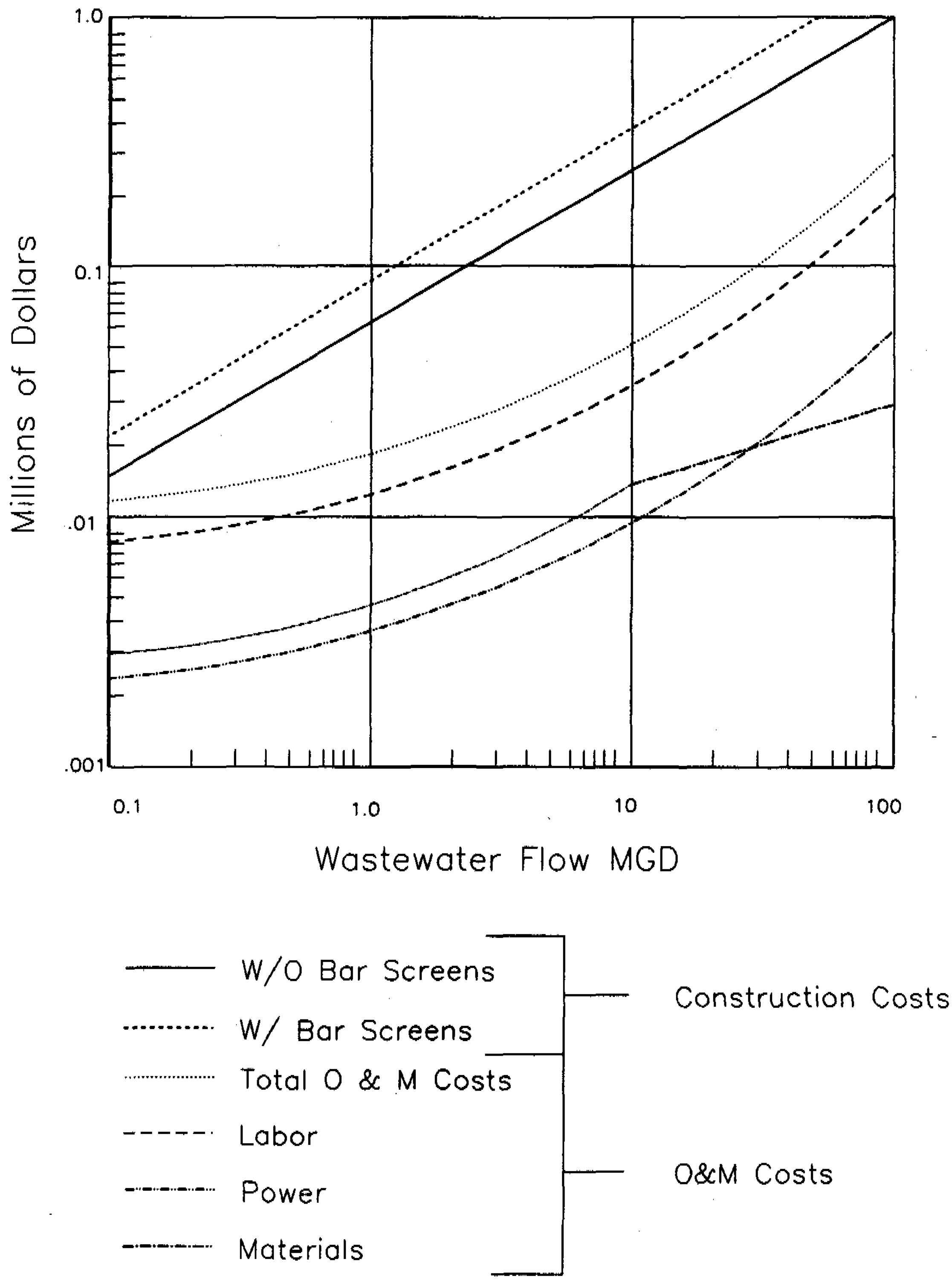
14.4 Availability

Preliminary treatment has been used since the earliest days of municipal wastewater treatment and water supply treatment.

14.5 Operation and Maintenance

Preliminary treatment usually consists of two separate and distinct unit operations - bar screening and grit removal. There are two types of bar screens or racks. The most commonly used and oldest technology, consists of hand-cleaned bar racks. Hand cleaning is generally used in smaller treatment plants. The second type of bar screen is the one that is mechanically cleaned, (commonly used in larger facilities).

Figure 14.2: Construction, Operation & Maintenance Costs for Preliminary Treatment.



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Grit is most commonly removed in chambers, which are capable of settling out high density solid materials, such as sand and gravel. There are two types of grit chambers: horizontal flow and aerated. In both types the settleables collect at the bottom of the unit. The horizontal units are designed to maintain a relatively constant velocity by use of proportional weirs of flumes in order to prevent settling of organic solids, while simultaneously obtaining relatively complete removal of the inorganic grit.

The aerated type produces spiral action. The heavier particles remain at the bottom of the tank to be removed, while lighter and generally organic particles are maintained in suspension by rising air bubbles. Advantages of aerated units are that the amount of air can be regulated to control the amount of grit/organic solids separation, and offensive odors are controlled. The aeration process also facilitates cleaning of the grit. The grit removed from horizontal flow units usually needs additional cleaning steps prior to disposal.

All unit operations, except for the ones with comminutors, will generate solids that will need disposal. Screens remove up to 1 yd^3 of 12 to 15% solids/Mgal. But this is very much related to the character of the water being treated. The grit and other solids are often landfilled.

14.6 Control

Preliminary treatment systems are extremely reliable and, in fact, are designed to improve the reliability of downstream treatment systems.

14.7 Special Factors

Many plants often use comminutors. These are mechanical devices that grind up the material normally not removed in the screening process. Therefore, these solids remain in the wastewater to be removed in downstream unit operations.

In recent years, the use of static or rotating wedge-wire screens has increased. These remove large organic particles just prior to degritting. These units have been found to be superior to comminutors in that they remove the material immediately from the waste instead of creating additional loads downstream. Other grit chamber designs are available including swirl concentrators and square tanks.

Odors are common when removed grit contains excess organic solids which are not disposed of a short time after removal.

14.8 Recommendations

This technology should be used at all municipal wastewater treatment plants and water treatment plants with the potential for high solids in the influent. They are also often used prior to wastewater pumping stations.

15. HORIZONTAL SHAFT ROTARY SCREEN

15.1 Description

The rotating drum filter operates intermittently or continuously, and can be used as needed. The rotating drum is covered with a plastic or stainless steel screen of uniform sized openings, installed and partially submerged in a chamber (see Figure 15.1) (2). The chamber is designed to permit the entry of water to the interior of the drum and collection of filtered (or screened) water from the exterior side of the drum. Coarse screens have openings of 1/4 inch or more; fine screens have openings of less than 1/4 inch. Screens with openings of 20 to 70 microns are called microscreens or microstrainers. Drum diameters are 3 to 5 ft with 4 to 12 ft lengths.

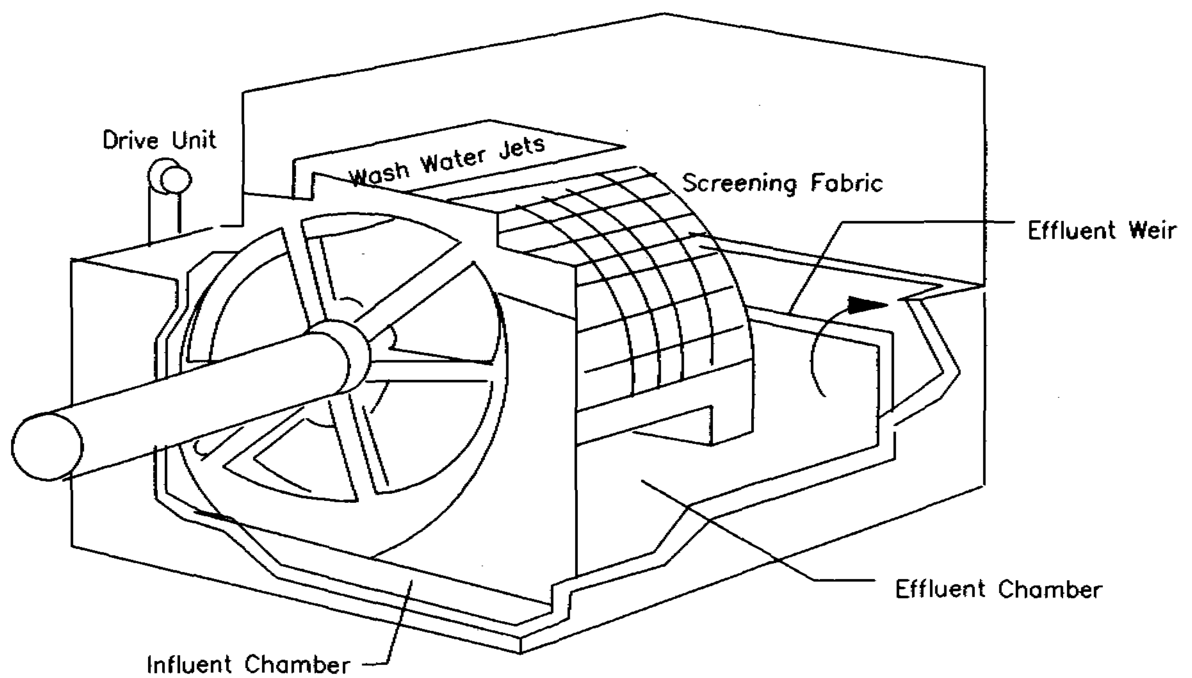
Design Criteria - Screen submergence is 70 to 80%. Loading rate: 2 to 10 gal/min/ft2 (0.82 to 4.1 $l/min/m^2$) of submerged area depending on pretreatment and mesh size (2). Flow rates may be as high as 10 to 30 gpm/ft^2 (4.1 to 12.3 $l/min/ft^2$), at head losses around 12 to 18 inches (0.3 to 0.46 m) of water through the filter system (55). Screen openings: 150 microns to 0.4 inches for pretreatment; 20 to 70 microns for fine particle removal. Drum rotations /min: 0 to 7. Screen materials: stainless steel or plastic cloth. Washwater = 2 to 5% of flow being treated. Performance of fine screen device varies considerably depending on influent solids type, concentration and loading patterns; mesh size, and hydraulic head.

Typical removal rates for some pollutants are as follows: BOD_5, 40 - 60% and SS, 50 - 70%. Head loss is usually from 0.3 to 2 ft.

15.2 Limitations

There is a dependence on pretreatment and inability to

Figure 15.1: Horizontal Shaft Rotary Screen



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handle solids fluctuations in tertiary applications. Reducing the speed of rotation of the drum and less frequent flushing of the screen has resulted in increased removal efficiencies, but reduced capacities.

15.3 Costs

See Figure 15.2 (2).

15.4 Availability

Widespread use for roughing pretreatment, and for secondary biological plant effluent polishing.

15.5 Operations and Maintenance

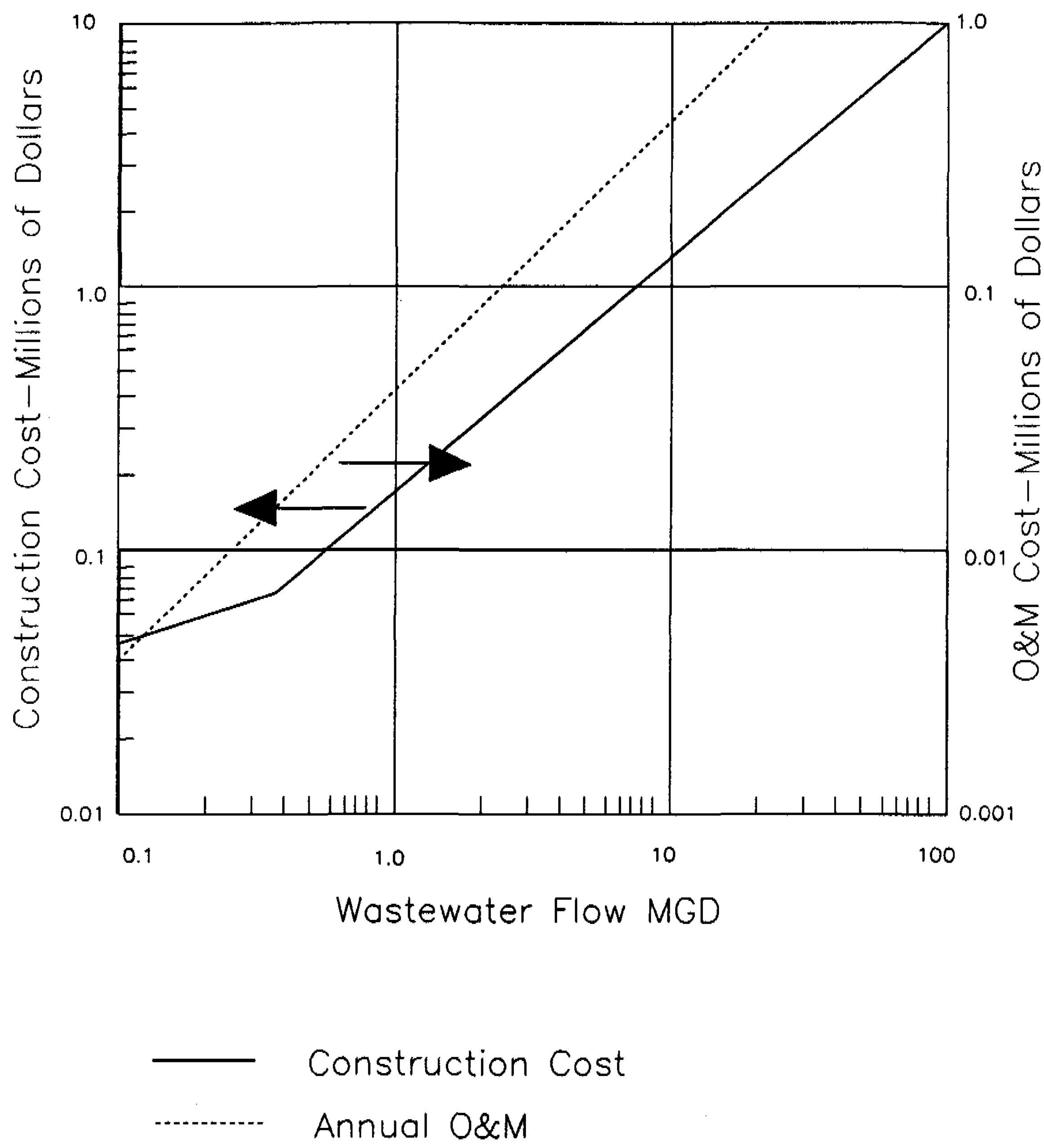
With each revolution of the drum, the solids are flushed from the exposed screen surface into a collecting trough, by water sprays.

Odor problems around equipment may be created if solids are not flushed frequently enough from the screen. This is particularly true if the screens are used in the pretreatment mode.

15.6 Control

There is a high degree of reliability from both the process and mechanical viewpoints. The process is simple to operate. Mechanical equipment is generally simple and straightforward. Occasional problems may arise because of incomplete solids removal by flushing. Hand cleaning with acid solution may be required for stainless steel cloths. Blinding by grease may be a problem when the screens are used in pretreatment applications.

Figure 15.2: Construction, Operation & Maintenance Costs for Horizontal Shaft Rotary Screen.



(File: Martin99)

15.7 Special Factors

Some common modifications -

- Tile chamber, reinforced concrete chamber, steel chamber, for special waste applications.
- Variable speed drive for drum.
- Addition of backwash storage and pumping facilities.
- Addition of ultra-violet light slime growth control equipment.
- Addition of chlorinating equipment.

15.8 Recommendations

This technology is most useful in the removal of coarse wastewater solids from the wastewater treatment plant influent after bar screen treatment; screen openings 150 microns to 0.4 inches. Also for polishing activated sludge effluent, screen openings 20 to 70 microns.

Screens are used in water supply source applications to protect against such things as leaves. Travelling, rather than rotary screens have been used successfully (48), with not less than 2 and sometimes as many as 8 meshes to the inch (79 to 315 per meter). Travelling screens should have a velocity of 3 1/2 inches/sec (8.89 mm/sec). The best operating region for screens is the 50 to 100 micron range (55).

16. WEDGEWIRE SCREEN

16.1 Description

Screening is used to remove coarse and/or gross solids from water or wastewater before subsequent treatment.

A wedgewire screen is a device onto which water to be treated is directed across an inclined stationary screen or a drum screen of uniform sized openings. Solids are trapped on the screen surface while the wastewater flows through the screen openings. The solids are moved either by gravity (stationary) or by mechanical means (rotating drum) to a collecting area for discharge. Stationary screens introduce the wastewater as a thin film flowing downward with a minimum of turbulence across the wedgewire screen, which is generally in three sections of progressively flatter slopes (See Figure 16.1) (2).

The design criteria of wedgewire screening based on wastewater flow over a very wide range, 0.05 to 36 MGD (0.0022 to 1.6 m3/sec):

Stationary	Parameter	Rotary Drum
0.01 to 0.06 inch	Screen opening	0.01 to 0.06 inch
4 to 7 ft	Head required	1.5 to 4.5 ft
10 to 750 ft	Surface required	10 to 100 ft2
--	Motor size	0.5 to 3 hp

Source: 13

1 inch = 2.54 cm; 1 m = 3.28 ft; 746 W = 1 hp

Pollutant removals are: BOD_5, 5% to 20% and SS, 5% to 25%. Head loss can be 4 to 8 ft.

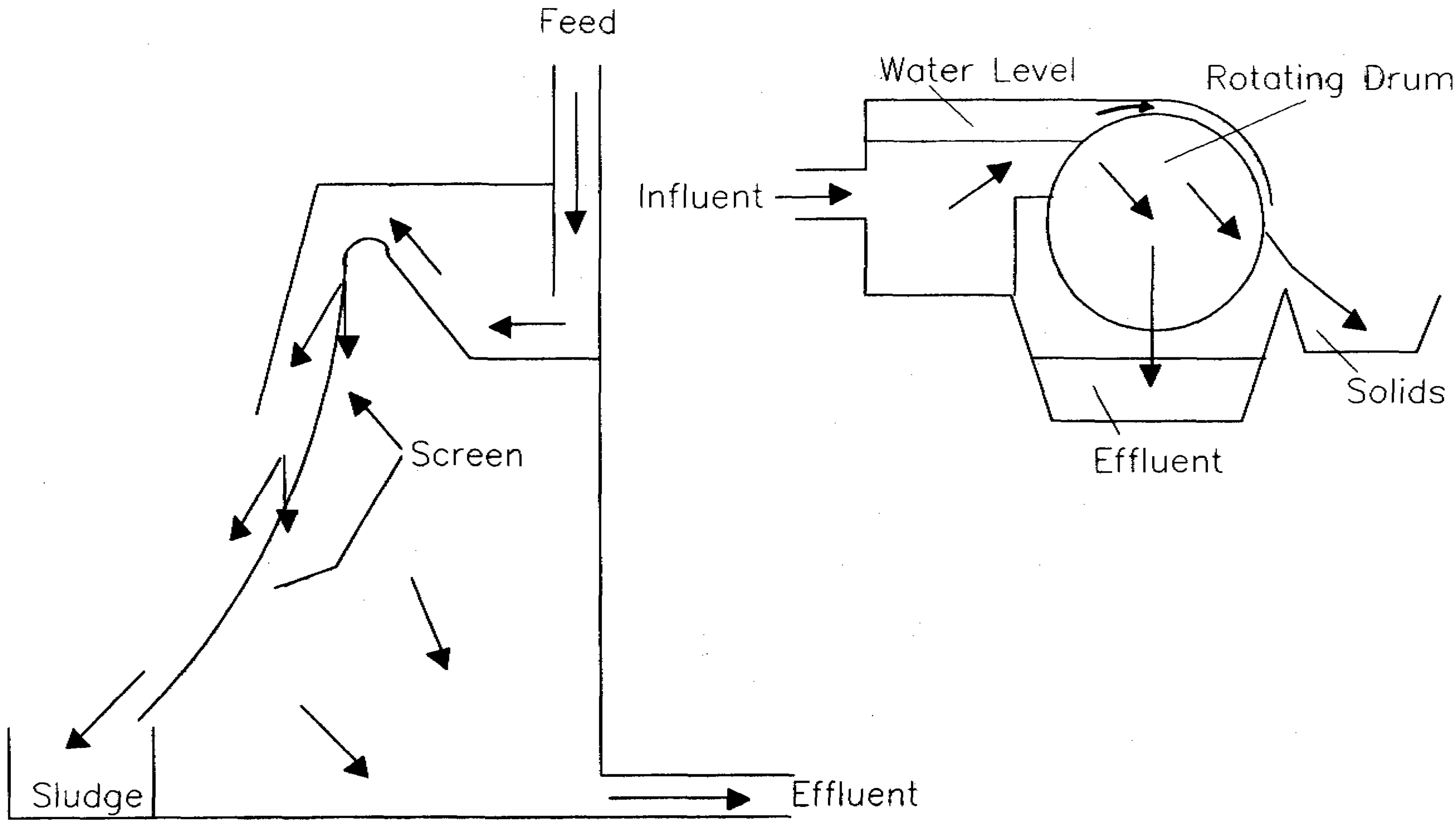


Figure 16.1: Typical Wedgewire Screen

(Source Ref. 13)

16.2 Limitations

Requires regular cleaning and prompt residuals disposal.

16.3 Costs

Figure 16.2 (2) shows the construction and O&M costs for wedgewire screen. The cost is based on wedgewire stainless steel screen 0.06 inch openings, equipment and installation. Pumping equipment and piping for effluent or sludge is included. Operation and maintenance cost is based on labor costs at $7.50/hour, power at $.02/Kwh and pumping head (for stationary screen), 4.5 feet.

16.4 Availability

This technology has been in use since 1965 and in municipal wastewater treatment since 1967. There are over 100 installations to date in the U.S., and many more worldwide.

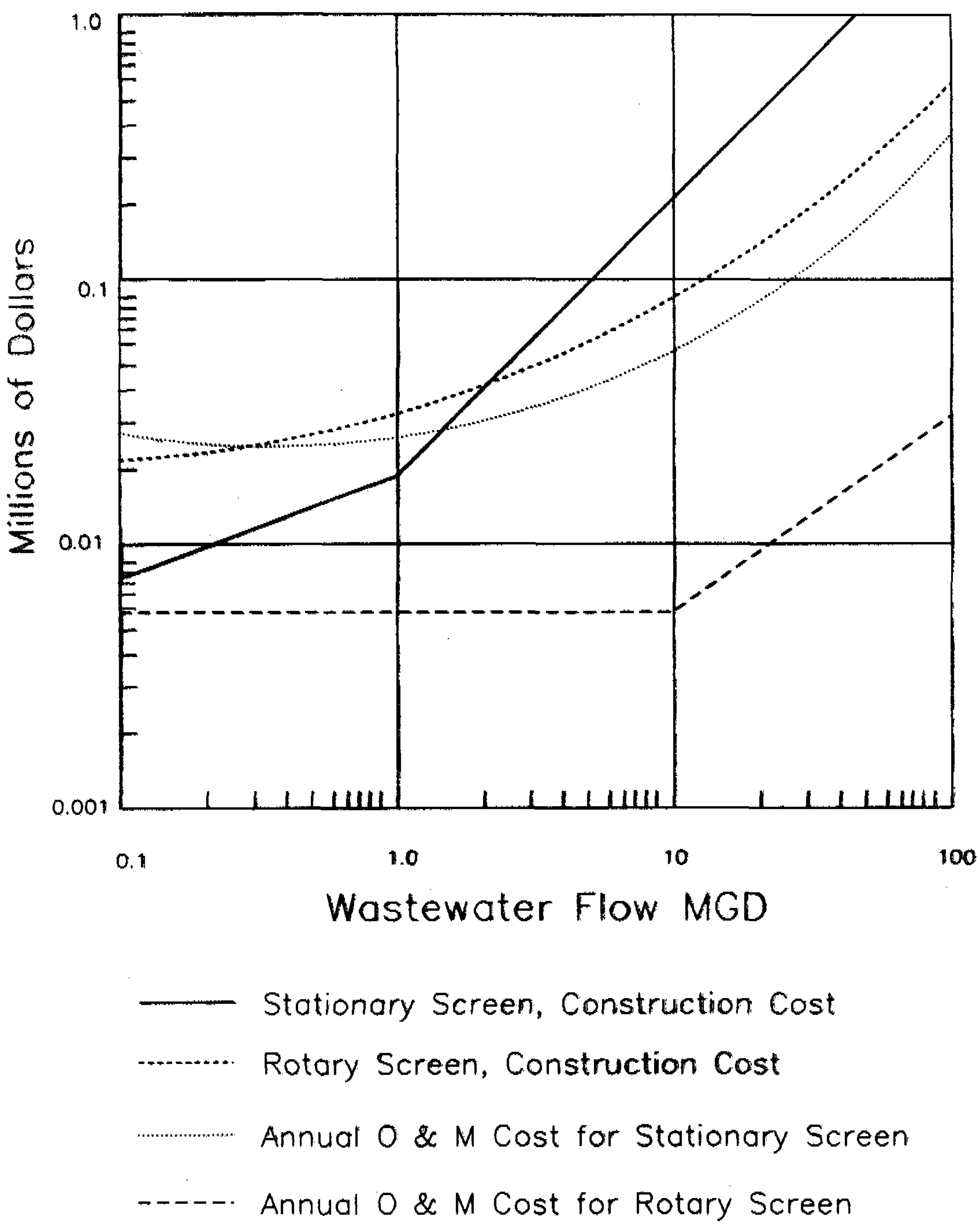
16.5 Operation and Maintenance

Solids are trapped on the screen surface while the wastewater flows through the openings. The solids are moved either by gravity (stationary) or by mechanical means (rotating drum) to a collecting area for discharge. Frequent cleaning of the screen medium is required. Wastes with high acidity or strong base characteristics, or strongly corrosive wastes, will react with some metals and fabrics.

16.6 Control

Screens have a relatively high mechanical reliability for many applications. Performance for solids removal to meet preset effluent requirements may be expected to be consistently high when cleaning and maintenance practices are good.

Figure 16.2: Construction, Operation & Maintenance Costs for Wedgewire Screen



(File: Mart58)

16.7 Special Factors

Wedgewire screen spacing should be selected based on specific applications, especially solids characteristics. For municipal wastewater treatment applications, spacings are generally between 0.01 and 0.06 inches (0.25 mm to 1.5 mm). Inclined screens can be housed in stainless steel or fiberglass; wedgewires may be curved or straight; the screen face may be a of single or multi-angle unit design, three separate multi-angle pieces, or a single curved unit. Rotary screens can have a single rotation speed drive or a variable speed drive.

16.8 Recommendations

Stationary and rotary drum screens are ideally suited to use following bar screens and prior to grit chambers. They have also been employed for primary treatment, scum dewatering, sludge screening, treatment of digester cleaning discharges, and for storm water overflow treatment. Generally, the rotary drum unit is preferred. Where grease problems exist, increased frequency of cleaning is required. especially for stationary units.

17. TRICKLING FILTER. PLASTIC MEDIA

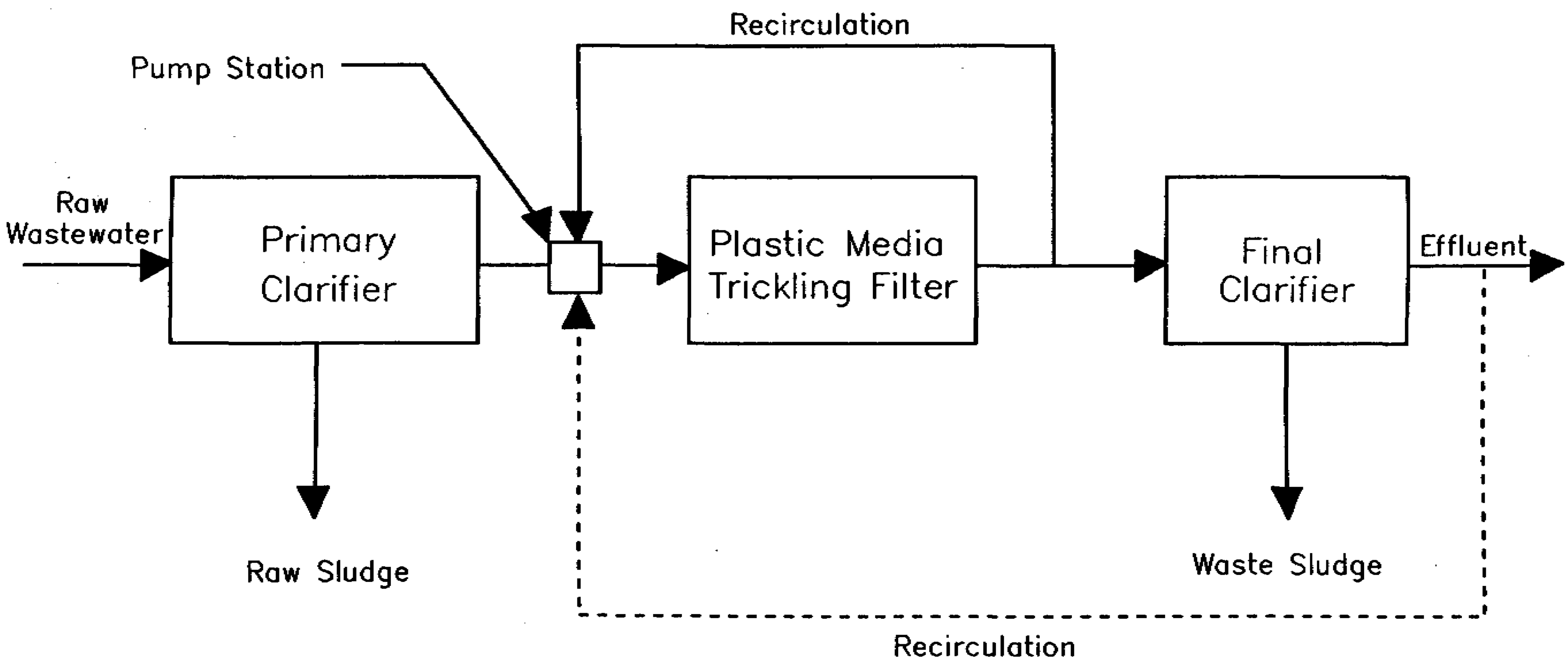
17.1 Description

The process consists of a fixed bed of plastic media over which wastewater is applied for aerobic biological treatment. The bed is dosed by a distributor system, and the treated wastewater is collected by an underdrain system. Primary treatment is normally required to result in satisfactory operation and performance. The process is shown on Figure 17.1. Recirculation may be accomplished with filter or settled effluent as shown.

The rotating system for distribution of the wastewater on the filter medium has become standard practice because of its reliability, ease of maintenance, but mostly because of improved performance. However, fixed nozzles are often used in roughing filters. Plastic media is comparatively light with a specific weight 10 to 30 times less than the rock media. Its high void space (about 95%) promotes better oxygen transfer to fixed organisms during passage of wastewater through the filter, than rock media. Rock media has approximately 50% void space. Because of its light weight, plastic media containment structures are normally built as elevated towers 20 to 30 ft high. The construction cost is much lower than other options. Excavated containment structures for rock media can sometimes serve as a foundation for elevated towers when converting an existing facility to plastic media, and where capacity expansion is desired.

Plastic media trickling filters can be employed to provide independent secondary treatment or roughing ahead of a second-stage biological process. When used for secondary treatment, the media bed is generally circular in plan and dosed by a rotary distributor. Roughing applications often utilize rectangular media beds with fixed nozzles for distribution.

Figure 17.1: A Flow Diagram of Trickling Filter Plastic Media



(Source: Ref. 2)

Design Criteria are presented on Table 17.1 (2). Using a single-stage configuration with filter effluent recirculation and primary and secondary clarification, removal rates for some pollutants are as follows: BOD_5, 80-90% Phosphorous, 10-30%; NH4-N, 20-30%; SS, 80-90%.

17.2 Limitations

This technology provides marginal treatment capability in single stage operation. It is less effective in treatment of wastewater containing high concentrations of soluble organics. It also has limited flexibility and control in comparison with competing biological treatment processes, particularly the activated sludge options. There is a potential for vector and odor problems, although high rate systems have less problems in this regard than low rate trickling filters.

17.3 Costs

See Figure 17.2 (2,11).

17.4 Availability

This technology has been used as a modification for rock media filters for 20 to 30 years.

17.5 Operation and Maintenance

The organic material present in the wastewater is degraded by a population of microorganisms attached to the filter media. As the microorganisms grow, the thickness of the slime layer increases. Periodically the liquid will wash some of the slime off the media, and a new slime layer will start to grow. This phenomenon of losing the slime layer (sloughing) is primarily a function of the organic and hydraulic loadings on the filter.

TABLE 17.1

TRICKLING FILTER DESIGN CRITERIA

Hydraulic loading (with recirculation)

a. Secondary treatment - 15 to 90 Mgal/acre/d
350 to 2050 $gal/d/ft^2$

b. Roughing - 60 to 200 Mgal/acre/d
1400 to 4600 $gal/d/ft^2$

Recirculation ration - 0.51:1 to 5:1

Dosing interval - Not more than 15 sec (continuous)
Sloughing - continuous

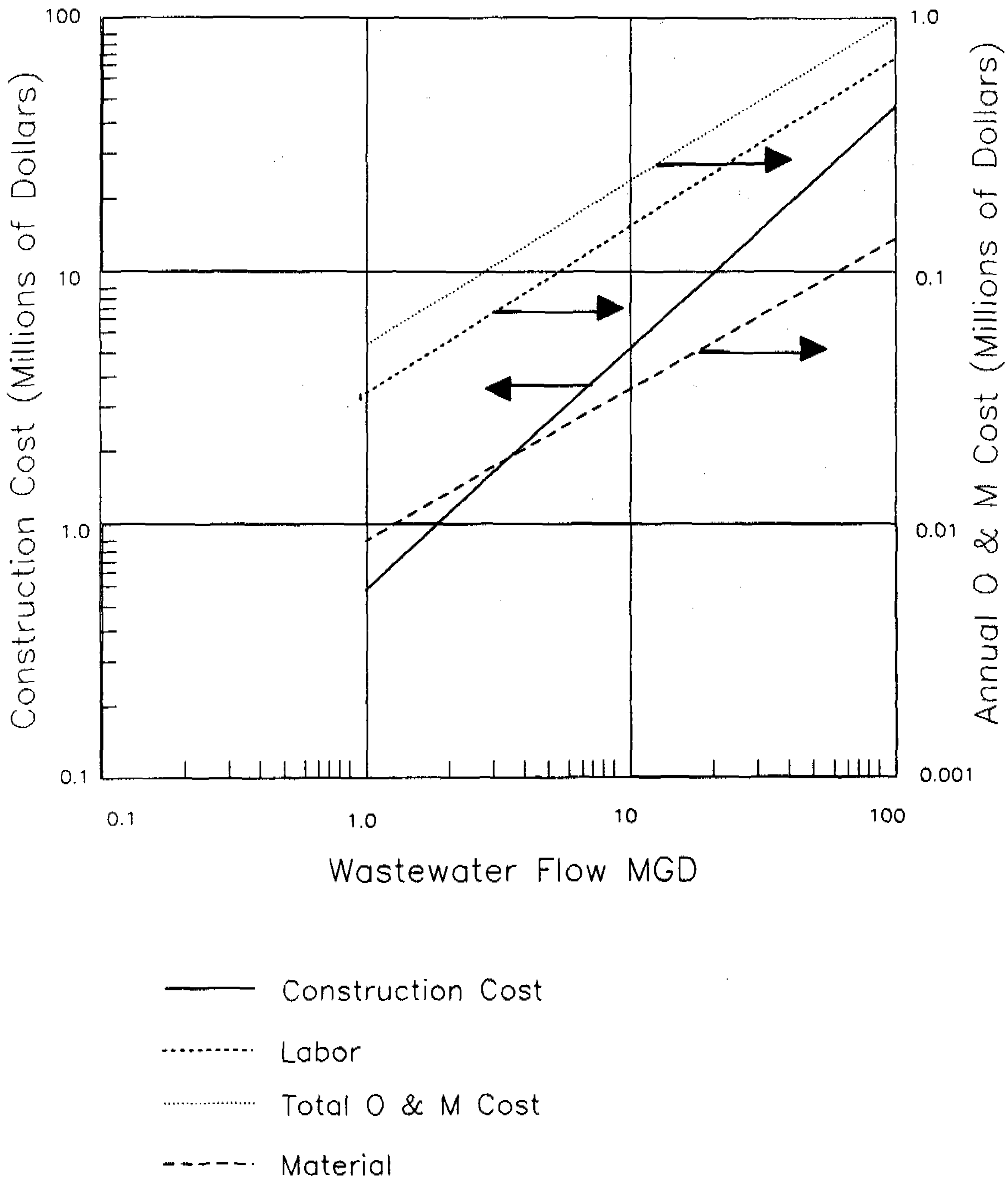
Organic loading

a. Secondary treatment - 450 to 1750 lb BOD_5/d/acre ft
10 to 40 lb BOD_5/d/1000 ft^3

b. Roughing - 4500 to 22,000 lb BOD_5/d/acre ft
100 to 500 lb BOD_5/d/1000 ft^3

Bed Depth - 20 to 30 ft
Power requirements - 10 to 50 hp/Mgal
Underdrain minimum slope = 1%

Figure 17.2: Construction, Operation & Maintenance Costs for Trickling Filter & Plastic Media.



(File: Martin57)

Filter effluent recirculation is important with all media, but especially with plastic media trickling filters to ensure continuous wetting of the media, and to promote effective sloughing control.

17.6 Control

Trickling filters may be expected to have a high degree of reliability if operated properly, waste variability is a minimum, and climate is favorable (wastewater temperatures above 13 degrees C). Mechanical reliability is high and the process is simple to operate.

17.7 Special Factors

Some common modifications of the system include recirculation flow schemes, multistaging, electrically powered distributors, forced ventilation, filter covers and use of various methods of pretreatment and post treatment of wastewater. It can also be used as a roughing filter at flow rates above 1400 gal/d/ft^2 (57.4 m^3/m^2/day) and also as a separate stage nitrification process.

Sludge is withdrawn from the secondary clarifier at a rate of 3000 to 4000 gal/Mgal (m^3/Mm^3) of wastewater, containing 500 to 700 lbs (225 to 315 kgs) of dry solids.

17.8 Recommendations

This technology is best suited for treatment of domestic and compatible industrial wastewaters amenable to aerobic biological treatment. Industrial and joint wastewater treatment facilities may use the process as a roughing filter prior to activated sludge or other unit processes. Existing rock filter facilities can be upgraded via elevation of the containment structure and conversion to plastic media. Finally, it can be used for nitrification following prior (first stage) biological treatment.

18. TRICKLING FILTER, HIGH RATE, ROCK MEDIA

18.1 Description

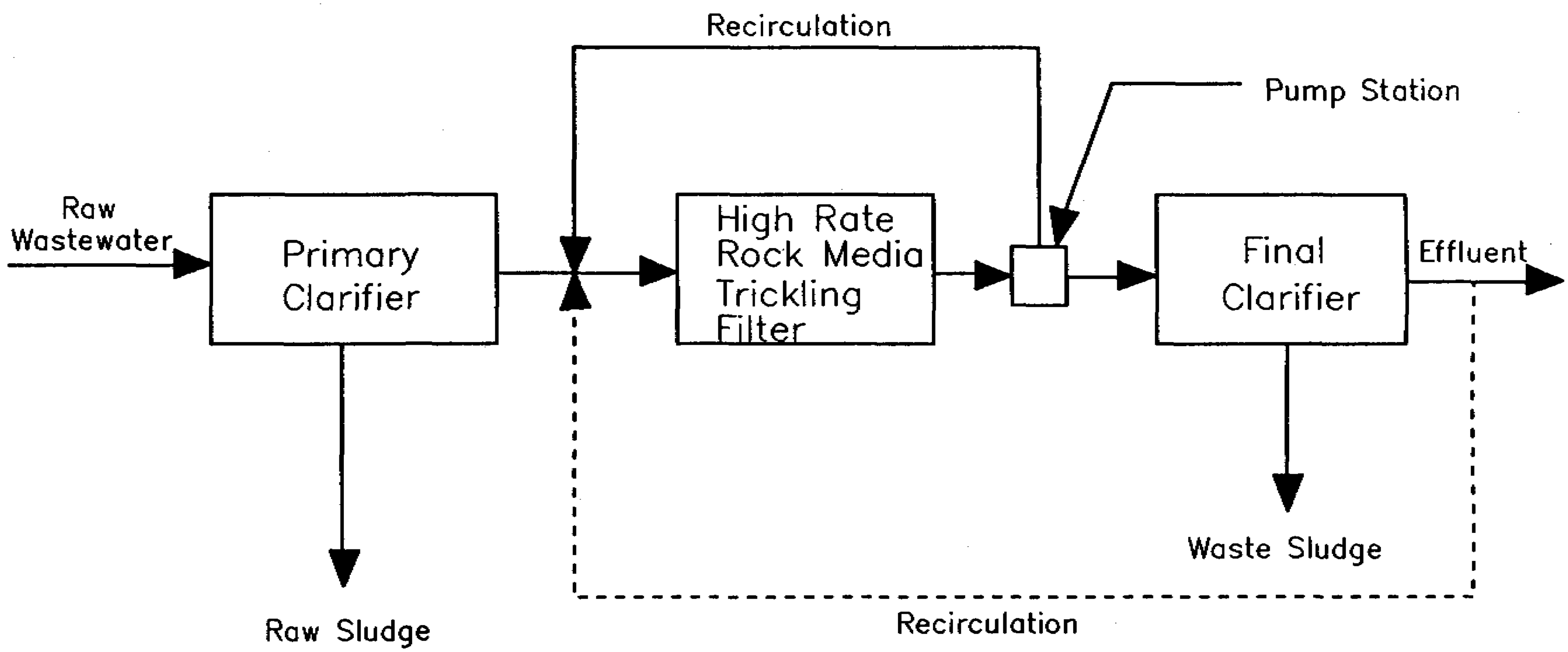
The process consists of a fixed bed of rock media over which wastewater is applied for aerobic biological treatment. Refer to the description in Section 17. Zoogleal slimes form on the media which assimilate and oxidize substances from the wastewater. The bed is dosed by a distributor system, and the treated wastewater is collected by an underdrain system. Primary treatment is normally required beforehand. Post-treatment is often necessary in order that the effluent matches effluent quality from activated sludge, or meets water quality limitations (See Figure 18.1). The primary difference between rock and plastic media systems is that lower density plastic medium requires less support, thus the filter can generally be above ground. The difference between Figures 17.1 and 18.1 is the pump station.

The rotating arms are mounted on a pivot in the center of the filter. Nozzles distribute the wastewater as the arms rotate. Continuous recirculation of filter effluent is used to maintain a constant hydraulic loading to the distributor arms, and subsequently to the filter.

Underdrains are manufactured from specially designed vitrified-clay blocks that support the filter media and pass the treated wastewater to a collection sump for transfer to the final clarifier.

The filter media consists of 1 to 5 inch stone. The high rate trickling filter bed generally is circular in plan, with a depth of 3 to 6 feet. Containment structures are normally made of reinforced concrete and installed in the ground to support the weight of the media.

Figure 18.1: Flow Diagram of Trickling Filter
High Rate, Rock Media



(Source: Ref. 2)

Design Criteria for the high rate systems are given on Table 18.1 (2).

Using a single-stage configuration with filter effluent recirculation and primary and secondary clarification, the removal rates for some pollutants are as follows: BOD5, 60-80%; Phosphorous, 10-30%; NH4-N 20-30%; SS, 60-80%.

18.2 Limitations

See the discussion for Section 17.

18.3 Costs

See Figure 18.2 (2,11).

8.4 Availability

This technology has been in widespread use since 1936 as a modification of the low rate trickling filter process.

18.5 Operation and Maintenance

The organic material present in the wastewater is degraded by a population of microorganisms attached to the filter media. As the microorganisms grow, the thickness of the slime layer increases. As the slime layer increases in thickness, the absorbed organic matter is metabolized before it can reach the microorganisms near the media face. As a result, the microorganisms near the media face enter into an endogenous phase of growth. In this phase, the microorganisms lose their ability to cling to the media surface. The liquid then washes the slime off of the media, and a new slime layer will begin to grow. Sloughing is primarily a function of the organic and hydraulic loadings on the filter. Filter effluent recirculation is vital with high rate trickling filters to promote the flushing action

TABLE 18.1

HIGH RATE TRICKLING FILTER DESIGN CRITERIA

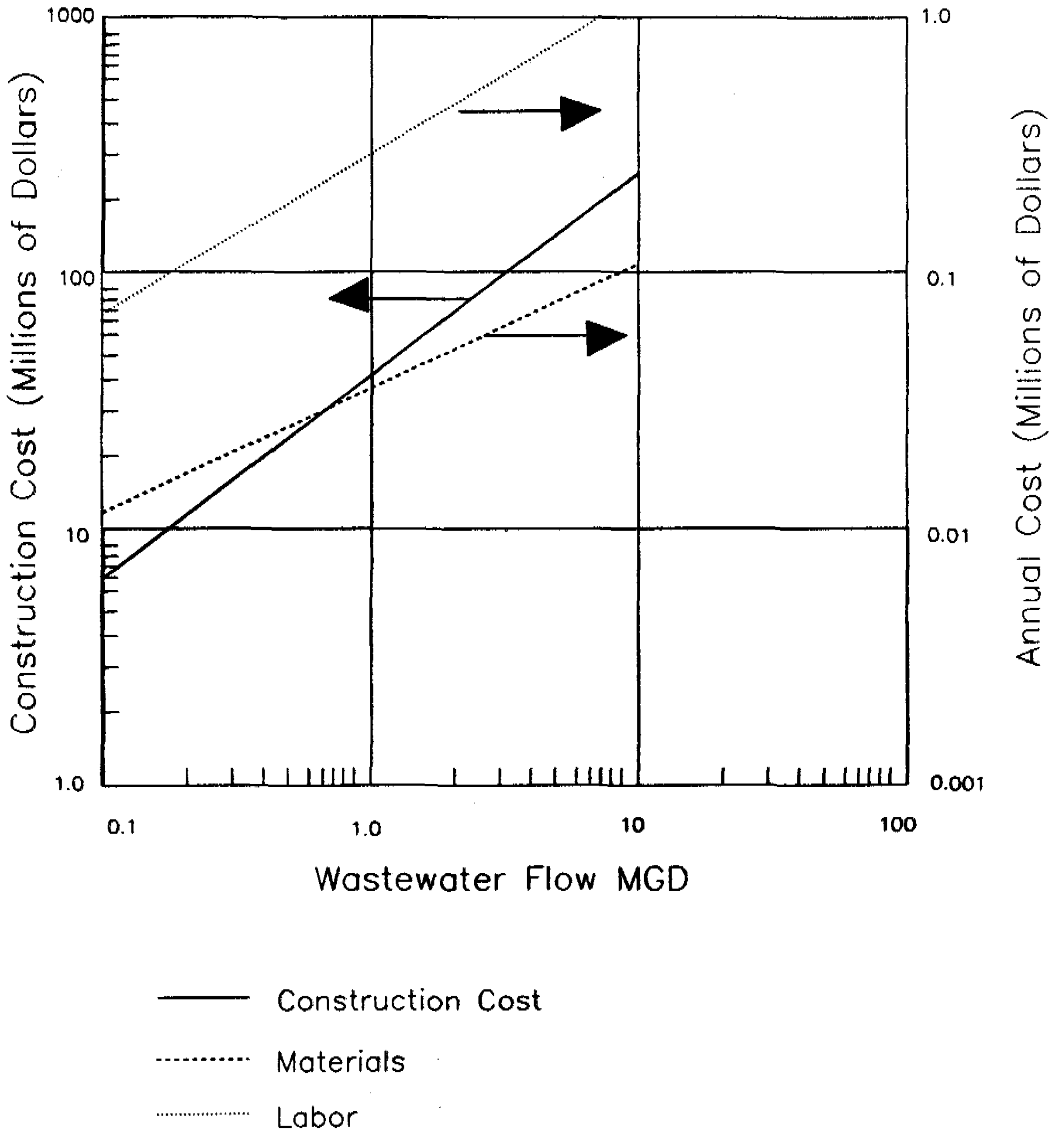
Hydraulic Loading (with recirculation - 10 to 50 Mgal/acre/d
230 to 1150 gal/d/ft^2

Recirculation ration - 0.5:1 to 4:1
Dosing interval - Not more than 15 sec (continuous)
Sloughing - continuous
Rock, 1" to 5", (using square mesh screen. Must meet sodium sulfate soundness test)

Organic Loading - 900 to 2600 lb BOD_5/d/acre ft
20 to 60 lb BOD_5/d/1000 ft^3

Bed Depth - 3 to 6 ft
Power requirements - 10 to 50 hp/Mgal
Undergrain minimum slope = 1 percent Media

Figure 18.2: Construction, Operation & Maintenance Costs for Trickling Filter, High Rate, Rock Media.



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necessary for effective sloughing control, without which media clogging and anaerobic conditions could develop.

18.6 Control

This process can be expected to have a high degree of reliability if operating conditions minimize variability and the installation is in a climate where wastewater temperatures do not fall below 13 degrees C for prolonged periods. Mechanical reliability is high and the process is simple to operate.

18.7 Special Factors

Common modifications can be applied to the system in the form of various recirculation methods, multistaging electrically powered distributors, forced ventilation to enhance oxygen supply to the microorganisms, and filter covers to reduce odor and insect problems.

18.8 Recommendations

This technology is best suited for treatment of domestic and compatible industrial wastewaters amenable to aerobic biological treatment. Industrial and joint wastewater treatment facilities may use the process as a roughing filter prior to activated sludge or other unit processes. The process is effective for the removal of suspended or colloidal materials and is less effective for removal of high concentrations of soluble organics. This high rate option should only be used when plastic media is not available, or in those cases where replacement supply may be uncertain.

Trickling filters have been used effectively as satellite treatment systems within the sewer collection networks, in Guatemala (60). Although this application is with the expectation of further treatment, perhaps at a treatment plant at the end of the collection network, the effluent quality is reported to be suitable for sand filtration.

19. TRICKLING FILTER, LOW RATE, ROCK MEDIA

19.1 Description

The process consists of a fixed bed of rock media over which wastewater is applied for aerobic biological treatment. Zoogleal slimes form on the media which assimilate and oxidize substances in the wastewater. The reader should refer to Sections 17 and 18. The bed is dosed by a distributor system, and the treated wastewater is collected by an underdrain system. Primary treatment is normally required. See Figure 19.1 (2). The schematic should be compared to Figures 17.1 and 18.1.

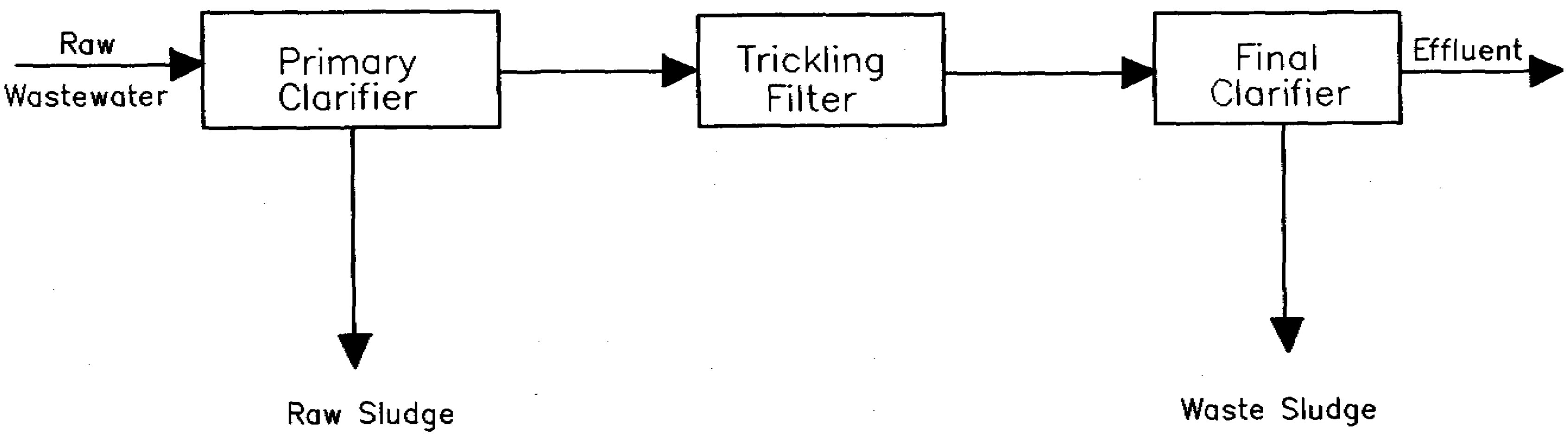
In contrast to the high rate trickling filter which uses continuous recirculation of filter effluent to maintain a constant hydraulic loading to the distributor arms, either a suction-level controlled pump or a dosing siphon is employed for that purpose with a low rate filter. Nevertheless, programmed rest periods may be necessary at times because of inadequate influent flow.

Underdrains are manufactured from vitrified-clay blocks that support the filter media and pass the treated wastewater to a collection sump for transfer to the final clarifier. The filter medium consists of 1 to 5 inch stone. Containment structures are normally made of reinforced concrete and installed in the ground to support the weight of the media.

The low rate media trickling filter bed generally is circular in plan, with a depth of 5 to 10 feet. Although filter effluent recirculation is generally not utilized, it can be provided as a standby tool to keep filter media wet during low flow periods.

Design Criteria are given on Table 19.1 (2, 4). Filter media material properties are given on Table 19.2 (4).

Figure 19.1: Typical Flowsheet for Trickling Filter, Low Rate, Rock Media



(Source: Ref. 2)

TABLE 19.1

TYPICAL DESIGN INFORMATION FOR TRICKLING FILTERS

Item	Low-rate filter	Intermediate-rate filter	High-rate filter	Super-rate (roughing) filter
Hydraulic, loading, $m^3/m^2 \cdot d$	1-4	4-10	10-40	40-200
Organic loading $kg/m^3 \cdot d$	0.08-0.32	0.24-0.48	0.32-1.0	0.80-6.0
Depth, m	1.5-3.0	1.25-2.5	1.0-2.0	4.5-12
Recirculation ratio	0	0-1	1-3; 2-1	1-4
Filter media	Rock, slag, etc.	Rock, slag, etc.	Rock, slag, synthetic materials	Synthetic materials redwood
Power requirements, $kW/10^3$	2-4	2-8	6-10	10-20
Filter flies	Many	Intermediate	Few, larvae are washed away	Few or more
Sloughing	Intermittent	Intermittent	Continuous	Continuous
Dosing intervals	Not more than 5 min. (generally intermittent)	15 to 60 s (continuous)	Not more than 15 s (continuous)	Continuous
Effluent	Usually fully nitrified	Partially nitrified	Nitrified at low loadings	Nitrified at low loadings

Note: m x 3.2808 = ft
$m^3/m^2 \cdot d \times 1.0691$ = mgal/acre · d
$m^3/m^2 \cdot d \times 0.0170 = gal/ft^2 \cdot min$
$kg/m^3 \cdot d \times 62.4280 = lb/10^3\ ft^3 \cdot d$
kW x 1.3410 = hp

Source: Reference 4

TABLE 19.2

PHYSICAL PROPERTIES OF TRICKLING-FILTER MEDIA

Medium	Normal size, mm	Mass/unit volume, kg/m^3	Specific surface area, $m/^2/m^3$	Void space percent
River rock				
Small	25-65	1250-1450	55-70	40-50
Large	100-120	800-1000	40-50	50-60
Blast-furnace slag				
Small	50-80	900-1200	55-70	40-50
Large	75-125	800-1000	45-60	50-60
Plastic				
Conventional	600 x 600 x 1200	30-100	80-100	94-97
High-specific surface	600 x 600 x 1200	30-100	100-200	94-97
Redwood	1200 x 1200 x 500	150-175	40-50	70-80

Note: mm x 0.03937 = in.
kg/m^3 x 62.4280 = $lb/10^3\ ft^3$
m^2/m^3 x 0.3048 = ft^2/ft^3

Source: Reference 4

For a low rate trickling filter with a single-stage configuration, primary and secondary clarification and no recirculation, the expected pollutant removal rates are as follows: BOD_5, 75-90%; Phosphorous, 10-30%; NH4-N, 20-40%; SS, 75-90%.

19.2 Limitations

Filter flies and odors are common in low rate systems; periods of inadequate moisture for slimes can be common; less effective in treatment of wastewater containing high concentrations of soluble organics; limited flexibility and process control in comparison with competing processes; and higher land and capital cost requirements than say activated sludge for similar capability and effectiveness.

19.3 Costs

See Figure 19.2 (2, 11).

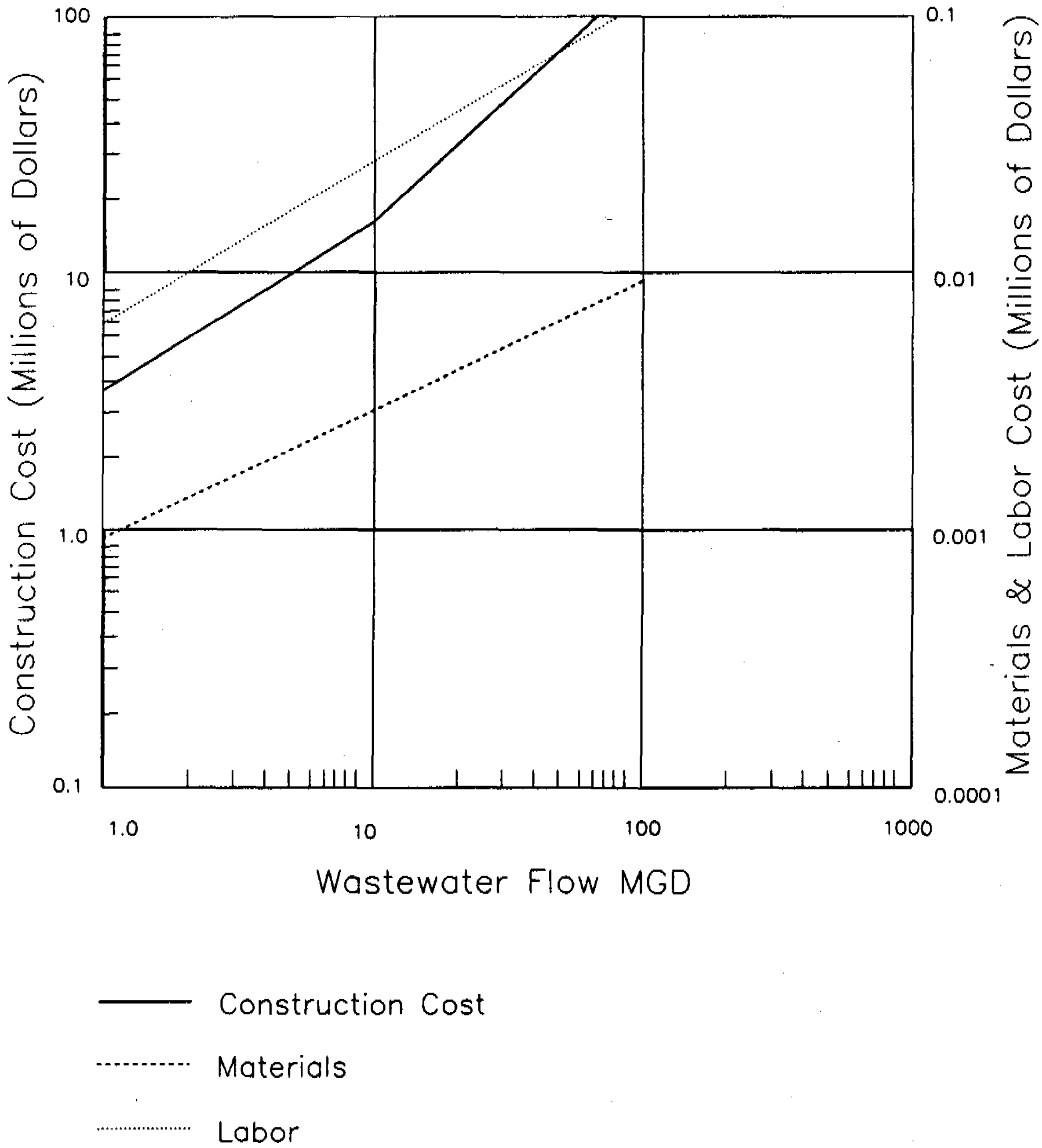
19.4 Availability

This process works well in moderate climates and is in widespread use. Use of post-treatment and multistaging has frequently been found necessary to insure uniform compliance with effluent limitations. It is presently being superseded more and more by plastic media systems.

19.5 Operation and Maintenance

The organic material present in the wastewater is degraded by a population of microorganisms attached to the filter media, as described in Section 19.1, and in Sections 17 and 18. The organism systems must be preserved by dosing at regular intervals, especially in intermittent flow situations. When the

Figure 19.2: Construction, Operation & Maintenance Costs for Trickling Filter, Low Rate, Rock Media.



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dosing period is longer than about 2 hours, the efficiency of the process deteriorates because the slime becomes dry. When sloughing has occurred, attention must be paid by the operator until regrowth has occurred.

Typically, only the top 2 to 4 ft (0.6 to 1.2 m) of the filter medium will have appreciable slime growth. The lower portions of the medium may be populated largely by nitrifying bacteria, oxidizing ammonia nitrogen to nitrite and nitrate forms. Thus, it is possible to produce both a well nitrified, and well treated effluent from the point of view of BOD removal.

19.6 Control

This process is highly reliable under conditions of a moderate climate and process operation requires little skill.

19.7 Special Factors

Some common modifications of the system include recirculation flow schemes, multistaging, electrically powered distributors, forced ventilation, filter covers and use of various methods of pretreatment and posttreatment of wastewater.

High head losses - 5 to 10 ft (1.5 to 3 m) may be possible.

19.8 Recommendations

This technology is best used for treatment of domestic and compatible industrial wastewaters amenable to aerobic biological treatment. Industrial and joint wastewater treatment facilities may use the process as a roughing filter prior to activated sludge or other unit processes. Existing rock filter facilities can be upgraded via elevation of the containment structure and conversion to plastic media. Finally, it can be used for nitrification following prior (first stage) biological treatment.

20. AERATED/AEROBIC LAGOONS

20.1 Description

This section is organized on the premise that aerobic lagoons operate naturally, while aerated lagoons are operated mechanically.

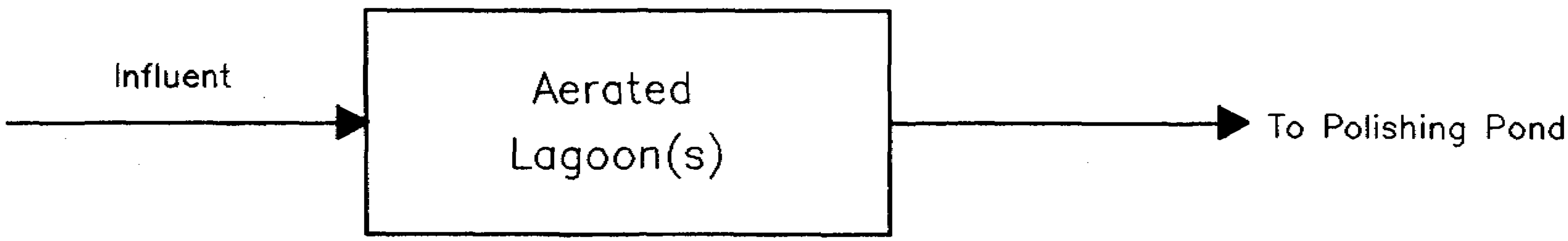
Aerated

Aerated lagoons are medium-depth basins designed for biological treatment of wastewater on a continuous basis. In contrast to stabilization ponds, which obtain oxygen from photosynthesis and surface reaeration, lagoons typically employ aeration devices which supply supplemental oxygen to the system (see Figure 20.1) (2). Aerated lagoons evolved from stabilization ponds when aeration devices were added to counteract odors arising from anaerobic conditions. The aeration devices may be mechanical (i.e., surface aerator), or diffused air systems. Surface aerators are divided into two types: cage aerators and the more common turbine and vertical shaft aerators. The many diffused air systems utilized in lagoons consist of plastic pipes supported near the bottom of the cells with regularly spaced sparger holes drilled in the tops of the pipes. Because aerated lagoons are normally designed to achieve partial mixing only, aerobic-anaerobic stratification will occur, and a large fraction of incoming solids and a large fraction of the biological solids produced from waste conversion settle to the bottom of the lagoon cells.

Removal rates for certain pollutants are as follows: BOD, for an influent concentration of 200-500 mg/1, 60-90%; COD, 70-90%; TSS, for an influent concentration of 200-500 mg/1, 70-90%.

Design Criteria: one or more aerated cells, followed by a settling (typically unaerated) cell. Detention time: 3 to 10

Figure 20.1: Typical Flow Diagram of Aerated Lagoons



(Source: Ref. 2)

days. Depth: 6 to 20 ft (1.8 to 6.1 m). pH: 6.5 to 8.0. Water temperature range: 0 to 40 degrees C. Optimum water temperature: 20 degrees C. Oxygen requirement: 0.7 to 1.4 times the amount of BOD5 removed. Organic loading: 10 to 300 lb BOD5/acre/d (approx. 10 to 300 kg/ha/day). Table 20.1 (4), presents design criteria for lagoons and aerated stabilization ponds. The distinction among the various types of lagoons and ponds is not clear in many cases. This is especially true since the systems are often operated differently than the original design intended.

Energy requirements are as follows - For aeration: 6 to 10 hp/million gals capacity (2 to 1.2 W/m3). To maintain all solids in suspension: 60 to 100 hp/million gals capacity (12 to 20 W/m3). To maintain some solids in suspension: 30 to 40 hp/million gals capacity (6 to 8 W/m3).

Aerobic

The aerobic stabilization ponds are typically large, shallow earthen basins that are used for the treatment of wastewater by natural processes involving the use of both algae and bacteria. These ponds also called high rate aerobic ponds, or maturation ponds when dissolved oxygen concentration levels are maintained throughout their entire depth. They are usually 12 to 18 inches deep, allowing light to penetrate the full depth. Artificially aerated ponds may be deeper. The mixing is primarily provided by the photosynthesis process of algae. Oxygen is provided by photosynthesis and surface reaeration and aerobic bacteria stabilize the wastewater.

There are two basic types of aerobic ponds. The first type, is a shallow with a limited depth 6 to 18 inches contains high population of algae. The second type, is a 5 feet deep pond consisting of large amounts of bacteria in the water. To achieve best results with aerobic ponds, both types of ponds are mixed

TABLE 20.1

TYPICAL DESIGN PARAMETERS FOR AEROBIC STABILIZATION PONDS AND LAGOONS

Parameter	Aerobic (high-rate) pond	Aerobic pond[a]	Aerobic (maturation) pond	Aerated lagoons
Flow regime	Intermittently mixed	Intermittently mixed	Intermittently mixed	Completely mixed
Pond size, ha	0.25-1	<4 multiples	1-4	1-4 multiples
Operation[b]	Series	Series or parallel	Series or parallel	Series or parallel
Detention time, d[b]	4-6	10-40	5-20	3-10
Depth, m	0.30-0.45	1-1.5	1-1.5	2-6
pH	6.5-10.5	6.5-10.5	6.5-10.5	6.5-8.0
Temperature range, C	5-30	0-30	0-30	0-30
Optimum temperature, °C	20	20	20	20
BOD_5 loading, kg/had[c]	80-160	40-120	≤15	
BOD_5 conversion	80-95	80-95	60-80	80-95
Principal conversion products	Algae, CO_2, bacterial cell tissue	Algae, CO_2, bacterial cell tissue	Algae, CO_2, bacterial cell tissue, NO_3	CO_2, bacterial cell tissue
Algal concentration, mg/L	100-260	40-100	5-10	
Effluent suspended solids, mg/L[d]	150-300	80-140	10-30	80-250

[a] Conventional aerobic ponds designed to maximize the amount of oxygen produced rather than the amount of algae produced.

[b] Depends on climatic conditions.

[c] Typical values (much higher values have been applied at various locations). Loading values are often specified by state control agencies.

[d] Includes algae, microorganisms, and residual influent suspended solids. Values are based on an influent soluble BOD_5 of 200 mg/L and, with the exception of the aerobic ponds, an influent suspended-solids concentration of 200 mg/L.

Note:
ha x 2.4711 = acre
m x 3.2808 = ft
kg/hd · d x 0.8922 = lb/acre · d

periodically using pumps or surface aerators. The oxygen released by the algae through the process of photosynthesis is used by the bacteria in the aerobic degradation of organic matter. The nutrients and carbon dioxide released in this degradation are used by the algae. The typical design parameters of aerobic ponds are shown in Table 20.2 (7, 15, 24).

There is considerable variation in design approaches to ponds and lagoons. This is probably because the actual operating conditions lay somewhere among aerobic/aerated, anaerobic and facultative. The various design approaches are based on studies during which, the actual state of dissolved oxygen is uncertain. At best, the dissolved oxygen was undoubtedly different at different points within the same pond. The design for any technology should be based on specific site related studies, but especially for ponds and lagoons. Then the operating conditions can be fixed given the waste quantity and quality characteristics. Table 20.3 (64) is shown to illustrate the results of several design approaches for facultative ponds. In warm climates. Loading rates may be higher. Potential cost savings from smaller sizes alone will pay for pilot plant studies. The loading rates on Table 20.3 should be compared to those on Table 30.2.

20.2 Limitations

Aerated

Lagoon systems are simpler to operate and maintain than trickling or activated sludge biological systems, but are less controllable. Thus, it is more difficult to treat highly variable waste streams. Toxicity impacts may affect the entire lagoon complex and require long periods of readjustment.

The algae group or bacterial species present in any section of aerobic ponds depends on several factors such as organic

TABLE 20.2

TYPICAL DESIGN PARAMETERS FOR AEROBIC, ANAEROBIC, AND FACULATIVE STABILIZATION PONDS

Parameter	Ponds/Lagoons Aerobic	Anaerobic	Faculative
Flow	Completely mixed	--	mixed surface layer
Size (acre)	2.5 to 10	0.5 to 2.5	2.5 to 10
Operation	Series	Series	Series
Detention Time (day)	3 to 10	20 to 50	7.20
Depth (ft)	10 to 33	8 to 16	3 to 8
pH	6.5 to 8	6.8 to 7.2	6.5 to 8.5
Temperature range (°F)	32 to 86	43 to 122	32 to 122
Optimum temperature (°F)	68	86	68
BOD Loading Rate lb/acre/day	10 to 300	178 to 446	45 to 178
Algal concentration (mg/l)	--	0 to 5	5 to 20
Effluent suspend solids (mg/l)	80 to 250	80 to 160	40 to 60
Principal conversion products	co_2 bacterial, cell time	CO_2, CH_4, bacterial	Algae, CO_2, CH_4
BOD_5 conversion	80 to 95	50 to 85	80 to 95

Source: Reference 7

TABLE 20.3

SUMMARY OF RESULTS FROM VARIOUS DESIGN METHODS FOR FACULTATIVE PONDS

DESIGN METHOD	DETENTION TIME, d		VOLUME, m^3		SURFACE AREA, ha		Depth m	No. Cells in Series	SURFACE LOADING RATE kg BOD_5/ha/d	
	Primary Pond	Total Systems	Primary Pond	Total System	Primary Pond	Total System			Primary Pond	Total System
AREAL LOADING RATE	66[a]	180	125,600[a]	386,800	9.5	22.3	2 (1.4)[c]	4	40	17
GLOYNA	--	140	125,600[a]	265,000	--	26.5	2 (1)[c]	-	-	14
MARAIS & SHAW	37[b]	74	69,300[b]	138,600	2.8	5.6	2.4	2	135	68
PLUG FLOW	66[a]	180	125,600[a]	386,800	9.5	22.3	2 (1.4)[c]	4	40	25
WEHNER & WILHELM	66[a]	80-132	125,600[a]	151,400 - 249,900	9.5	10.8 - 17.9	2 (1.4)[c]	4	-	30-48

[a] Controlled by state standards and is equal to value calculated for an areal loading rate of 40 kg/ha/d and an effective depth of 1.4 m.

[b] Also would be controlled by state standards for areal loading rate; however, the method includes a provision for calculating a value and this calculated value is shown.

[c] Effective depth.

Source: Reference 64.

loading, degree of pond mixing, nutrients, sunlight, pH and temperature. Temperature is the key limiting factor to high loading rates for aerobic biological treatment processes.

20.3 Costs

See Figures 20.2, and 20.3 (2, 11).

O & M costs for aerobic lagoons should be lower than the costs, both construction and O & M, for aerated lagoons, because of the absense of mechanical aerators. On the other hand, aerated lagoons may be smaller in size because of improved oxygen transfer.

20.4 Availability

While not widely used when compared with the large number of stabilization ponds in common use throughout Latin America, the Caribbean, and the U.S., it has been fully demonstrated, and used for years.

Ponds are the most economical method of sewage treatment wherever land is available at relatively low cost. Aerobic ponds have found only limited application due to higher cost and somewhat more complex operation than for facultative or anaerobic ponds.

20.5 Operation and Maintenance

Aerated

As the solids begin to build up, a portion will undergo aerobic decomposition. Some volatile toxics can potentially be removed by the aeration process (more recent work shows that volatile organics may be adsorbed onto the biomass and later degraded or become part of the sludge solids), and incidental

Figure 20.2 : Construction Cost of Aerated Lagoons.

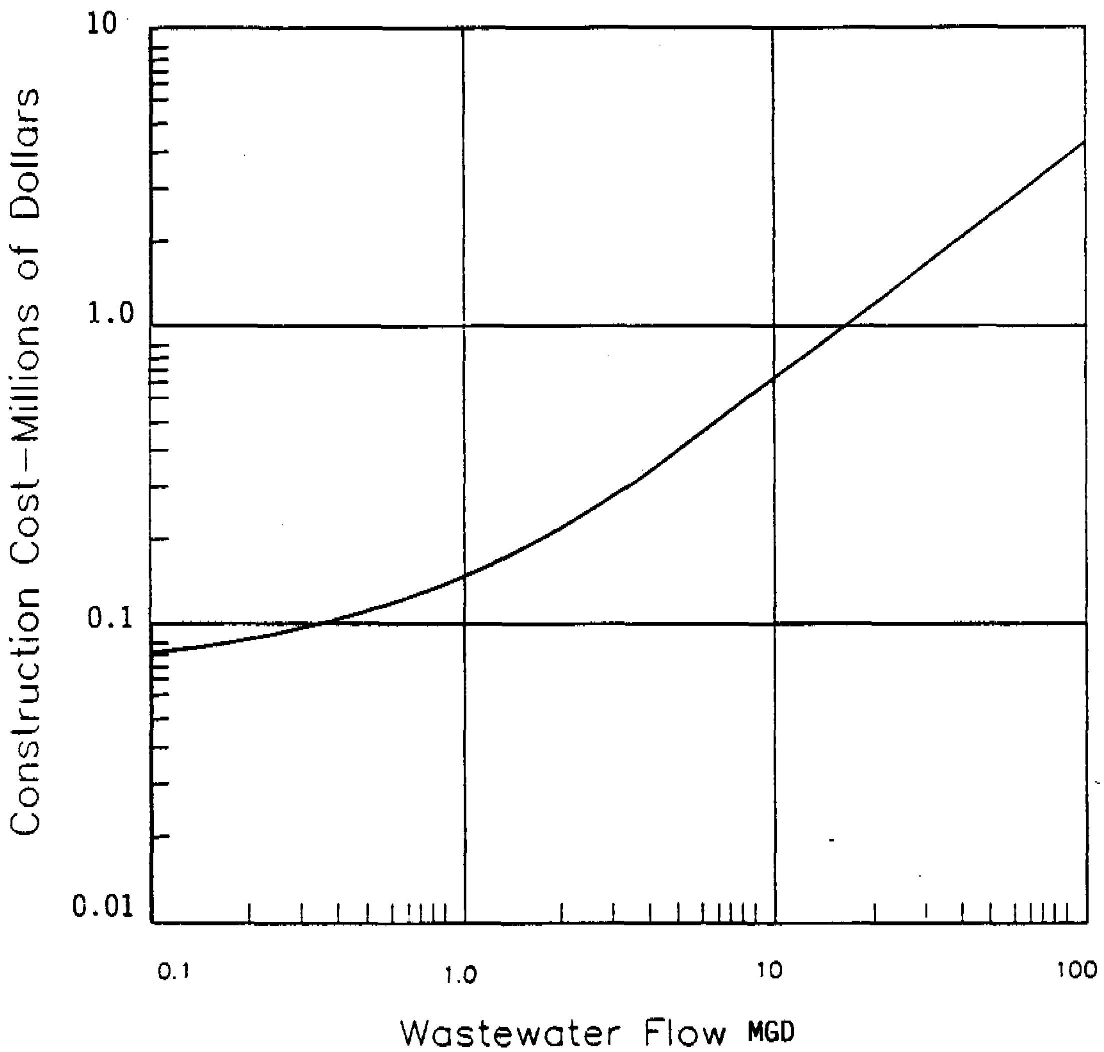
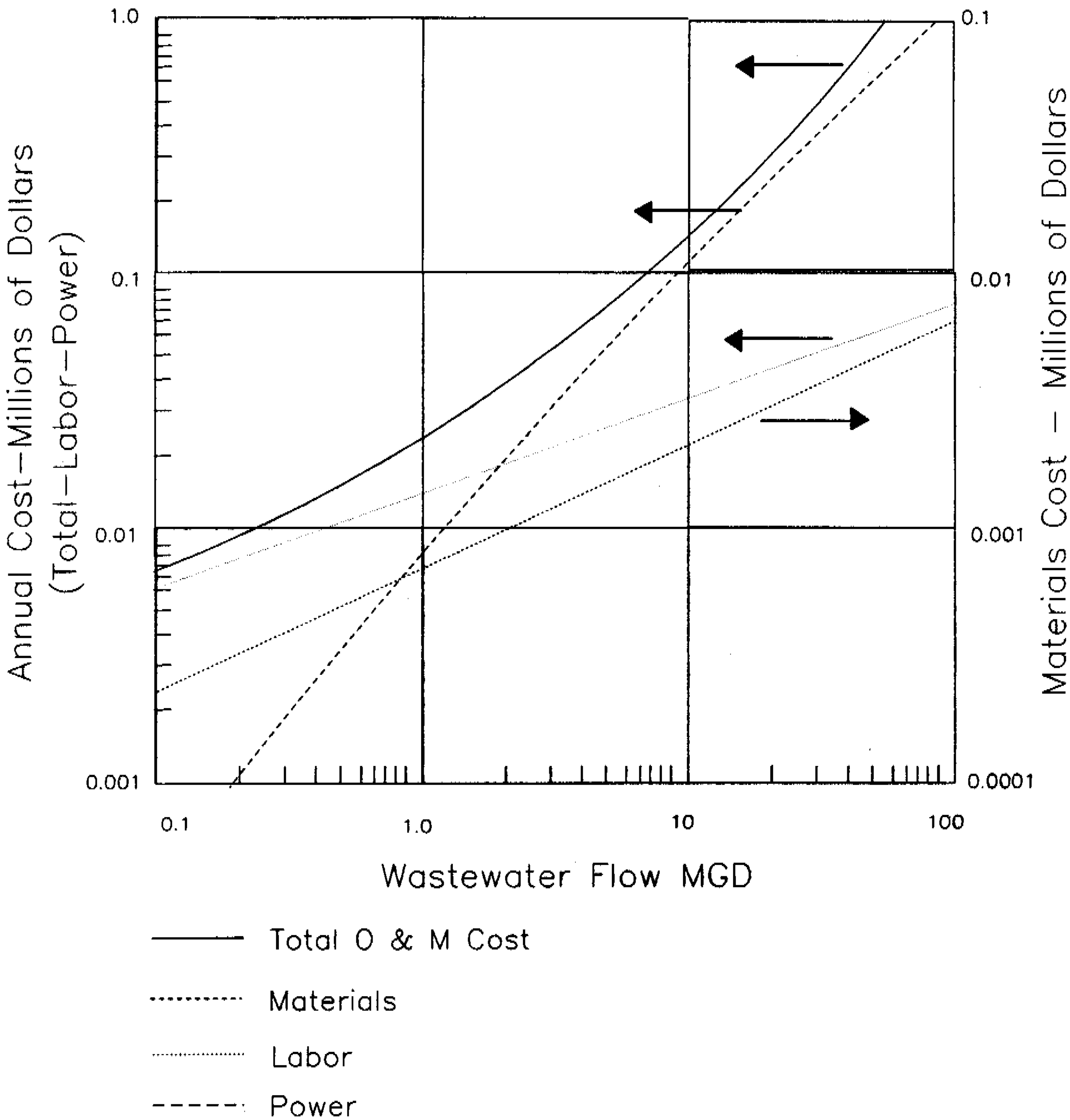


Figure 20.3: Operation & Maintenance Cost of Aerated Lagoons.



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removal of other toxics can be expected to be similar to an activated sludge system. If volatile materials escape, they can become an air pollution problem. Several smaller aerated lagoon cells in series are more effective than one large cell. Tapering aeration intensity downward in the direction of flow (through successive cells) promotes settling out of solids in the last cell. A non-aerated polishing cell following the last aerated cell is an optional, but recommended, design technique to enhance suspended solids removal prior to discharge.

Aerobic ponds are large, shallow earthen basins in which organic wastes are decomposed by algae and bacteria in a combination of cyclic-symbiotic relationship. The principal advantage of this process is that it removes excreted pathogens at a much lower cost than any other form of treatment. Ponds should be cleaned and maintained regularly. It is particularly important to remove grass and other plant growth from the periphery and in the pond itself. Some pond installations in Latin America have been observed to have such significant growth that capacity was severely reduced. High solids wastes should be pretreated before pond treatment. Solids accumulation in ponds can significantly reduce capacity and thus treatment efficiency will be lowered. Floating scum mats should be removed or oxygen transfer is impaired.

Abnormal operation occurs when lagoons are overloaded because the BOD loads are too high. Excessive BODs can occur when influent loads exceed design capacity due to population increases, industrial growth, or industrial dumps. Under these conditions new facilities must be constructed or the BOD loading must be reduced at the source, such as an industrial dump.

Another type of overloading can occur when too much flow is diverted to one lagoon. This can happen when an operator accidentally feeds one lagoon more than the other or when a pipe opening is blocked by rags, solids or grit due to low pipe velocities, and thus too much flow is diverted to another lagoon.

Once this happens and the overloaded lagoon starts producing odors, it must be taken out of service and the flow diverted to the other lagoons until the overloaded lagoon recovers. Hopefully the lagoons in service will not become overloaded. Also be sure to remove any rags, solids or grit which caused the overloading and inspect the other pipes to prevent this problem from happening again in the other lagoons.

Usually lagoons do not become overloaded during storms and periods of high runoff because there is not a significant increase in the BOD loading on the lagoons.

Large amounts of brown or black scum on the surface of a pond is an indication that the pond is overloaded. Scum on the surface of a pond often leads to odor problems. The best way to control scum is to take corrective action as soon as possible.

During winter conditions the pond can become covered with ice and snow. Sunlight is no longer available to the algae and oxygen cannot enter the water from the atmosphere. Without dissolved oxygen available for aerobic decomposition, anaerobic decomposition of the solids occurs. Anaerobic decomposition takes place slowly because of the low temperatures. By keeping the pond surface at a high level, a longer detention time and less heat loss will be obtained. During the period of ice cover, odorous gases formed by anaerobic decomposition accumulate under the ice and are dissolved into the wastewater being treated.

Some odors may be observed in the spring just after the ice cover breaks up because the lagoon is still in an anaerobic state and some of these dissolved gases are being released. Melting of ice in the spring provides dilution water with a high oxygen content, thus the lagoons usually become facultative in a few days after breakup of the ice if they are not organically (BOD) overloaded.

20.6 Control

Aerated

The service life of a lagoon is estimated at 30 years or more. The reliability of equipment and the process is high. Little operator expertise is required, but more than a simple pond. Aeration equipment requires regular maintenance, cleaning and attention.

Aerobic ponds are designed for high BOD removal (95 percent), and may be used as post-treatment for facultative and anaerobic ponds. They may also be used for destruction and removal of highly concentrated organic matter (especially industrial wastes) and pathogens. Consideration should always be given to designing and operating ponds in series configurations for added control, and also to combining aerobic with anaerobic and facultative ponds.

Aerated/aerobic pond operation in somewhat more tricky and complex to operate than facultative and anaerobic ponds, because the effluent will contain high suspended solids and algae. Also the relatively shorter detention time may result in higher amounts of coliform in the effluent. Because of their shallow depth, covering/paving the bottom may be required to prevent weed growth.

20.7 Special Factors

Both aerated and aerobic lagoons may be lined with concrete or an impervious flexible lining, depending on soil conditions and environmental regulations. When high-intensity aeration produces completely mixed (all aerobic) conditions, a final settling tank is required. Solids are recycled to maintain about 800 mg/l MLVSS (mixed liquor volatile suspended solids) in this mode.

There is an opportunity for volatile organic material and pathogens in aerated lagoons to enter the air with any aerated wastewater treatment process. This opportunity depends on air/water contact afforded by the aeration system. There is a potential for seepage of wastewater into ground water unless a lagoon is lined. Compared to other secondary treatment processes, aerated lagoons generate less solid residue for immediate disposal, since solids retention times are very long and degradation is more complete.

20.8 Recommendations

These processes are used for domestic and industrial wastewater of low and medium organic strength. They are commonly used when land is inexpensive and costs and operational control are to be minimized. It is relatively simple to upgrade existing oxidation ponds, lagoons and natural bodies of water to this type of treatment. Aeration increases the oxidation capacity of the pond and is useful in overloaded anaerobic and/or facultative ponds that generate odors. It is also useful when supplemental oxygen requirements are high or when the wastewater treatment requirements are either seasonal or intermittent, such as for canning or other food industries.

21. SLUDGE DRYING BEDS

21.1 Description

Drying beds are used to dewater sludge both by drainage through the sludge mass to underlying soil, and by evaporation from the surface exposed to the air. Collected filtrate may be returned to the treatment plant, as shown in Figure 21.1. Drying beds usually consist of 4 to 9 inches (10.2 to 23 cm) of sand which is placed over 8 to 18 inches (20 to 46 cm) of graded gravel or stone. The sand typically has an effective size of 0.3 to 1.2 mm and a uniformity coefficient of less than 5.9. Gravel is normally graded from 1/8 to 1.0 inch (3 to 25.4 mm). Drying beds have underdrains that are spaced from 8 to 20 ft (2.44 to 6.1 m) apart. Underdrain piping is often vitrified clay laid with open joints, has a minimum diameter of 4 inches (10.1 cm), and a minimum slope of about 1 percent.

This process achieves a cake of 40 to 45% solids in 2 to 6 weeks in good weather and with a well digested waste activated, primary or mixed sludge. Codisposal of wastewater treatment sludge with water treatment plants sludges has been suggested (48) and should be practiced more often. With chemical conditioning dewatering time may be reduced by 50% or more. Chemical conditioning can be accomplished with water treatment plant sludges. Also, sludges from wastewater treatment and water treatment possess considerable soil conditioning properties (see Section 22). Solids content of between 85 and 90% have been achieved on sand beds, but normally the time required to achieve such high concentrations are impractical.

Design Criteria - Open bed are = 1.0 to 1.5 ft2/capita (0.093 to 0.135 m2/capita) (primary digested sludge); 1.75 to 2.5 ft2/capita - 0.163 to 0.23 m2/cap (primary and activated sludge); 2.0 to 2.5 ft2/capita - 0.19 to 0.23 m2/cap (alum or iron precipitated sludge). Experience has shown that enclosed

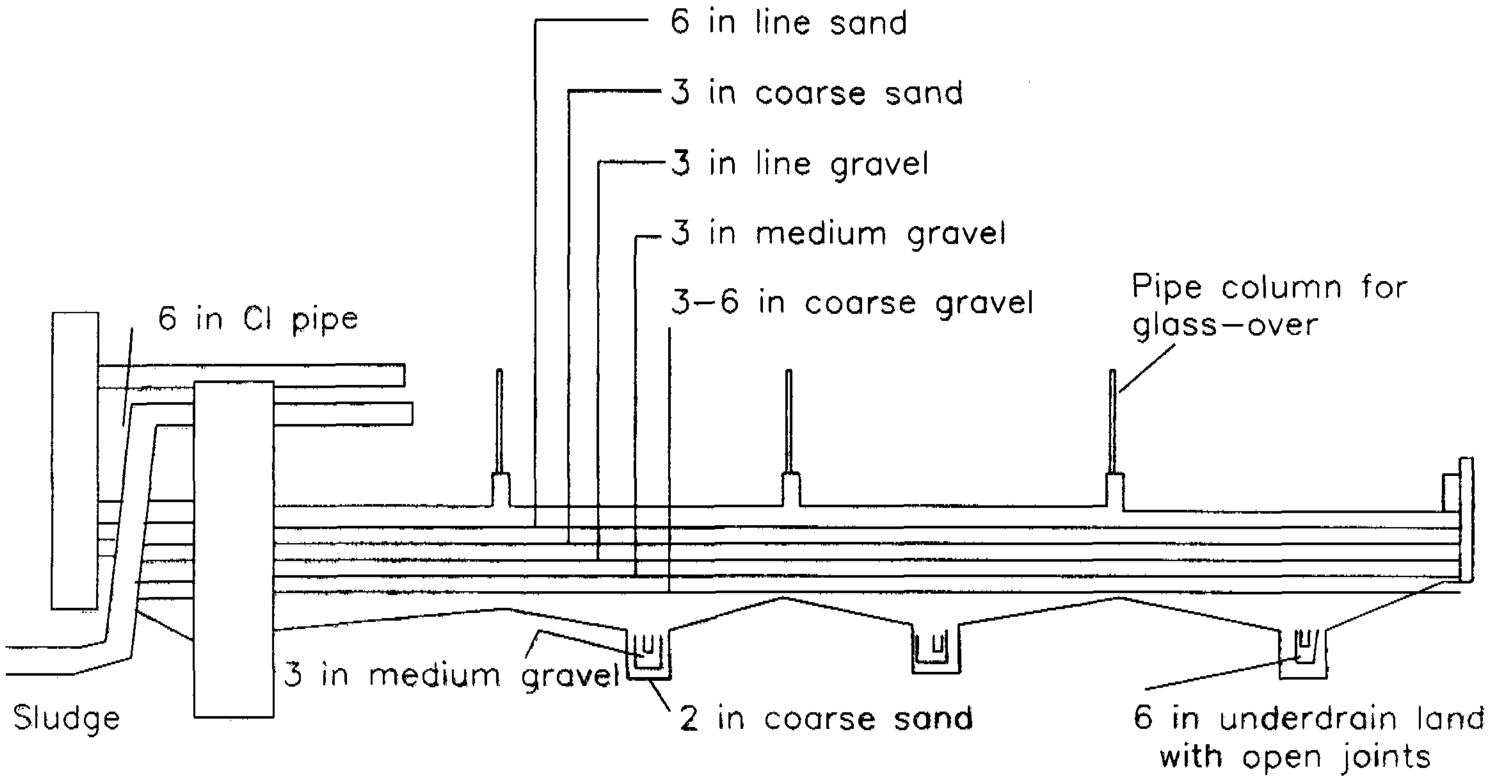


Figure 21.1: Diagram of Sludge Drying Beds

(Source: Ref. 2) (File:Martin49)

beds 60 to 75% of the open bed area. Solids loading rates vary from 10 to 28 lb/ft2/yr (49 to 137 kg/m2/yr) for open beds and 12 to 40 lb/ft2/yr (59 to 196 kg/m2/yr) for closed beds. Sludge beds should be located at least 200 ft (61 m) from dwellings to avoid odor complaints due to poorly digested sludges. There little or no data available for codisposal, or for drying water treatment plant sludges on beds.

Sludge is placed on the bed in an 8 to 12 inch (20 to 30 cm) layer. The drying area is partitioned into individual beds, approximately 20 ft (6.1 m) wide by 20 to 100 ft (6.1 to 30 m) long, of a convenient size so that one or two beds will be filled with a normal withdrawal of sludge from digesters, Imhoff tanks, or other treatment systems. The interior partitions commonly consist of two or three creosoted planks, one on top of the other, to a height of 15 to 18 inches (38 to 46 cm). These may stretch between slots in precast concrete posts, or any convenient construction may be used. The outer boundaries of the bed(s) may be of similar construction or earthen embankments for open beds. But concrete foundation walls are typically required if the beds are to be covered. The covers are supported by the walls.

Piping to the sludge beds is generally made of cast iron and designed for a minimum velocity of 2.5 ft/sec (0.75 m/sec). It is arranged to drain into the beds and provisions are made to flush the lines and to prevent freezing in cold climates. Distribution boxes are provided to divert sludge flow to the selected bed, splash plates are used at the sludge inlets to distribute the sludge over the bed and to prevent erosion of the sand.

21.2 Limitations

Air drying is usually restricted to well digested or stabilized sludge wastewater sludges, because raw sludge is odorous, attracts insects, and does not dry satisfactorily when applied at reasonable depths. Oil and grease clog sandbed pores and thereby seriously retard drainage. The design and use of drying beds are affected by weather conditions, sludge characteristics (especially dewatering characteristics), land values and proximity of residences. Operation is severely restricted during periods of prolonged freezing and rain. Land requirements are also large.

21.3 Costs

See Figure 21.2 (2, 11).

21.4 Availability

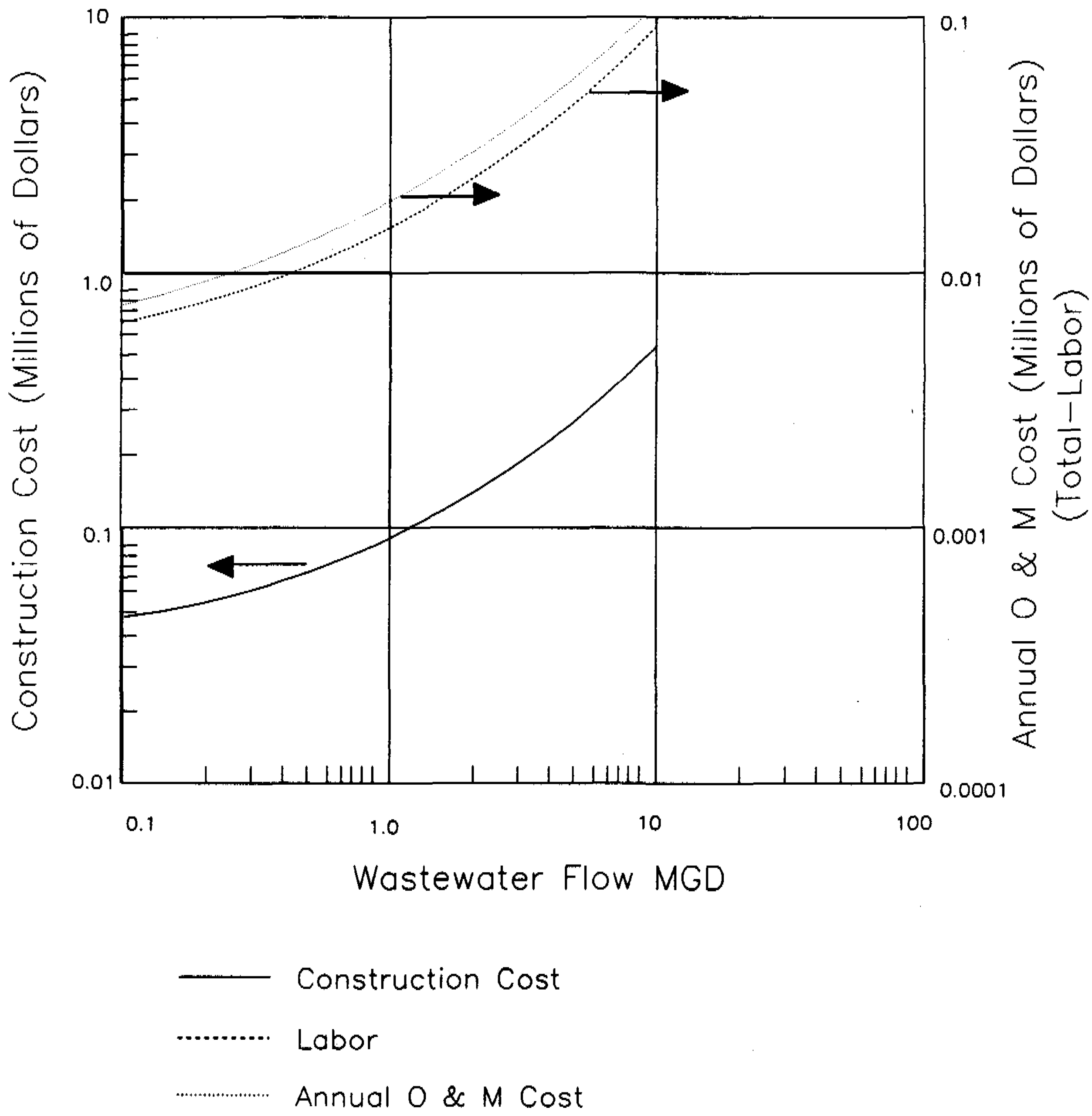
Over 6,000 plants in the U.S. use open or covered sandbeds. Brick underlain beds are used in Uruguay for drying Imhoff tank sludges. Sand, gravel and bricks are definitely available locally almost everywhere.

21.5 Operation and Maintenance

Sludge can be removed from the drying bed after it has drained and dried sufficiently to be spadable. Sludge removal is accomplished by manual shoveling into wheelbarrows or trucks or by a scraper or front-end loader. Provisions should be made for driving a truck onto or alongside the bed to facilitate loading. Mechanical devices can remove sludges of 20 to 30% solids while cakes of 30 to 40% generally require hand removal.

Paved drying beds with limited drainage systems permit the use of mechanical equipment for cleaning. Field experience indicates that the use of paved drying beds results in shorter drying times as well as more economical operation when compared with conventional sandbeds because, as indicated above, the use of mechanical equipment for cleaning permits the removal of

Figure 21.2: Construction, Operation & Maintenance Costs for Sludge Drying Beds.



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sludge with a higher moisture content than in the case of hand cleaning. Paved beds have worked successfully with anaerobically digested sludges but are less desirable than sandbeds for aerobically digested activated sludge.

21.6 Control

Sandbeds are generally used to dewater sludges in small plants. They require little operator attention or skill. Drying beds should be cleaned regularly. Indeed, the material from the beds may be reused for local soil conditioning. In Peru, there has been a demand by the local farmers for treated sludge from water treatment plants. All localities can use chemical and biological sludges. Some effort may be required on the part of the utility or the plant operator to find disposal opportunities nearby.

21.7 Special Factors

Sand beds can be enclosed by glass. Glass enclosures protect the drying sludge from rain, control odors and insects, reduce the drying periods during cold weather, and can improve the appearance of a wastewater treatment plant.

Wedgewire drying beds have been used successfully in England. This approach prevents the rising of water by capillary action through the media; the construction approach lends itself well to mechanical cleaning. The first U.S. installations have been made at Rollingsford, New Hampshire and in Florida. It is possible, in these small plants, to place the entire dewatering bed in a tiltable unit from which sludge may be removed merely by tilting the entire unit mechanically. This approach is applicable to all small plants, and even larger plants where the drying units are made small enough for easy hand tilting.

21.8 Recommendations

Sludge disposal practices in Latin America and the Caribbean need considerable development. There are many simple approaches such as the one described in this section which should be applied immediately.

Sand drying beds are ideal for dewatering sludge in small or large plants. Some mechanization such as the use of screens and/or a tilting apparatus, can result in a more easily operable process.

The soil conditioning potential for most sludges is considerable. The possibility of using sludges in this manner should be actively explored with local agricultural persons.

22. LAND APPLICATION OF SLUDGE

22.1 Description

Techniques for applying liquid sludge, dried sludge and sludge cake to the land include tank truck, injection, ridge and furrow spreading and spray irrigation. Sludge can be incorporated into the soil by plowing, discing or other similar methods.

Municipal sludge contains all of the essential plant nutrients, but not necessarily in sufficient quantities to act as a fertilizer. It can be generally considered a fertilizer supplement, and soil conditioner. Its strongest potential use is as a soil conditioner. Also, the application of water treatment plant sludge to the land has been virtually ignored. Both water and wastewater sludges should be actively considered for land application in all areas where they are generated, but especially in poor soil areas.

Wastewater treatment plant sludge can be applied at rates which will supply all the nitrogen and phosphorous needed by most crops. When application rates are high however, the concentration in plants of certain elements, especially metals, may increase to objectionable levels. Animal diets are often deficient in trace elements such as zinc, copper, nickel, chromium and selenium. Thus sludge application may improve the quality of feeds and forages used for animal consumption. On the other hand, applications of some industrial wastes may result in animal and even human toxicity if food chain crops are harvested.

Design Criteria - Application rates of wastewater sludge depend on sludge composition, soil characteristics (usually 3% N; 2% P; 0.25% K), climate, vegetation and cropping practices. Annual application rates have varied from 0.5 to 10 tons (0.45 to 9.1 mt) per acre. Rates based on phosphorous needs are lower.

A pH of 6.5 or greater will minimize heavy metal uptake by most crops. Table 22.1 presents design criteria.

There are eight basic sludge disposal options for water treatment plant sludge (48):

- o discharge to waterway

- o discharge to sanitary sewers (not a good option in

In Latin America, because most sewer systems do not lead to treatment plants):

- o Codisposal with wastewater treatment plant sludge (few plants in Latin America)

- o Lagooning, requiring ultimate disposal of the residue

- o Mechanical dewatering with landfilling (soil conditioning value not recovered)

- o coagulant recovery

- o Land application, especially of softening sludge

- o Use for building or fill material.

Land application is an especially attractive option for water treatment plant sludges. No data could be found for costs of disposal of these sludges. It is hoped that as time passes, the land disposal option will be viewed as an increasingly attractive option, and more data will be developed. It is possible to use the data and information in this section with care for either water or wastewater sludges. Engineering judgement can be applied to arrive at realistic preliminary estimates of cost, and for sludge application rates. Land

TABLE 22.1

SLUDGE FERTILIZER VALUE

SLUDGE	NITROGEN	PHOSPHATE	POTASH
1 ton dry sludge provies	60 lbs (50% avail)	40 lbs	5 lbs
Typical corn fertilizer provides	180	50	60
If 6 tons dry sludge/acre were applied, would provide (lb/acre)	180 (avail)	240 (avail)	30 (avail)

Source: Reference 2

application has not been adequately utilized anywhere as illustrated on Table 22.2 (48). The basis for the data shown is a survey of U.S. facilities in 1981. But two previous surveys within the previous twenty years had shown almost no land application.

Design Approach for All Types of Sludges - The approach to be used for land application of virtually any sludge is based on the limiting contaminant concept. The maximum rates of sludge application are determined for EACH contaminant (e.g., nitrogen, water, metals, organics, etc.). That contaminant which possesses the limiting concentration (and thus will limit the mass application rate) limits the application rate of the sludge for the particular site (62). This is discussed further below.

Once a suitable application site (see Table 22.3) has been selected and the process objectives defined, proper sludge loading rates are determined. This process often involves characterizing the waste for a number of constituents. The following constituents are generally of most concern for municipal sludges (61): pathogens, phosphorus, nitrogen, cadmium, copper, nickel, lead, and zinc. When sludge is applied at rates to meet the nitrogen requirements of the crops being grown, nitrogen losses in excess of those expected from commercial fertilizer use should not be expected.

Regarding heavy metals, the useful life of land application sites can usually be based on the cumulative amounts of the five metals listed above. The recommended limits shown in Tables 22.4 and 22.5 should not interfere with crop growth or use of the crops at any future time, while serving to protect human and animal health.

After the allowable loading rate of each constituent has been calculated, the actual sludge loading rate is based on the most limiting constituent of those being considered. Frequently,

TABLE 22.2

METHODS FOR DISPOSAL OF WATER TREATMENT PLANT WASTE

	Percent of Plants Using Indicated Disposal Method	
	Softening Sludge	Coagulation Sludge
Sludge lagoon	34*	43
Sanitary sewer	8	27
River or lake	13	20
Recalcination	5	--
Direct land application	5	--
Other	--	10

* Fifty-six percent of plants surveyed had sludge lagoons, 60 percent of which were considered "permanent lagoons," thus 34 percent of plants used sludge lagoons for disposal.

Source: Reference 48.

TABLE 22.3

MAJOR SITE CONDITIONS FOR LAND APPLICATION OF SLUDGE

- Soil type
- Site susceptibility to flooding
- Slope
- Depth to seasonal ground water table
- Permeability of the most restrictive soil layer
- Cropping patterns and vegetative cover
- Nutrient and organic matter content

Source: Reference 61

TABLE 22.4

MAXIMUM AMOUNT OF METAL
(LB/ACRE, CUMULATIVE) SUGGESTED FOR
APPLICATION TO AGRICULTURAL SOILS

Metal	Soil Cation Exchange Capacity (meq/100g)		
	<5	5 to 15	>15
Lead	500	1,000	2,000
Zinc	250	500	1,000
Copper	125	250	500
Nickel	125	250	500
Cadmium	5	10	20

Source: Reference 61

TABLE 22.5

MAXIMUM AMOUNT OF CADMIUM (CUMULATIVE) SUGGESTED FOR APPLICATION TO AGRICULTURAL SOILS

Soil Cation Exchange Capacity (meq/100g)	If Background Soil pH is Below 6.5 (lb Cd/acre)	If Background Soil pH is Above 6.5 or Maintained at 6.5 by Liming (lb Cd/acre)
<5	5	5
5 to 15	5	10
>15	5	20

Source: Reference 61

the most limiting rate is dictated by the nitrogen (or alternatively the phosphorus) loading to meet crop needs. Higher loading rates than those calculated above can be used if: (1) non-food chain vegetation is used; (2) the site is well monitored.

- o Use sludges that are well stabilized and that have relatively low concentrations of critical contaminants.

- o Apply sludges at rates which meet crop nitrogen or phosphorus requirements.

- o Maintain soil pH near neutral.

- o Provide adequate monitoring of sludge quality and soils.

For high application rate disposal practices, or when sludges contain relatively high concentrations of critical contaminants and/or sludge stabilization is limited, one or more of the following additional precautions can be helpful:

- o Select remote application sites or limit access to the sites.

- o Apply sludge to areas used for the production of animal feed or non-food chain crops.

- o If needed, provide adequate monitoring of vegetation, surface and ground water.

22.2 Limitations

Constituents of sludge may limit the acceptable rate of application, the crop that can be grown, or the management or location of the site. Trace elements added to soil may accumulate in a concentration that is toxic to plants or is taken up and concentrated in edible portions or plants in a concentration that is harmful to animals or man. Trace element problems can be prevented by limiting the amount of sludge to be applied, industrial pretreatment, selection of tolerant or non-accumulating crops, selection of crops not used in the human food chain, and adapting appropriate agronomic practices such as lining of the soil. Where population is concentrated and agricultural land limited, sufficient land for sludge application may not be available. Terrain must be properly selected; steep slopes and low lying fields are less suitable and require more management. Equipment with standard tires can cause ruts, compacted soil and crop damage or get stuck in muddy terrain.

22.3 Costs

See Figures 22.2 and 22.3 (Figure 22.1 not used)(2, 11). Additional cost information is presented in Figures 22.4 and 22.5 for application to marginal lands (lands which have been disturbed). Table 22.6 presents the assumptions for the cost estimates in Figures 22.4 and 22.5.

22.4 Availability

Land application of wastewater sludge is practiced throughout the U.S. Perhaps as much as 40% of the sludge produced finds its way eventually to the land (61). This practice could be used immediately in Latin America and the Caribbean with no additional technology required. Equipment needs are simple. Once the nutrient requirements of the land are understood, the technology may be used.

Land application of softening sludge, the mixing of a waste material into the natural environment, is not a new method of disposal but has never been widely applied (48). Farmers were

Figure 22.2: Construction Cost of Land Application of Sludge

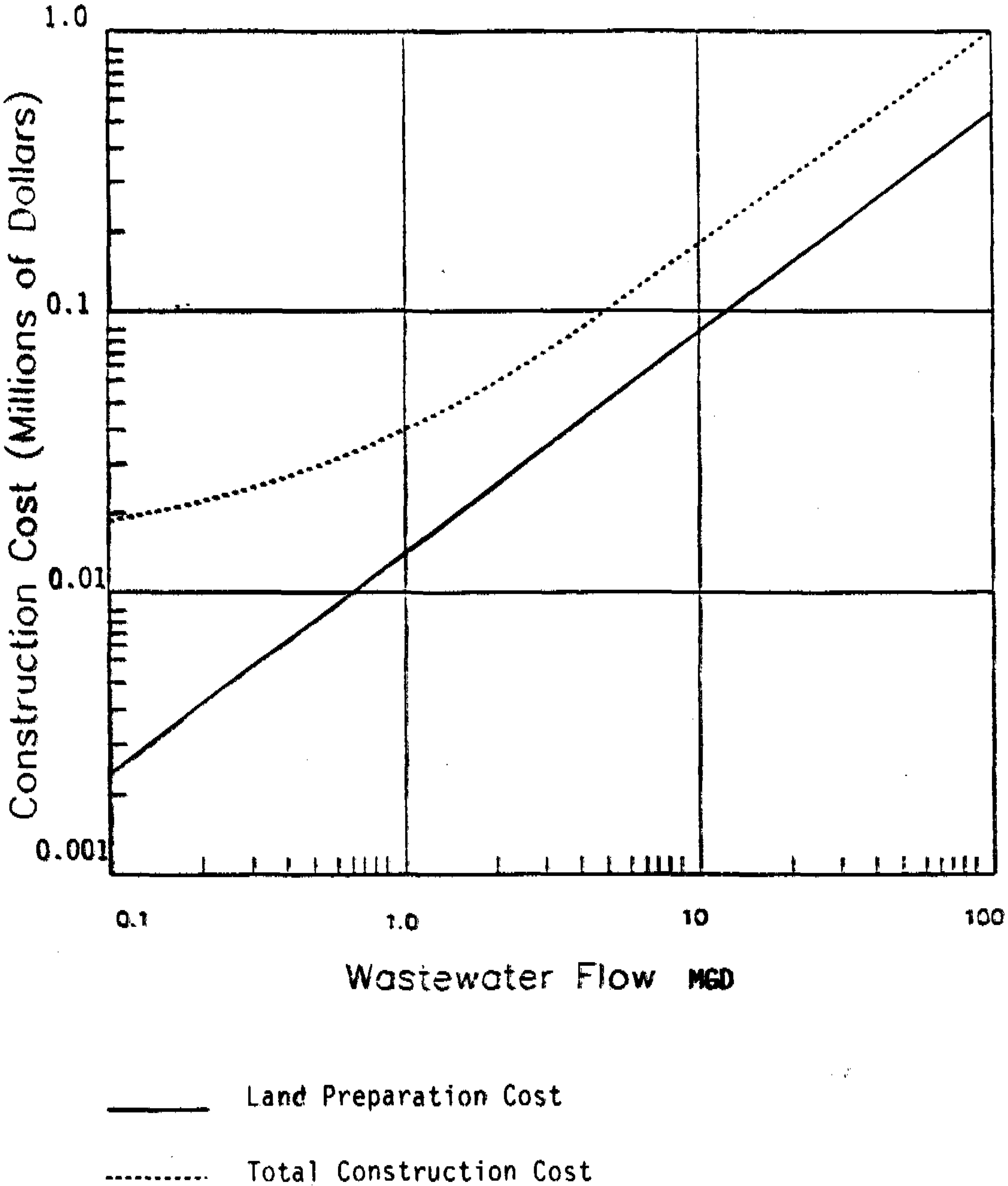

Figure 22.3: Operation & Maintenance Cost of Land Application of Sludge

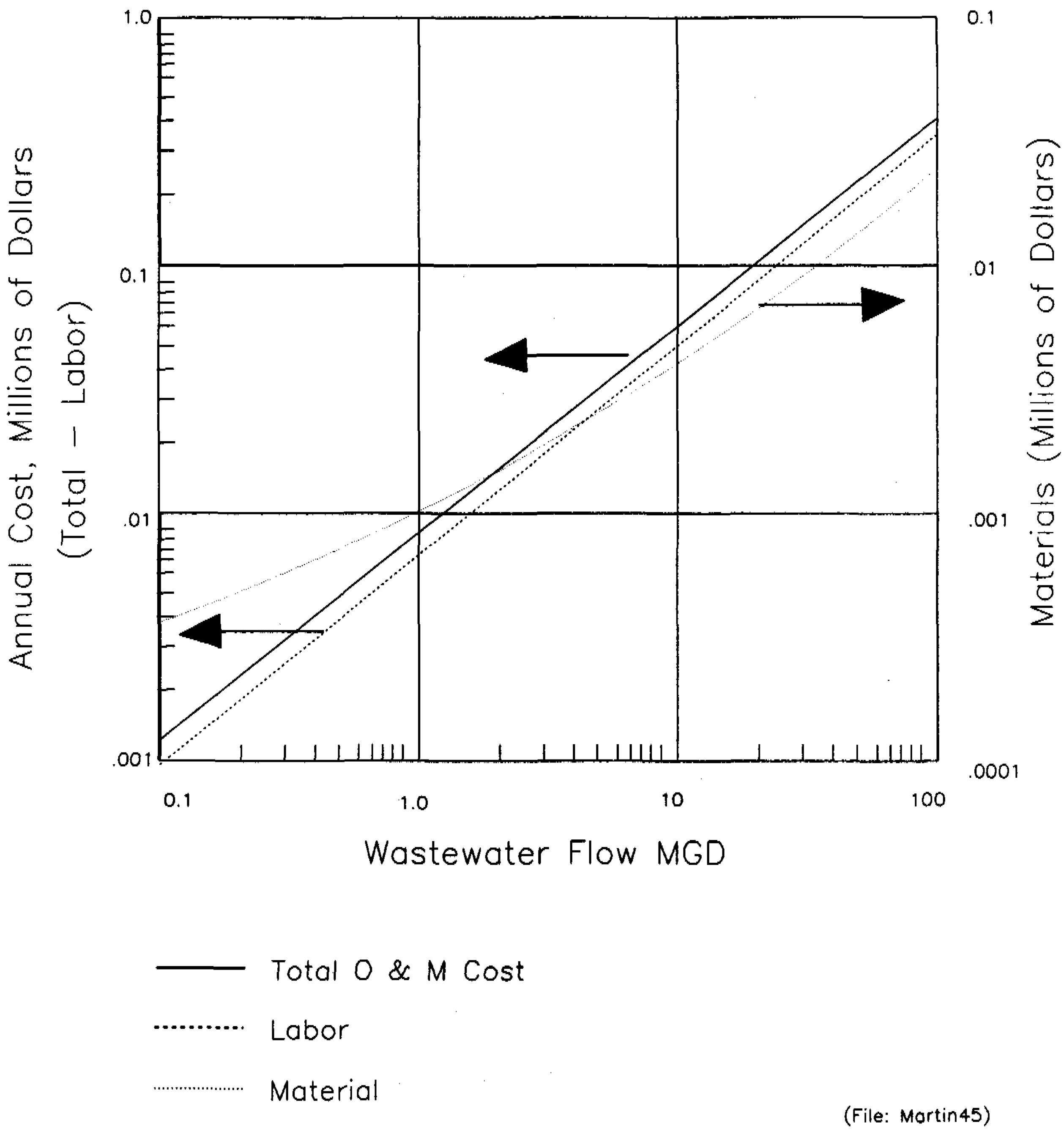

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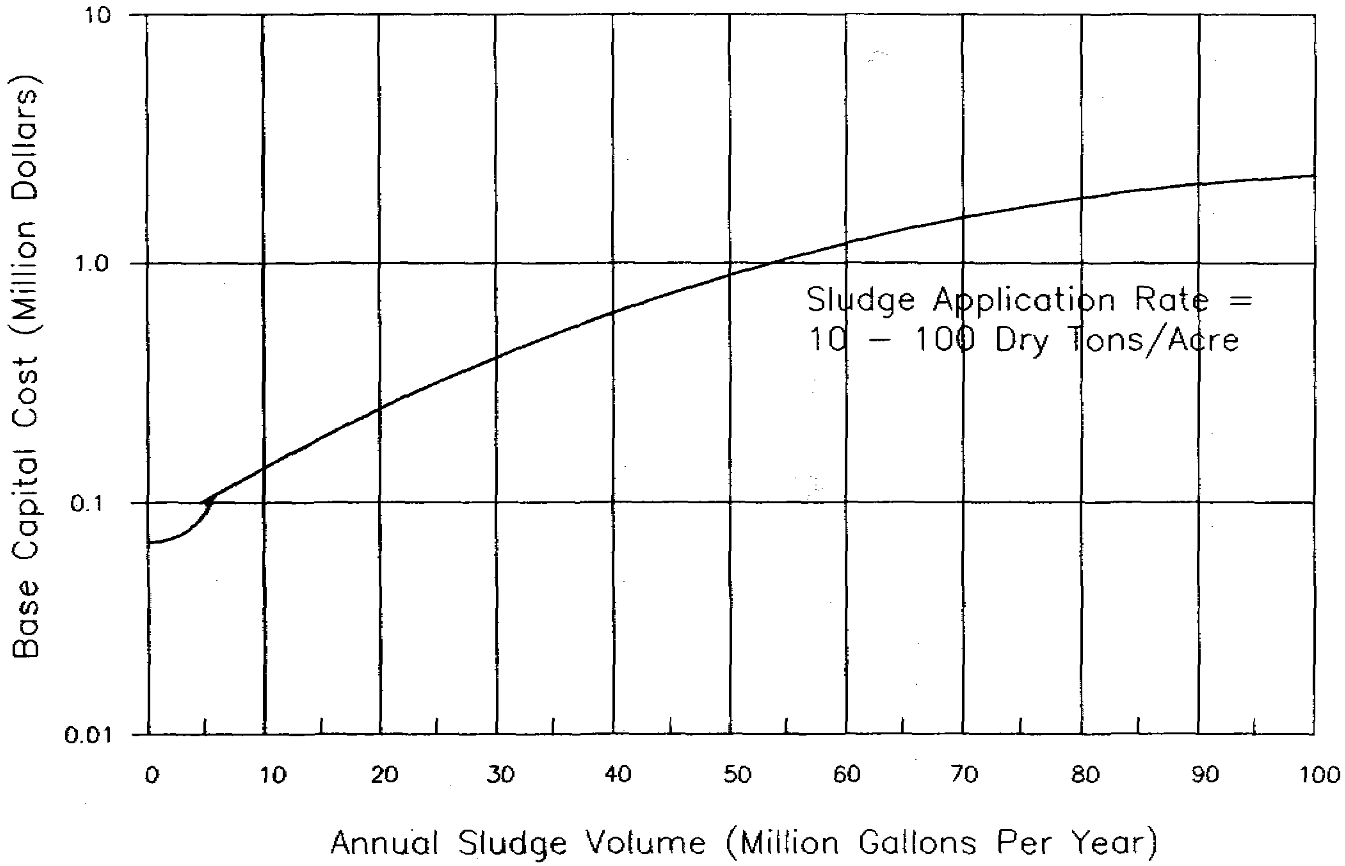
Figure 22.4

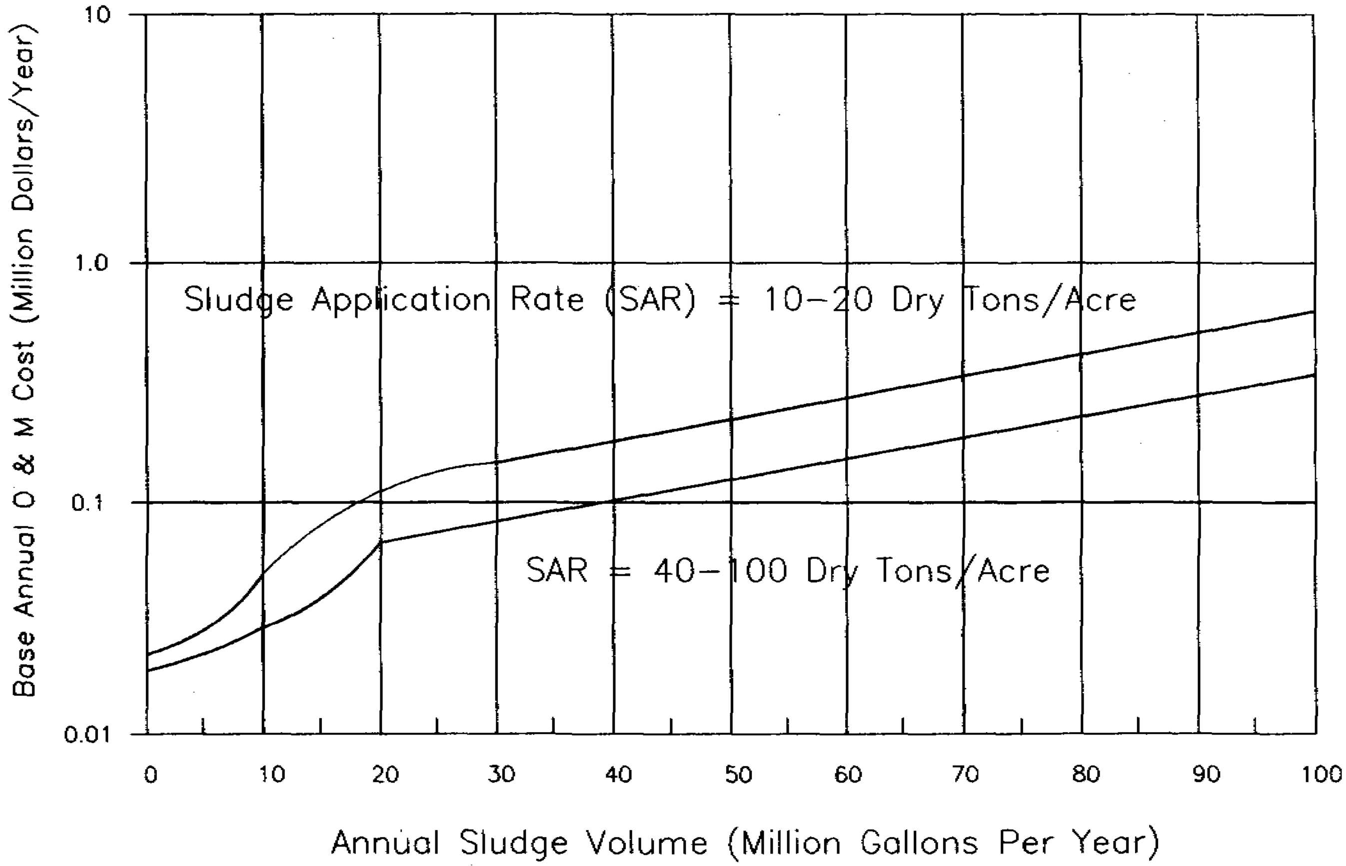
Base Capital Cost of Applying Sludge to Marginal Land For Reclamation As A Function of Annual Sludge Volume Applied



(Figure: Mart67)

Figure 22.5

Base Annual O & M Cost for Applying Sludge to Marginal Land For Reclamation As A Function of Annual Sludge Volume Applied And Dry Solids Application Rate



(Figure: Mart68)

Figure 22.6

Multiplication Factor to Adjust Sludge Application to Marginal Land Costs In Figure 22.4 For Variations In Days of Application Per Year

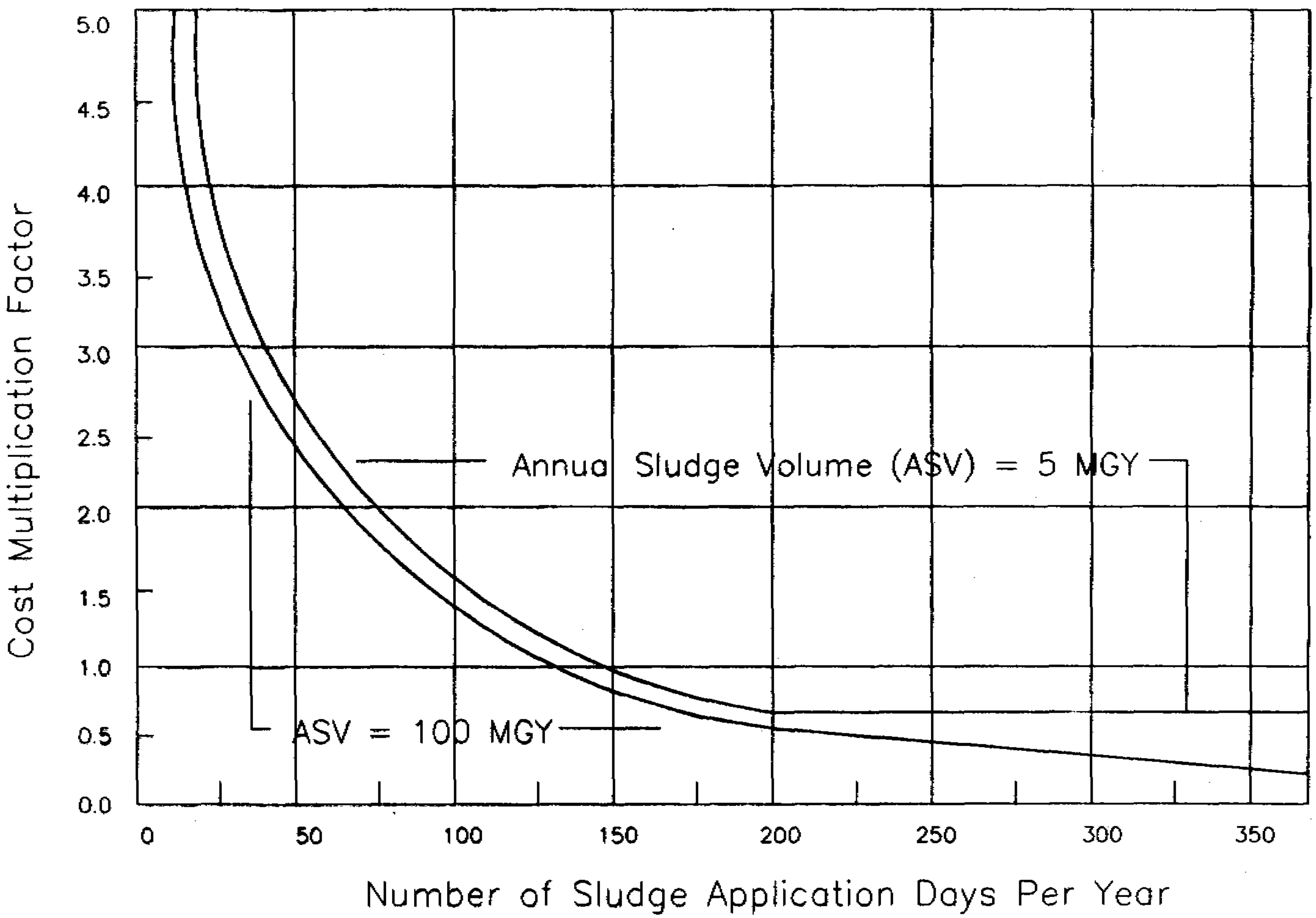


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TABLE 22.6

ASSUMPTIONS USED IN DEVELOPING COST REQUIREMENT CURVES FOR LAND APPLICATION OF SLUDGE TO MARGINAL LAND

Parameter	Assumed Value
Sludge Solids Concentration	5 percent
Daily Application Period	7 hr/day
Annual Application Period	140 days/yr
Fraction of Land Required in Addition to Application Area	0.3
Fraction of Land Area Requiring Lime Addition	0
Fraction of Land Area Requiring Grading	0
Cost of Land	0
Cost of Lime Addition	0
Cost of Grading Earthwork	0
Cost of Operation Labor	$ 14.90/hr
Cost of Diesel Fuel	$ 1.50/gal
Cost of Monitoring Wells	$ 5,700 each

Source: Reference 51

allowed to remove dewatered softening sludge from a plant in Ohio approximately 30 to 40 years ago.

The solids content of softening sludge discharged from clarifiers is 1 to 5 percent. If land application of softening sludge is employed, this sludge should be thickened and applied for soil conditioning as a liquid at 8 to 10 percent solids or as a solid after dewatering to about 40 percent solids. If the solids content of the sludge is between these values, and if conventional farming equipment is used, handling problems will be encountered.

In farming regions, the application of nitrogen fertilizers causes a reduction in soil pH. If optimum pH conditions do not exist, crop yields will be reduced. Therefore, farmers must apply sufficient quantities of calcium carbonate as a means of counteracting the fertilizer applications. For each 100 pounds (45.4 kg) of ammonia fertilizer, 3 to 4 pounds (1.4 to 1.8 kg) of limestone must be applied.

In 1969, the Ohio Department of Health reported that the total neutralizing power of lime sludge is greater than that of marketed liming materials. To bring the soil pH into the desirable range, 3 tons/acre (0.6t7 kg/m^2) lime, or about 10 tons (9.07 metric tons) of lime sludge at a 30 percent solids concentration, are required. Subsequent lime applications are required to maintain the desired pH.

In Illinois, a calcium carbonate equivalent test performed on several softening sludges indicate that the softening sludges were superior to agricultural limestones available locally. Because softening sludges contain a high quantity of calcium carbonates and offer a high degree of neutralization, this resource should be used when it is practical for soil conditioning. The addition of softening sludge also increased the porosity of tight soils, making them more workable for agricultural purposes.

Both alum and lime sludge increase the cohesiveness of soils, but, more important, both sludges also widen the moisture content over which soil remains cohesive. Alum sludge increases soil cohesiveness at high moisture contents, while lime sludge increases soil cohesiveness at lower moisture contents.

Use of lime sludge on agricultural land has had limited success because farmers are unfamiliar with its use as a soil conditioner and with the logistics of its transportation and applications to their lands. In addition, the availability of dewatered sludge needs to be scheduled to coincide with farmers' demands. Agricultural use of dewatered sludge would offset some disposal costs. Alum sludge has little soil conditioner value.

22.5 Operation and Maintenance

Ridge and furrow methods involve spreading sludge in the furrows and planting crops on the ridges. Utilization of this technique is generally best suited to relatively flat land and is well suited to certain row crops. Spray irrigation systems are more flexible, require less soil preparation and can be used with a wider variety of crops. High application rates are commonly used to reclaim strip mine spoils or other low quality land. Sludge spreading in forests has been limited, but offers opportunities for improved soil fertility and increased tree growth.

22.6 Control

As a disposal process, very reliable; as a utilization process, careful control should be exercised as described above.

22.7 Special Factors

Potential for toxins and pathogens to contaminate soil, water, air, vegetation and animal life, and ultimately hazardous to humans. Accumulation of toxins in the soil may cause phytotoxic effects, the degree of which varies with the tolerance level of the particular plant species and variety. Toxic substances such as cadmium that accumulate in plant tissues can subsequently enter the food chain, reaching human being directly through ingestion or indirectly through animals. If available nitrogen exceeds plant requirements and it can be expected to reach groundwater in the nitrate form. Toxic materials and pathogens can contaminate groundwater supplies or can be transported by runoff or erosion to surface waters if improper loading occurs. Aerosols which contain pathogenic organisms may be present in the air over a landspreading site, particularly where spray irrigation is the means of sludge application. Some pathogens remain viable in the soil and on plants for periods of several months; some parasitic ova can survive for a number of years. Other potential impacts include public acceptance and odor.

22.8 Recommendations

This technology is a popular method of disposal because it is simple, which is of extreme importance in developing countries. It also serves as a utilization measure since it is beneficial as a soil conditioner for agricultural, marginal or drastically disturbed (e.g, mined) land. Finally, wastewater treatment sludges contain considerable quantities of organic matter, all of the essential plant nutrients and a capacity to produce water-containing humus.

23. CHLORINATION (Disinfection)

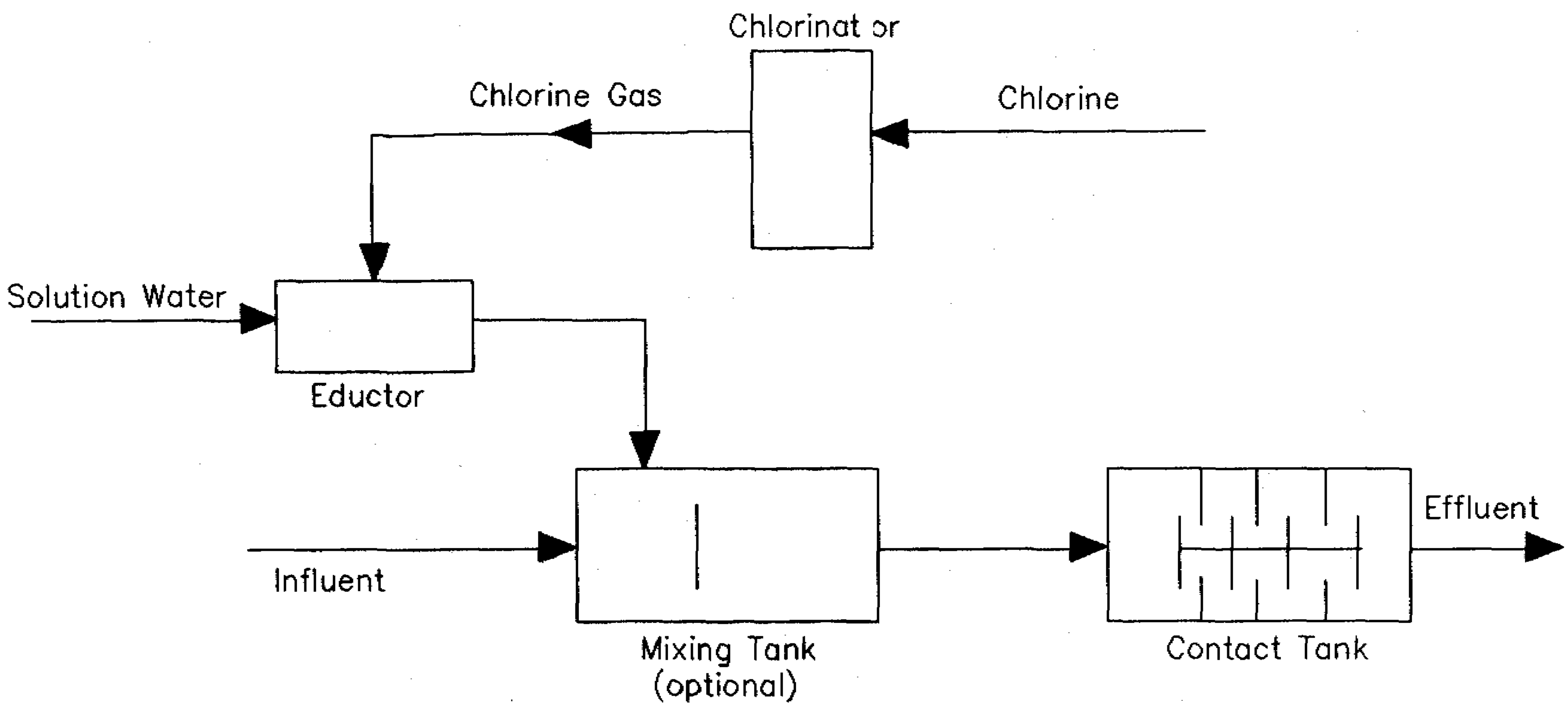
23.1 Description

Chlorination is the most commonly used water and wastewater disinfection process worldwide. This process involves the addition of elemental chlorine or hypochlorite, either calcium or sodium (the calcium form most frequently used High Test Hypochlorite, HTH), to the wastewater Figure 23.1 shows a gas chlorination unit schematically (2).

Chlorine is supplied as a liquified gas under high pressure in containers varying in size from 100 lb., 150 lb., and up to 1 ton (0.91 mt) as well as tank cars of larger sizes. Precautions should be taken when handling chlorine gas: 1) Chlorine gas is both very poisonous and very corrosive. Adequate exhaust ventilation at floor level should be provided since chlorine gas is heavier than air. 2) Chlorine-containing liquid and gas can be handled in black wrought-iron piping, but chlorine solution is highly corrosive and should be handled in rubber-lined or resistant plastic piping with hard rubber parts where necessary. 3) Cylinders in use are set on platform scales flush with the floor, and the loss of weight is used as a positive record of chlorine dosage (4, 5, 6, 7). See Figure 23.2 (3).

Calcium hypochlorite is available commercially in either a dry or wet form. High-test calcium hypochlorite (HTH) contains about 60% available chlorine. Because calcium hypochlorite granules or pellets are readily soluble in water and, under proper storage conditions are relatively stable, they are often favored over other available forms. Because of its oxidizing potential, calcium hypochlorite should be stored in a cool, dry location away from other chemicals in corrosion-resistant containers (4, 6). Figure 23.3 shows a typical hypochlorite installation (7).

Figure 23.1: A Flow Diagram of Chlorination



(Source: Ref. 2)

Figure 23.2: Electrical Pump Connection to Hypochlorinator with On-Off Control

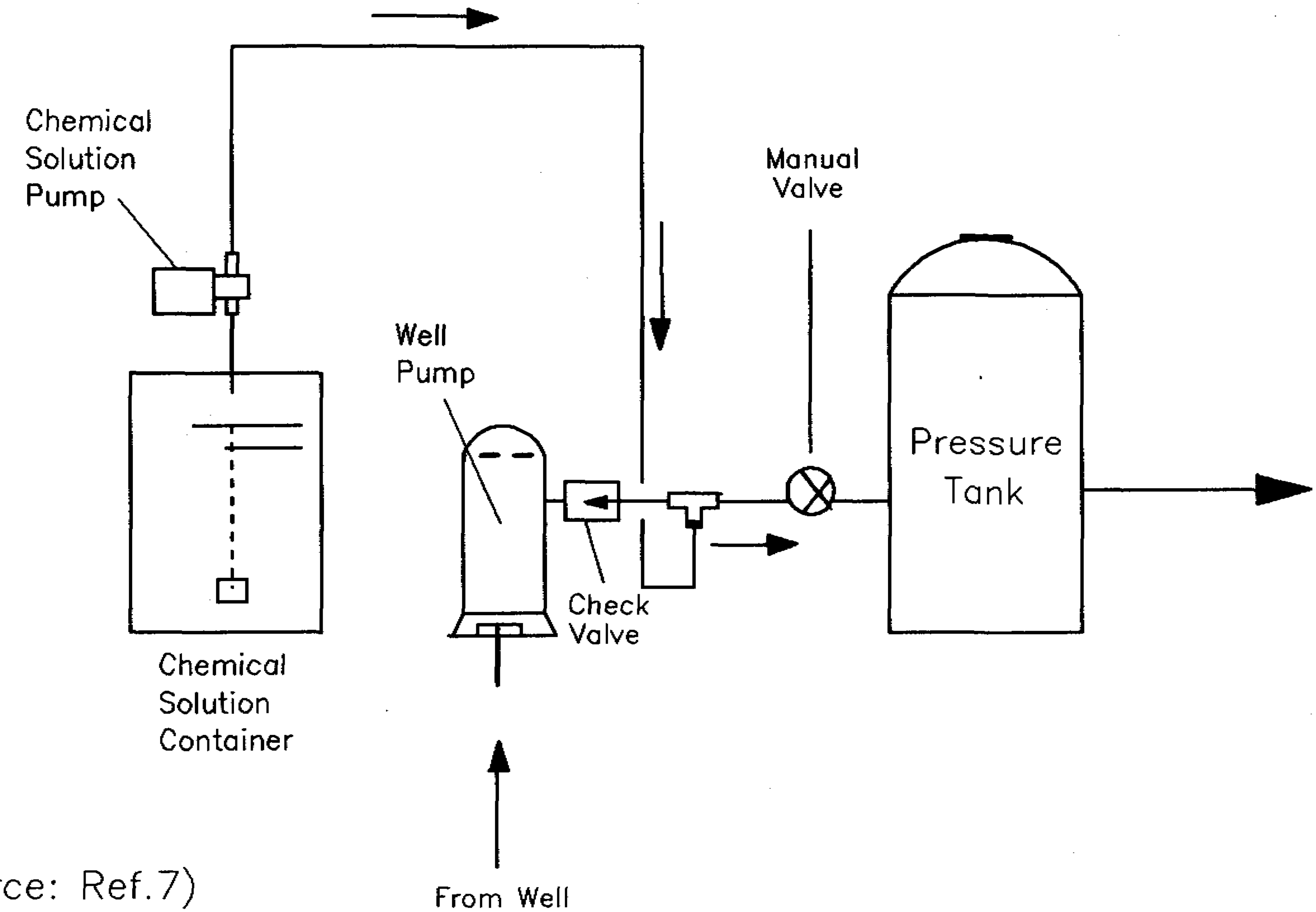


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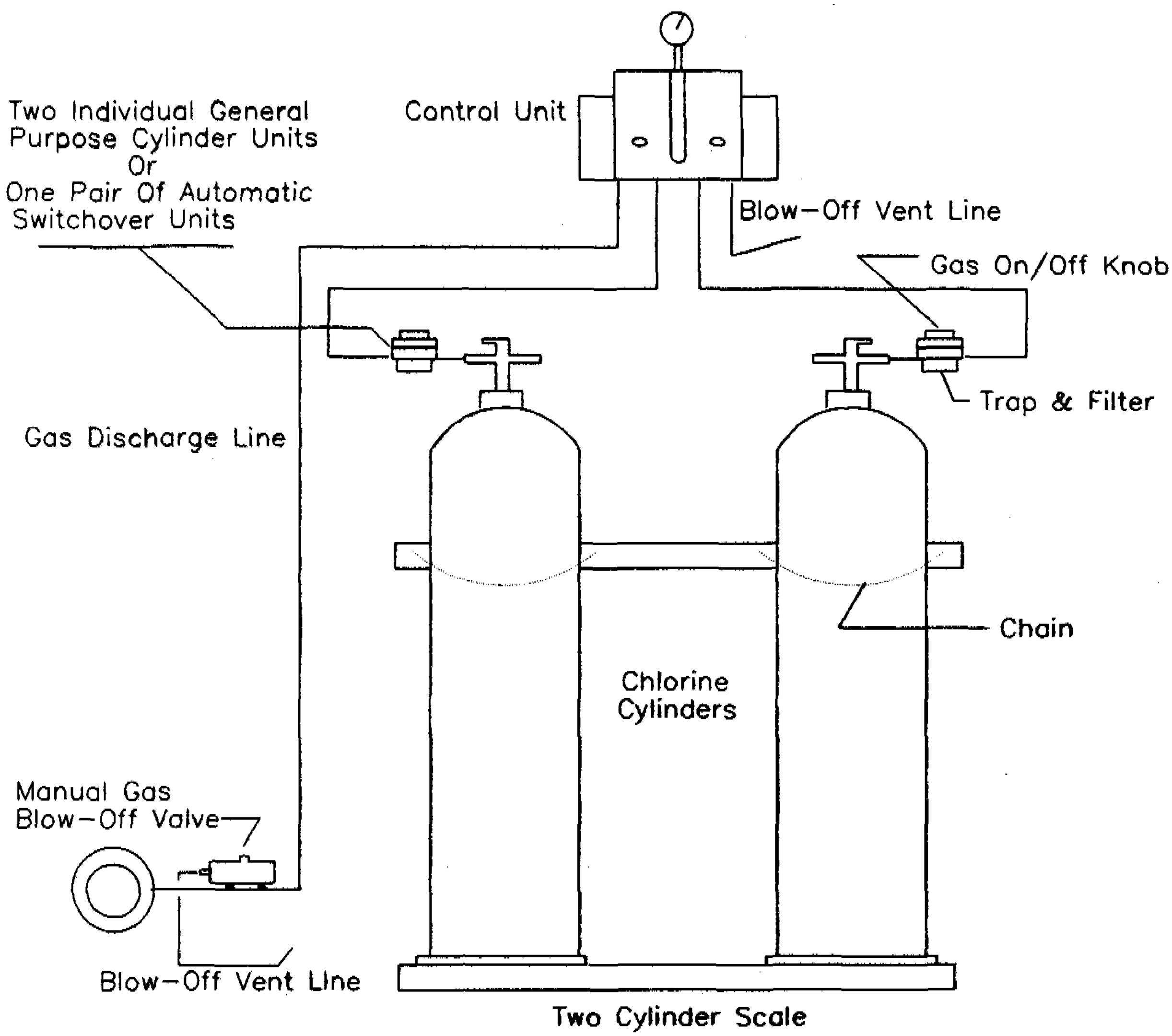
Figure 23.3: A Typical Chlorine Cylinder Setup For Chlorination Treatment



(Source: Ref. 3)

(File:Martin76)

Sodium hypochlorite is available in strengths from 1.5% to 15% with 3% the typical maximum strength used; thus transportation costs may limit its application. The solution decomposes more readily at high concentrations and is affected by exposure to light and heat. It must therefore be stored in a cool location in a corrosion-resistant tank.

Disinfection is designed to kill harmful organisms, and generally does not result in sterile water (free of all microorganisms). Table 23.1 (2) shows some wastewater dosages and results which may be expected. Typically, 30 minutes of chlorine contact time is required with good mixing. Water supply treatment dosages are established on the basis of maintaining a residual in the treated water.

Design Criteria - Generally a contact period of 15-30 minutes at peak flow is required. Detention/contact tanks should be designed to prevent short-circuiting. This usually involves the use of baffling as shown schematically in Figure 23.1. Baffles can either be the over-and-under or the end-around varieties. Residual concentrations of at least 0.5 mg/l in water supply are necessary, and desirable for chlorination of wastewater discharges.

Chlorine may be applied by two basic methods: 1) gas chlorination employing compressed chlorine gas or 2) hypochlorination employing a chemical feed pump to inject a water solution of chlorine compounds.

Gas Chlorination - A gas chlorinator must be employed to meter the gas flow and mix it with water. The mixture is then injected as a water solution. Small water supplies can effectively handle the 100 or 150 lb container but the larger containers are not recommended for small systems as special hoists and cradles are required for handling. Chlorine gas is a highly toxic lung irritant and special facilities are required

TABLE 23.1

TYPICAL CHLORINE DOSAGES AND RESULTS

Chlorine Residual, mg/l	Total Coliform MPN/100 ml Primary Effluent	Secondary Effluent
0.5 - 1.5	24,000 - 400,000	1,000 - 12,000
1.5 - 2.5	6,000 - 24,000	200 - 1,000
2.5 - 3.5	2,000 - 6,000	60 - 200
3.5 - 4.5	1,000 - 2,000	30 - 60

In normal low dose disinfection treatment, the COD, BOD_5, and TOC of the treated wastewater are not measurably changed.

Effluent From	Dosage range, mg/l
Untreated wastewater (prechlorination)	6-25
Primary sedimentation	5-20
Chemical-precipitation plant	3-10
Trickling-filter plant	3-10
Activated-sludge plant	2-8
Multimedia filter following activated-sludge plant	1-5

Source: Reference 2

for storing and housing gas chlorinators. The advantage of this method is the convenience afforded by a relatively large quantity available for continuous operation for several days or weeks without mixing chemicals. Gas chlorinators have an advantage where variable water flow rates are encountered, as feed rates may be synchronized to inject variable quantities of chlorine. Capital costs are somewhat greater for gas chlorination but chemical costs may be less.

Hypochlorination - Water solutions of either the liquid or dry forms are prepared in predetermined stock solution strengths. Solutions are injected into the water supply using special chemical metering pumps called hypochlorinators. The positive displacement types are highly accurate and reliable and are preferred over hypochlorinators employing other feed principals (usually based on a suction principal). Positive displacement type hypochlorinators are readily available at relatively modest costs. These small chemical feed pumps are designed to pump (inject under pressure) an aqueous solution of chlorine into the water systems. They are designed to operate against pressures as high as 100 psi (6.9 x 105 Pa)but may also be used to inject chlorine solutions at atmospheric or negative head (suction side of water pump) conditions. Hypochlorinators come in various capacities ranging from 1.0 to 60 gals/day (3.8 to 227 l/day). The pumping rate is usually manually adjusted by varying the stroke of the piston or diaphragm. Once the stroke is set, the hypochlorinator feeds accurately at that rate, maintaining a constant dose. This works effectively if the water supply rate is fairly constant, as with the output of a pump. If the water supply rate varies considerably, a metering device may be used to vary the hypochlorinator feed rate synchronized with the water rate. When a well pump is used, the hypochlorinator is connected electrically with the on-off controls of the pump.

23.2 Limitations

Chlorination may cause the formation of chlorinated hydrocarbons, some of which are known to be carcinogenic compounds (e.g, halomethanes). The effectiveness of chlorination is greatly dependent on pH and temperature of the wastewater. Chlorine gas is a hazardous material (as mentioned above), and requires sophisticated handling procedures. Chlorine will react with certain chemicals in the wastewater, leaving only the residual amounts of chlorine for disinfection. Chlorine will oxidize ammonia, hydrogen sulfide, as well as metals present in their reduced states.

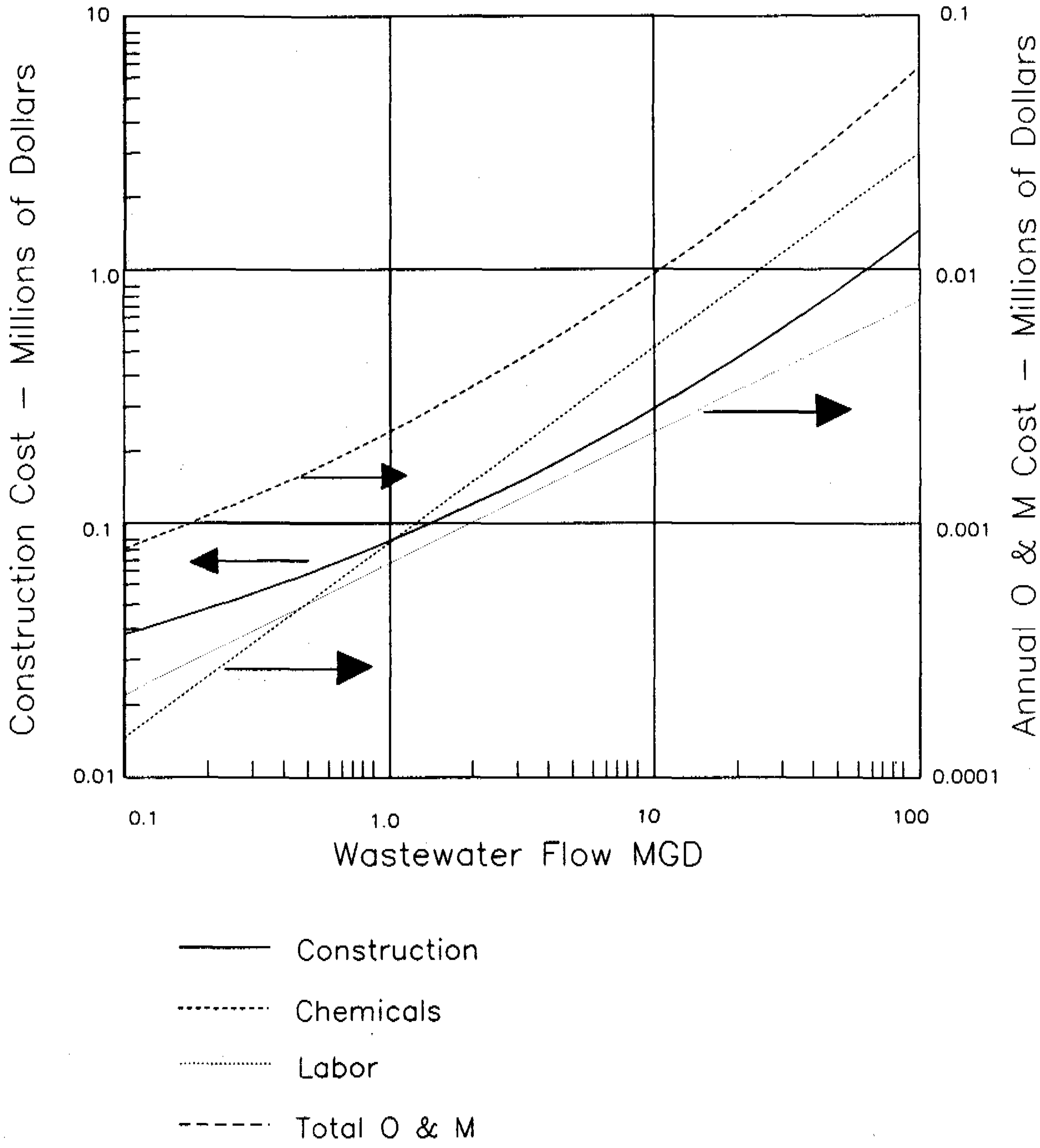
23.3 Costs

See Figure 23.4 (2, 11).

23.4 Availability

Chlorination of water supplies on an emergency basis was practiced as early as about 1850. Presently, chlorination of both water supplies and wastewaters is a widespread practice throughout the world. Designs for systems using either gas or liquid external (i.e, not produced at the site) sources of chlorine should be very cognizant of the need to continuously and consistently supply chlorine in order for the process to be effective. In place systems which are not operating are not disinfecting supplies of water before use. Inconsistent disinfection of wastewaters may adversely impact the downstream water quality, and potentially downstream users. Throughout the course of many visits, chlorination systems were typically not operational, especially in small and medium sized communities. There were many reasons, including the inability to get gas cylinders to the top of the Andes, except by llama.

Figure 23.4: Construction, Operation & Maintenance Costs for Chlorination (Disinfection).



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23.5 Operation and Maintenance

Gas Chlorinator

Normal operation of a gas chlorinator requires routine observation and preventive maintenance. Duties of an operator should include:

DAILY

- o Read the chlorinator rotameter
- o Record the reading time, date and initial the entries.
- o Read the meters and record the number of gallons of water pumped.
- o Check the chlorine residual. If the residual is too low in the distribution system, increase the feed rate by adjusting the rotameter. If the residual is too high, lower the feed rate by adjusting the rotameter.
- o Calculate the chlorine usage.

WEEKLY

- o Clean the equipment and the building.
- o Perform preventive maintenance on the equipment.

Hypochlorinator

Operators should follow the maintenance plans outlined below.

DAILY

An operator should:

- o Read and record the level of the solution tank at the same time every day.
- o Read the meters and record the amount of water pumped.
- o Check the chlorine residual (0.2 mg/L) in the system and adjust the chlorine feed rate as necessary. Try to maintain a chlorine residual of 0.2 mg/L at the most

remote point in the distribution system. The suggested free chlorine residual for treated water or well water is 0.5 mg/L at the point of chlorine application provided the 0.2 mg/L is maintained throughout the distribution system.

- o Check the chemical feed pump operation. Most hypochlorinators have a dial with a range from 0 to 10 which indicates the chlorine feed rate initially set the pointer on the dial to approximately 6 or 7 on the dial and use a two percent hypochlorite solution. The pump should be operated in the upper ranges of the dial. This will require the frequency of the strokes or pulses from the pump to be frequent enough so that the chlorine will be fed continuously to the water being treated. Adjust feed rate after testing chlorine residual levels.

WEEKLY

An operator should:

- o Clean the building.
- o Replace the chemicals and was the chemical storage tank. Try to have a 15 to 30-day supply of chlorine in storage for future needs. When preparing hypochlorite solutions, prepare only enough for a two or three-day supply.

MONTHLY

An operator should:

- o Check the operation of the check valve.
- o Perform any required preventive maintenance.
- o Cleaning

  Commercial sodium hypochlorite solutions (such as chlorox) contain an excess of caustic (sodium hydroxide or NaOH). When this solution is diluted with water containing calcium and also carbonate alkalinity, the resulting solution becomes supersaturated with calcium carbonate. This calcium carbonate tends to form a coating on the poppet valves in the solution feeder. The coated valves will not seal properly and the feeder will fail to feed properly.

Please note that hypochlorinators on small systems are normally small sealed systems that cannot be repaired, so replacement of the entire unit is the only solution. Maintenance requirements are normally minor, such as changing the oil and lubricating the moving parts.

23.6 Control

These systems are extremely reliable. The hypochlorite system is somewhat easier to operate than the gas system. Operators need not be as skilled or as cautious.

23.7 Special Factors

For safety reasons the following hazards should be noted for all operating personnel:

Chlorine hazards

Chlorine is a gas, heavier than air, extremely toxic and corrosive in moist atmospheres. Dry chlorine can be safely handled in steel containers and piping, but with moisture must be handled in corrosion-resisting materials such as silver, glass, teflon, and certain other plastics. Chlorine gas at container pressure should never be piped in silver, glass, teflon, or any other material that cannot handle the pressure. Even in dry atmospheres, chlorine combines with the moisture in the mucous membranes of the nose and throat, and with the fluids in the eyes and lungs; a very small percentage in the air can be very irritating and can cause severe coughing. Heavy exposure can be fatal.

Hypochlorite Safety

Hypochlorite does not present the hazards that gaseous chlorine does and therefore is safer to handle. When spills occur, wash with large volumes of water. The solution is messy to handle. Hypochlorite causes damage to your eyes and skin upon contact. Immediately wash affected areas thoroughly with water. Consult a physician if the area appears burned. Hypochlorite solutions are very corrosive. Hypochlorite compounds are non-flammable; however, they can cause fires when they come in contact with organics or other easily oxidizable substances.

Calcium hypochlorite (HTH) is used by a number of small water supply systems. A problem occurs in these systems when sodium fluoride is injected at the same point as the hypochlorite. A severe crust is formed when the calcium and fluoride ions combine.

Dechlorination may be used for wastewater treatment, which generally involves the addition of sulfur dioxide, aeration, or even activated carbon, when chlorine residual standards are strict.

23.8 Recommendations

Chlorination for disinfection is used to prevent the spread of waterborne diseases and to control algae growth and odor. Chlorination is the most popular (and probably the only) method of disinfection used in developing countries. Economics, ease of operation and convenience are the qualities used for evaluation.

For safety and transport (consistency of supply of chlorine) reasons, it is useful to consider on-site generation of chlorine

(6). Most commercially available equipment for chlorine generation will operate on seawater, as well as brine solutions prepared for the purpose. About 2.5 kWh per lb of available chlorine is required, and is fairly equal for all commercial equipment. Hypochlorite solutions prepared from seawater are usually limited to about 1800 mg/l available chlorine, and those produced form brine to about 8000 mg/l. Heavy metal ions present in seawater interfere with stability of hypochlorite solutions prepared from this source.

Hypochlorite can also be made on-site by using common salt, manganese dioxide, (these products can be mined nearby), low grade slaked lime and sulfuric acid. The manganese dioxide and common salt are mixed and placed in a reaction tank which is suspended above an open type water boiler. Above the chemical reaction tank is a sulfuric acid tank with a control valve which allows regulation of the flow of acid to the reaction tank. The rate of chlorine gas generation is regulated by the flow of sulfuric acid and the temperature of the water. The chlorine gas generated is passed through a foam trap and desiccator, where it is dried before going to an absorption chamber. This chamber consists of a quarter inch of slaked lime. Chlorine reacts and a bleaching powder is formed. It has been proven that this method will produce about 30 pounds of bleaching powder with 35 percent available chlorine in 12 hours of operation. (67)

24. ULTRAVIOLET DISINFECTION

24.1 Description

The primary function of ultraviolet (UV) radiation is to disinfect drinking water as well as wastewater. Commercially, the UV light is generated artificially by a wide variety of arcs and incandescent lamps. For disinfection, the UV radiation is generated from special low pressure mercury-vapor lamps that produce UV radiation as a result of an electron flow between the electrodes through ionized mercury vapor and also laser generated UV radiation systems although the necessary lamps are not commonly used. The inactivation of micro-organisms by UV radiation is based on photochemical reactions in the DNA that result in coding system errors. The UV lamp is submerged in or suspended above the treated water. Figure 24.1 (20) is a schematic representation of UV disinfection unit showing submerged (insertion into a quartz sleeve) lamp placed perpendicular to the direction of waste water flow (18, 2(.

Ultraviolet (UV) radiation has been considered as an alternative means of disinfecting small drinking water supplies. A major impetus for this was the increase in reported waterborne disease outbreaks caused by Giardia Lamblia, an organism that is highly resistant to conventional chlorination. Other advantages include:

1. No chemical consumption -- eliminates large scale storage, transportaion and handling, and potential safety hazards.
2. Low contact time -- no contact basin is necessary and space requirements are reduced.
3. No harmful by-products are formed.
4. A minimum of or no moving parts -- high reliability.
5. Low energy requirements.

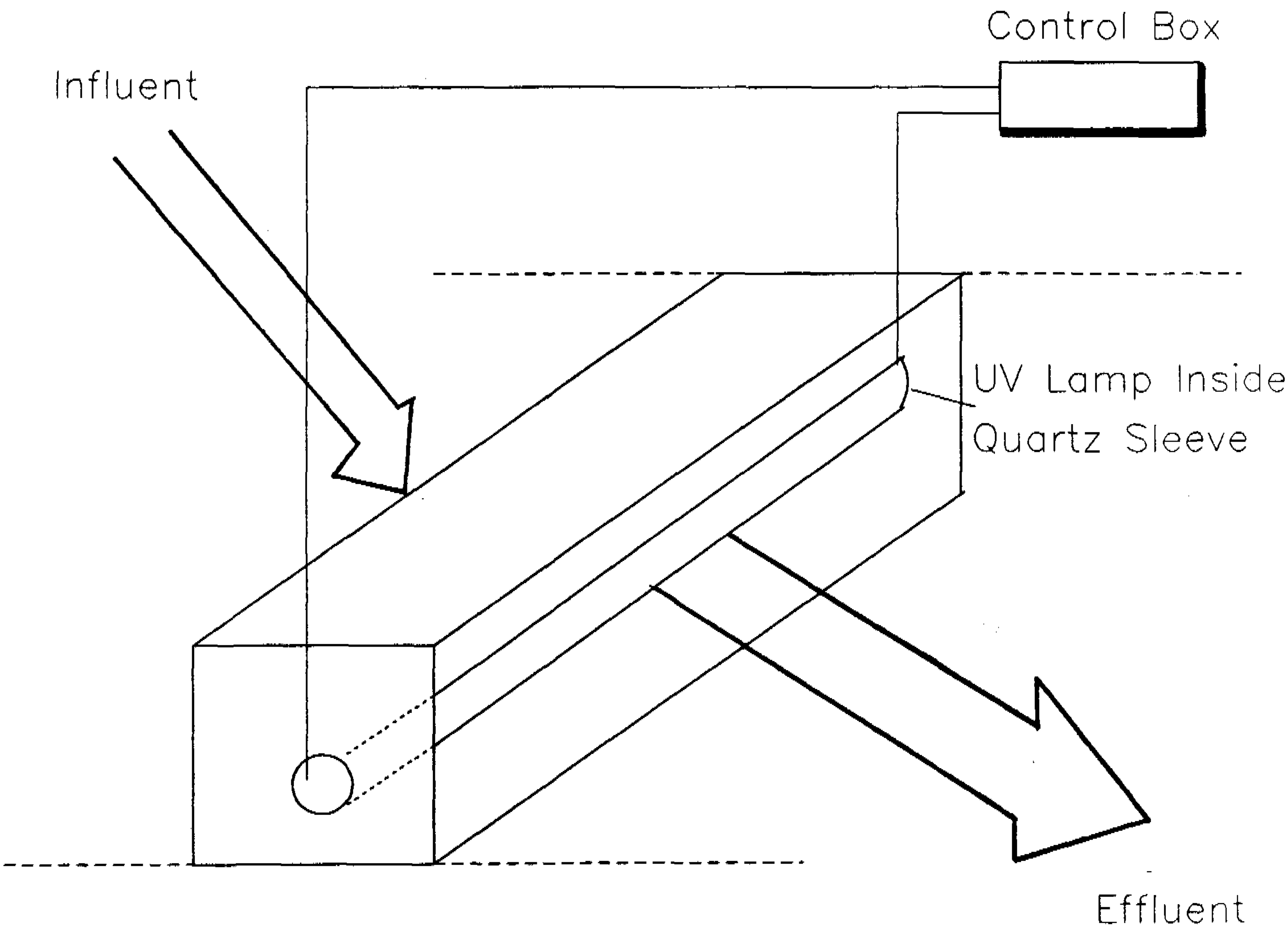
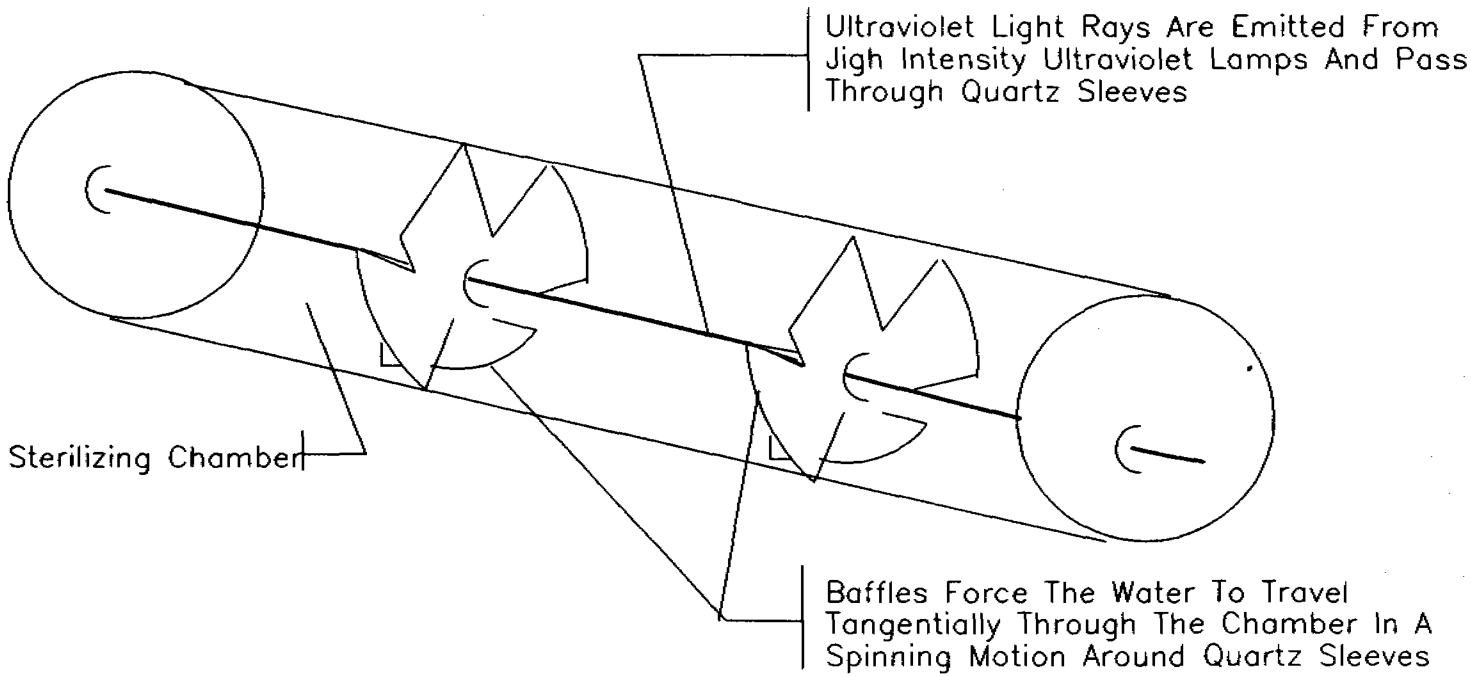


Figure 24.1: Ultraviolet Design Schematic

(Source Ref. 20)

Figure 24.2: Typical UV Sterilizing Chamber



Note: Typical Sterilizers Employ One To Twelve Lamps Per Sterilizing Chamber

(Source: Ref. 19)

(File:Martin10)

The UV lamp is submerged in or suspended above the water to be treated. A relatively new configuration marketed by a firm in Brazil uses a series of vertical flow-through cylinders with the UV bulbs located in an annular tube. Thus, the flow is parallel to (longitudinal) rather than perpendicular to the UV source in Figure 24.1. Also see Figure 24.2. The wavelength of useful UV light for disinfection is 4 to 400 nm. The maximum microbiological kill is in the narrow band between 200 and 310 nm. Apparently the most efficient wavelength is around 260 nm. This is a limitation of UV radiation as an effective disinfection process, although mercury vapor lamps can now be commercially manufactured to emit near this wavelength.

Testing of UV has been done by U.S.EPA contractors for small water supplies, and found to be effective on Giardia, Yersinia and G. Muris cysts, as well as the lesser resistant E. Coli and Y Enterocolitica cysts. Giardia cysts have been found to be resistant to chlorine disinfection.

The lethal effect of UV radiation (lethal dose: 110 MW. sec/cm^2 @ a 20 min. duration) occurs because of a photochemical reaction initiated by absorption of a photon by the molecular structure of the microorganism, rather than by the presence of an oxidizing agent or toxic chemical (as in the case of chlorine or ozone). Thus, another limitation is that the germicidal action depends on the absorption of UV radiation in the lethal wavelength range. The organisms must actually be exposed to the light source. Process effectiveness is interfered with by the presence of suspended solids (which must be kept in the range of 10-15 mg/l and probably lower). Turbidity and color are as important, perhaps more important than suspended solids concentration. Initial studies on G. Muris have shown that color increases cyst survival at 254 nm and that turbidity has little or no affect on cyst survival. Other studies on G. Muris cysts indicated that storage time and temperature affected the viability of the cysts and that the rate of decrease in viability approximately doubled with each 10oC increase in temperature above freezing, however, cyst viability was shortened to hours rather than to days for above freezing conditions. Physical stress produced by pressure and alum addition in water treatment processes appeared to damage and even destroy cysts. All of these factors aid in UV disinfection of water, and should be considered if a UV system is to be installed. Future work may provide additional data on these aspects.

Reactor design should include isolatable modules so that maintenance can be easily done. Lamps and ballast MUST be accessible. Operators must be trained to wear goggles and must be made aware of the hazards. Design should include redundant modules. Given the lack of actual plant operating experience, perhaps complete redundancy should be considered.

The evaluation process should include consideration of:

- power supply - continuity
- current (1988) UV equipment reliability
- safety tradeoffs - all disinfection systems
- achievable UV exposure indices under actual plant conditions
- impact of TSS, color and turbidity variations

24.2 Limitations

Intensity of the lamps and consistency of output at required wavelengths has been a problem. What is required is manufacturing standards, equivalent to say, pump manufacturers criteria, to insure equipment manufacturing consistency, output, and effectiveness.

The efficiency of a UV system is related to a dispersion index, which in turn is related to exposure time and lamp configuration (largely spacing of lamps). An ideal dispersion index is zero. Exposure time is the order of 10 to 30 seconds, similar to the requirements for chlorination, thus larger basic systems are not required. Lamp aging affects the required exposure time however.

The deterioration of the serviceable life for germicidal lamps at 8000 hours may be as high as 40%. Thus, corrections for treatment capacity must be made to correct for deterioration.

The liabilities which have operated against it the most are: lack of a persisting residual, low level of equipment reliability in the face of poor industry standards and probably poor operating capability in the "typical" small plant, a previous industry-wide commitment to chlorination (momentum), and lack of efficient process designs.

24.3 Costs

Cost data for application of UV and other disinfection technologies in Latin America are virtually non-existent. Table 24.1 contains comparative costs for a wide range of applications to wastewater, mostly in the U.S.(6).

24.4 Availability

The sources of UV radiation are two classes - natural and artificial. The sun is the most important natural source of UV light. Since the maximum UV sensitivity of microorganisms and the UV emission of the low-pressure mercury vapor lamp are well matched, the nearly monochromatic low-pressure mercury lamp has prevailed as the dominant radiation source in research and practical applications.

UV has significant disinfection qualities. It has even been used successfully where STERILIZATION is required, in the pharmaceutical, cosmetic, and beverage industries for example.

TABLE 24.1

COST SUMMARY

Plant Size (mgd)	1	10	100
Capital Cost	\$K	\$K	\$K
Process			
Chlorine	60	190	840
Chlorine/SO_2	70	220	930
Chlorine/SO_2/aeration[b]	120	360	1,580
Chlorine/carbon	640	2,800	8,400
Ozone/air[a]	190	1,070	6,880
Ozone/oxygen[a]	160	700	4,210
Ultraviolet[a]	70	360	1,780
Bromine chloride	50	130	410
Activated Sludge	1,450	5,790	39,800
Disinfection Cost	¢Kgal.	¢Kgal.	¢Kgal.
Process			
Chlorine	3.49	1.42	0.70
Chlorine/SO_2	4.37	1.75	0.89
Chlorine/SO_2/aeration[b]	7.66	2.39	1.19
Chlorine/carbon	19.00	8.60	3.28
Ozone/air	7.31	4.02	2.84
Ozone/oxygen[a]	7.15	3.49	2.36
Ultraviolet[a]	4.19	2.70	2.27
Bromine chloride	4.52	3.04	2.65
Activated Sludge	55.90	20.20	14.00

[a]Tertiary treatment stage is not included in these costs.
[b]Aeration is not required following dechlorination by SO_2 because a properly designed system will not remove any DO in the effluent.

Source: Reference 6

Application of UV for disinfection of potable water supplies however, has achieved little if any success.

The use of UV for disinfecting wastewater has realized greater success, largely as a result of EPA funded research under plant conditions. The use of UV for highly treated effluents appears to hold some promise. Use of ozone together with UV may hold promise also. See Ozone section.

There are 51 facilities in operation in the U.S., most below 1.5 MGD (19,000 m^3/day) wastewater plant size. About 30 are under construction, and another 30 under design (18).

24.5 Operation and Maintenance

Operation depends on continuous availability of power (so does most of the plant operation). An alternate power supply for the UV system must be provided to provide dependable continuous disinfection of a water supply.

Proportioning dose to flow is accomplished by changing the banks of lights exposed to the varying quantity to be treated. Wastewater quality absorbance should be checked continuously (best done with a "slave" UV lamp so that comparisons with the operating system are appropriate).

Unless the quartz ultraviolet system is disinfecting an ultra-pure water, the quartz sleeves readily foul with suspended and dissolved matter in the water. Therefore, it is necessary to employ a technique to remove the fouling matter on an almost continuous basis to preserve the high UV transmittance of quartz and the disinfection capability of the system.

Two non-chemical cleaning methods have been used with limited success. The first is a mechanical wiper system, in which a wiper periodically scrapes fouling deposits off of the outer surface of the quartz sleeves. For this technique to work effectively, very close tolerances are required on quartz sleeve outer diameter and alignment. Close tolerances are also required on the wiper system as well. These severe tolerance requirements add to manufacturing expense and are very difficult to achieve with large tube bundles.

Misalignment or improper tolerance control will cause reduced ultraviolet intensity in some portions of the unit, and a lower than expected dosage. Supplemental chemical cleaning is often required on wastewater disinfection systems.

Another non-chemical cleaning system used on quartz ultraviolet units is high frequency ultrasound. Ultrasonic cleaning is a new technique and has not been used on large capacity systems over an extended time period. This method is based on the same principle that is used for cleaning laboratory glassware. Whether or not the following potential operational problems exist with this technique will only be answered by construction of a large system and by longer term operation. The potential problems are:

1. Deleterious effects of ultrasound on the viability of quartz sleeves, germicidal lamps, and o-ring end seals.
2. Inadequate penetration of the ultrasound into the interior portion of large quartz tube bundles.

Field testing has shown that ultrasonic cleaning has had to be supplemented with frequent chemical cleaning.

Expected lives of lamps are variable, normally ranging from 7,000 to 12,500 hours. It is good practice, however, to replace lamps every 10 months, or when metered UV intensity falls below acceptable values. A complete cleaning of quartz glass enclosures with alcohol is required during lamp replacement. Based on limited operational experience, it is estimated that 10 to 12 man-hr per yr are required to maintain the UV system. Power requirements for the UV system for design flow rates up to 4 gpm (0.25 l/sec) are approximately 1.5 kWh/day.

24.6 Control

Giardia cyst inactivation is a function of UV energy absorption and thus depends on the amount of UV light that reaches the cyst and the time of exposure. Where commercial UV unit permits short-circuiting, the time for 100 percent inactivation can be expected to be protracted.

24.7 Special Factors

Size and morphological characteristics of organisms and particles appear to be very important factors in shielding them from UV radiation.

Though the laser-generated UV radiation has a considerably greater intensity than the mercury-vapor UV lamps, the detention time for the laser pulse is on the order of 10 nano-seconds. Thus, equivalent dose ranges can be obtained from both sources. Data suggest that the commercial UV units are much more effective than the excimer laser unit in inactivation of Giardia cysts.

24.8 Recommendations

It is preferred to have an operating disinfection system to having none. If power supplies are expected to be more reliable than chemical supplies, UV and other disinfection technologies are preferred. Most operating systems at treatment plants depend on power and power supplies may be more consistent. Operating UV systems are in operation in Brazil for both wastewater and water supply applications. Serious consideration should be given to chlorination alternatives.

SIMPLIFIED WATER TREATMENT PLANTS
INTRODUCTION TO SECTIONS 25 AND 26

Simplified water treatment plants may be considered as a single unit rather than individual treatment process technologies. There is sufficient experience in Latin America and the Caribbean to present combined simplified plants. A detailed review has been made of such plants (21, 22). The technologies presented in this handbook may be used in conjunction with the unit processes in the "simplified treatment plant" approach, or as replacements. The combined water treatment plant approach was also used in the cost model developed for Latin America and the Caribbean (11). The processes which are suggested for a simplified approach are:

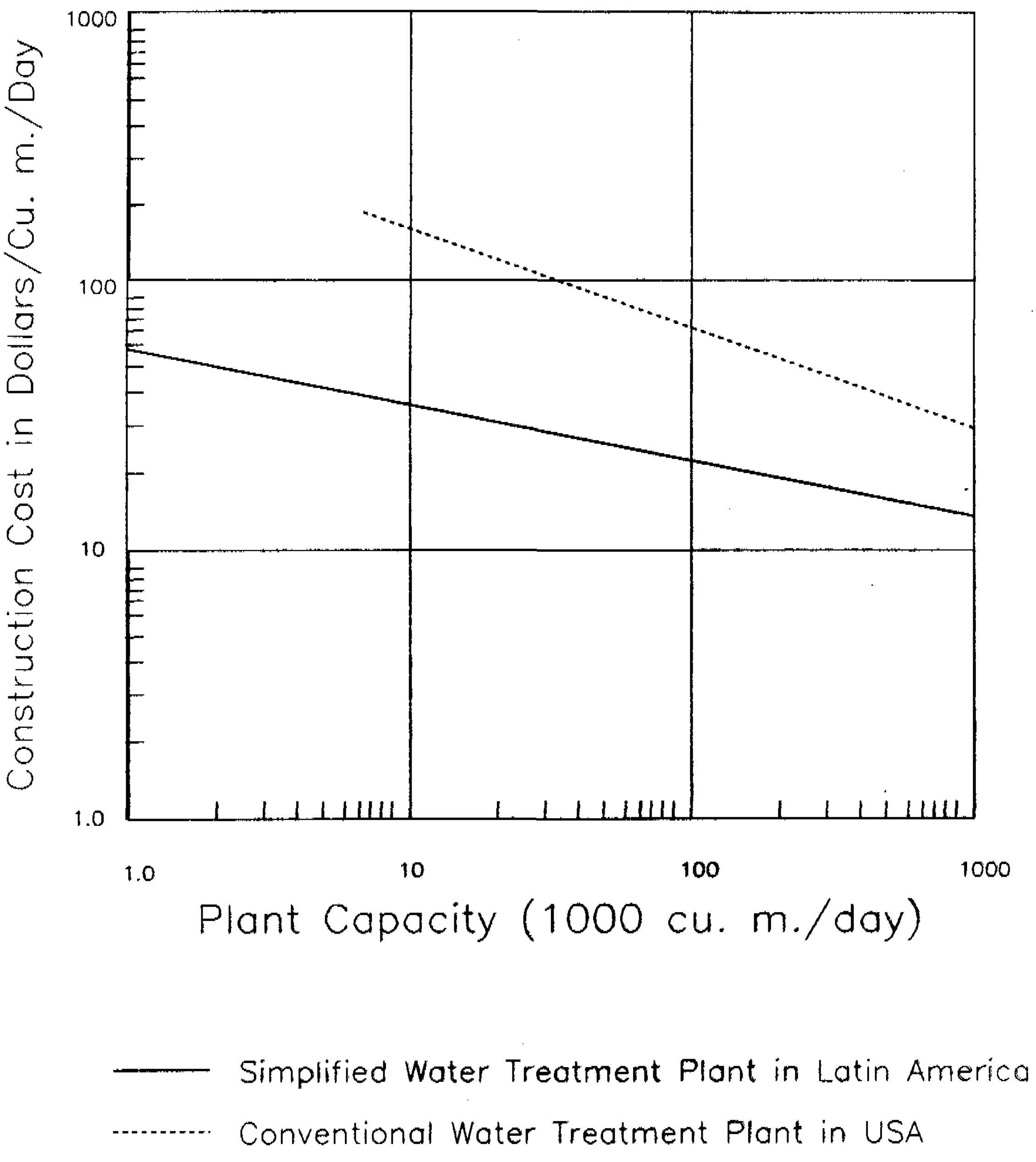
CHEMICAL DOSING, MIXING, AND FLOCCULATION - See Sections 2A and 8.

SEDIMENTATION - Using largely plate settlers because of improved efficiency over classical designs. See Section 25.

DUAL MEDIA FILTRATION - See Section 3. Declining rate filtration is suggested. Filtration rate is adjusted as the filtration process proceeds, rather than remaining constant. In actual practice the rate declines as the head loss increases. Declining rate operation allows the filtration rate to adjust to the degree of cleanliness of individual units. The ratio of maximum to average flow may be as high as 2. Brazil requires 1.3, and practice in the U.S. is around 1.5.

Costs for simplified plants are presented in Figure 25.3. There are observed reductions possible with the approach outlined above.

Figure 25.3: Construction Cost Comparison Between Conventional Water Treatment Plant and Simplified Water Treatment Plant in Latin America.



(Source: Ref. 21 & 22)

(File: Mart159)

TABLE 25.4

COST OF SIMPLIFIED WATER TREATMENT PLANTS IN LATIN AMERICA

Location of Treatment Plant (City, Country)	Rate Capacity X1000 gpd	Rate Capacity X1000 m³/d	(X1000) Construction Cost (Adjusted to $US 1988)	Unit Cost in $US gpd	Unit Cost in $US m³/d
Columbia					
Filandia	571	2.16	188	14500	55
Rio de Oro	571	2.16	68	8300	32
Paz del Rio	571	2.16	160	20000	74
Manaure	571	2.16	127	16000	60
Circasia	914	3.46	126	9600	36
Pailitas	914	3.46	200	15000	57
Becerril	914	3.46	140	11000	41
Santa Fè	914	3.46	178	13600	52
Girardota	1028	3.89	290	20000	75
Abrego	1028	3.89	115	7700	29
Mompos	1598	6.05	165	7100	27
Andalucia	1963	7.48	290	10000	39
Chiquinquirà	2830	10.71	459	11400	43
La Paz San Diego	3197	12.10	445	9600	36
Zarzal	3424	12.96	592	12000	46
Florencia	5478	20.74	452	5900	22
Pereira	13696	51.84	1000	5200	20
Manizales	18261	69.12	1700	6500	25
Barranquilla	22827	86.40	3400	10500	40
Cali	22287	86.40	2800	8300	32
Brazil					
Prudentopolis	264	1.00	70	18000	69
Paranà	528	2.00	123	16000	61
Paranà	661	2.50	84	9000	34
Paranà	1585	6.00	250	11000	41
Paranà	3170	12.00	400	8700	33
Paranà	4491	17.00	374	5900	22
Paranà	6605	25.00	425	4700	18
Paranà	8454	32.00	510	4300	16
Paranà	11361	43.00	760	4600	18
Paranà	11889	45.00	636	3700	14
Aracayu	17120	64.80	1600	6500	25
Paranà	26420	100.00	1275	3400	13
Bolivia					
Cochabamba	5284	20.00	515	6800	26
Chile					
Santiago	91307	345.60	5900	4600	18

Source: Reference 21

25. PLATE SETTLING

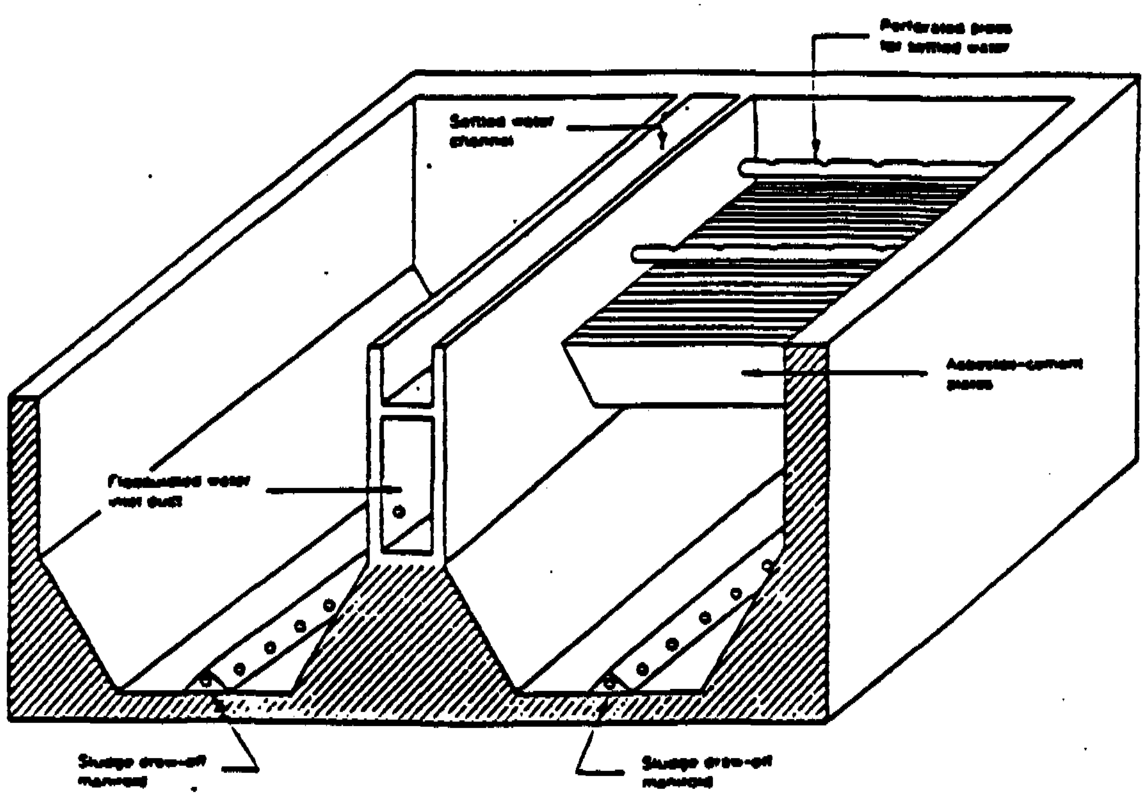
25.1 Description

Plate settlers are used in Latin America not only for new plants, but also for upgrading existing facilities. They are more economical to build, produce better effluent quality than horizontal sedimentation tanks, and are more stable and reliable. Plate settlers essentially consist of a series of parallel trays inclined at a steep enough angle to stimulate self-cleaning of the trays (See Figures 25.1 and 25.2). The water flows upward while the sludge slides toward the bottom of the tank where it is concentrated and then manually or automatically removed. Because the flow velocity near the plates is almost zero, the particles that fall on them are not subject to drag forces and can easily move in an opposite direction to the main water flow.

The design of the sedimentation basin with plates is governed by three basic criteria: 1) The quantity of water to be treated 2) Selected detention time, and 3) The selected surface loading rate. These three criteria and validity ranges are given in Tables 25.1 and 25.2 The asbestos - cement and wood plates are most common usage in South and Central America, because of their low weight and low cost. The standard plate is 4 ft x 8 ft (and 0.2 to 0.3 inch thickness. Plates of this type can handle without damage, a concentrated load of 176 lb at the center. These plates are installed 7.5 ft space apart (See Figure 25.3).

The following equation, which is derived from geometrical considerations and laboratory performance studies, can be used for designing inclined-plate and tube settlers. Refer to Figure 25.2.

Figure 25.1: Plate Settler with Longitudinal Hoppers at the Bottom and an Inlet Water Manifold.



(Source: Ref. 21)

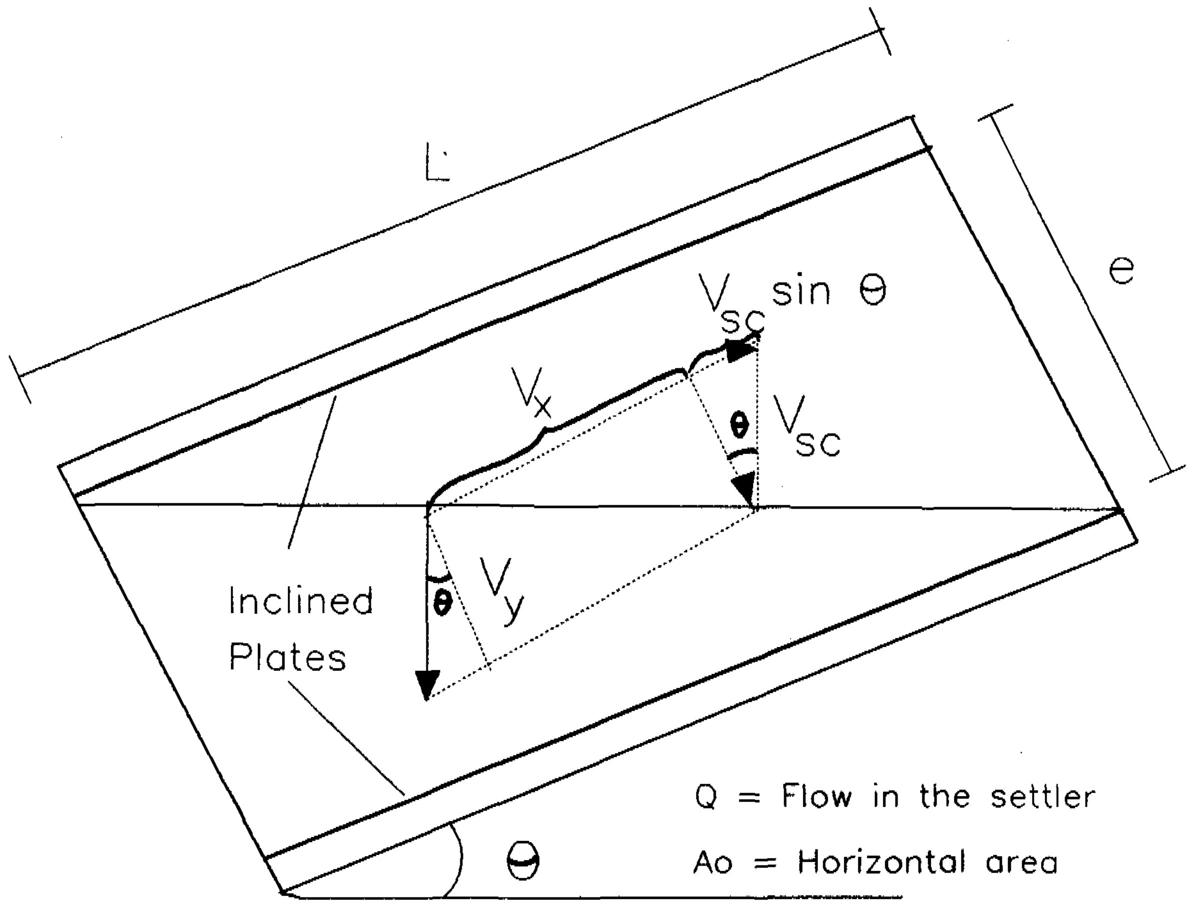


Figure 25.2: Geometrical relationships of an Inclined-Plate Settler.

(Source: Ref. 22)

TABLE 25.1

AREA FACTOR

Angle	Useful Relative Depth L L-0.013N 40 ft (12 m)	53 ft (16 m)	67 ft (20 m)
0	12.00	16.00	20.00
15	11.84	15.71	19.58
30	10.89	14.36	17.82
45	9.19	12.02	14.84
60	6.86	8.85	10.86
75	4.07	5.11	6.14
90	1.00	1.00	1.00

Source: Reference 21

TABLE 25.2

VELOCITIES AND SURFACE LOADS USED IN PILOT SETTLER

	Flow		Surface Load		Velocity Under the Plates	
Run Number	cfs	m^3/s	gpm/sq ft	m/h	m/min	cm/s
1	11,405	323	5	12.25	1.02	1.7
2	8,298	235	3.6	8.9	0.74	1.24
3	6,850	194	3.0	7.3	0.72	1.02
4	5,826	165	2.5	6.0	0.5	0.82

Source: Reference 21

$$v_{sc} = \frac{Q}{A_o f} = \frac{Q}{A}$$

where v_{sc} = critical surface loading rate, or settling velocity (m/day)
f = sin θ--L_u cos θ-
A = surface area of conventional horizontal settling tank (m^2)
A_o= surface area of high rate settling (m^2)

Hence, for a unit flow Q:

$$A_o = \frac{A}{f}$$

The f factor then becomes the number of times the area of a horizontal settling tank must be divided to obtain the area of an inclined-plate or tube settler. For example, for an effectrive relative depth L_u = 20, and angle θ = 60; the area factor f= 10.9.

Recommended surface loading rates for horizontal-flow sedimentation basins equipped with inclined-plate or tube settlers are listed in Table 25.3 for two categories of raw water turbidity; 0 to 100 NTU and 100 to 1000 NTU. These loadings apply specifically to warm water areas (temperature nearly always above 10°C) and apply to most developing countries. For efficient self-cleaning, tubes or inclined plates are usually arranged at an angle of 40 to 60° to the horizontal. The most suitable angle for a particular design depends on the sludge characteristics of the water being treated, usually 55° above the horizontal. The distance between parallel-inclined plates or, similarly, the diameter of settling tubes, is about 5 cm. The passageways formed by the plates, or inside the tubes, are commonly about one meter long. Care needs to be exercised in the design of outlet collection systems to assure even distribution of flow through plate or tube modules. This is more easily done with overflow weir outlets than with submerged launders.

TABLE 25.3

LOADING FOR HORIZONTAL-FLOW SETTLING BASINS EQUIPPED WITH INCLINED-PLATE OR TUBE SETTLERS IN WARM-WATER AREAS (ABOVE 10°C)

Settling Velocity Based on Total Clarifer Area (m/day)	Settling Velocity Based on Portion Covered by Plates (m/day)	Probable Effluent Turbidity (NTU)
(A) Raw Water Turbidity 0 to 100 NTU		
120	140	1 to 3
120	170	1 to 5
120	230	3 to 7
170	200	1 to 5
170	230	3 to 5
(B) Raw Water Turbidity 100 to 1000 NTU		
120	140	1 to 5
120	170	3 to 7

Source: Reference 48.

Local materials and labor may be used for the construction of inclined-plate or tube settlers. For included-plate settlers, the individual trays can be fabricated from polyethylene (or similar type of plastic) or of wood. Asbestos-cement plates should be coated with plastic or similar type of protective covering because of their susceptibility to corrosion from alum-treated water. Where wood is used on law slopes, trays are commonly 30 cm apart. It may also be necessary to drain the tank for cleaning occasionally, because sludge does not readily slide down wooden trays while the basin is in service. Wood trays have worked successfully with frequent cleaning. Tube settlers, on the other hand, are easily fabricated from PVC pipes (3 to 5 cm internal diameter), which are packed closely together to form a module. In countries with indigenous plastics industries, commercially available tube modules that are prefabricated at the factory are suitable for larger installations. Plastic tube settler inserts are manufactured in Brazil.

25.2 Limitations

The capacity of the sludge hoppers at the bottom of the tanks must be considered so that too frequent withdrawal of the sludge can be avoided. The main factor is the volume of sludge produced. This value can be estimated with the following expression (21):

$$V = Q(K1*D+K2T)/100$$

Where V = Volume of sludge per day

Q = Water flow

D = Optimal dosage of coagulants in grams per cubic meter

T = Water turbidity

K1= Coefficient that varies between 0.015 and 0.025

K2= Coefficient that varies between 0.004 and 0.0001.

This equation is valid for turbidities ranging from 100-800 NTU and coagulant dosage of 15-60 mg/l.

For the worst possible raw water conditions, hoppers should be emptied a maximum of once per hour.

25.3 Availability

The plate settler is widely used in developing countries. It is available with different modifications to suit site specific problems and get the most cost-effective result. For example the cost of covering a settler with plates is between half to three fourths the cost of installing plastic tube modules. In addition, the efficiency of asbestos - cement or wood plates is higher because of their better length-to-width relationship and their better hydraulic characteristics.

25.4 Operation and Maintenance

Plates are installed at one end of the tank allowing easy access for removal and cleaning. Regardless of the steepness of the plate angle, it is necessary to remove the plates and clean them on a regular basis. Algae growth on the plates can be a troublesome problem, especially in warm climates.

25.5 Control

To obtain equal distributions through all the parts, water must flow by gravity. To equilibrate the flow, the discharge of the laterals in the central channel must be free.

25.6 Special Factors

In both the inlet and outlet systems, the spacing between the materials should not be more than 5 ft (1.5m), and troughs or pipes must be installed over the entire surface of the tank.

(This helps achieve uniform distribution together with the installation of a central channel with lateral troughs or lateral perforated pipes.)

25.7 Recommendation

Plate settlers could be used for updating and plant expansion. Land area is generally not limited in Latin America, but if conventional plants are built initially, the use of plate settlers would allow plant capacity expansion by 50% to 150% (25).

26. CHEMICAL STORAGE AND DOSING, MIXING AND FLOCCULATION

26.1 Description and Operations

Chemical dosing (see also section 33 on chemical addition) is done with simple gravity feeders, using only two-story buildings and applying the coagulants in non-concentrated solution from large storage tanks. Because liquid alum is not usually available, the solution is prepared from solid alum (lumps). No dry feeders or dosing pumps are included, and chemical storage facilities are located on the second floor to avoid vertical transport of materials. Trucks unload materials directly on this floor by means of ramps if necessary. In larger plants, such as the 24 m3/sec Los Berros plant in Mexico City, the chemical storage and solution tanks are placed on the first floor, and the chemical solution is centrifugally pumped to a small constant level elevated dosing tank from which the solution flows by gravity (21).

Mixing is always done hydraulically, usually with Parshall flumes and Creagers weirs (i.e. a round-crested overflow spillway). Parshall flumes have the additional advantages of allowing simultaneous mixing and flow measurement.

Flocculation systems are commonly constructed with hydraulic drives for small treatment plants and mechanical drives for large treatment plants. See Figure 26.1 (21).

Mechanical flocculation is more versatile than hydraulic flocculation and easily allows for an increase or decrease in the velocity gradient and thus the mixing intensity. It does, however, induce short circuiting. Hydraulic flocculators produce almost pure piston flow.

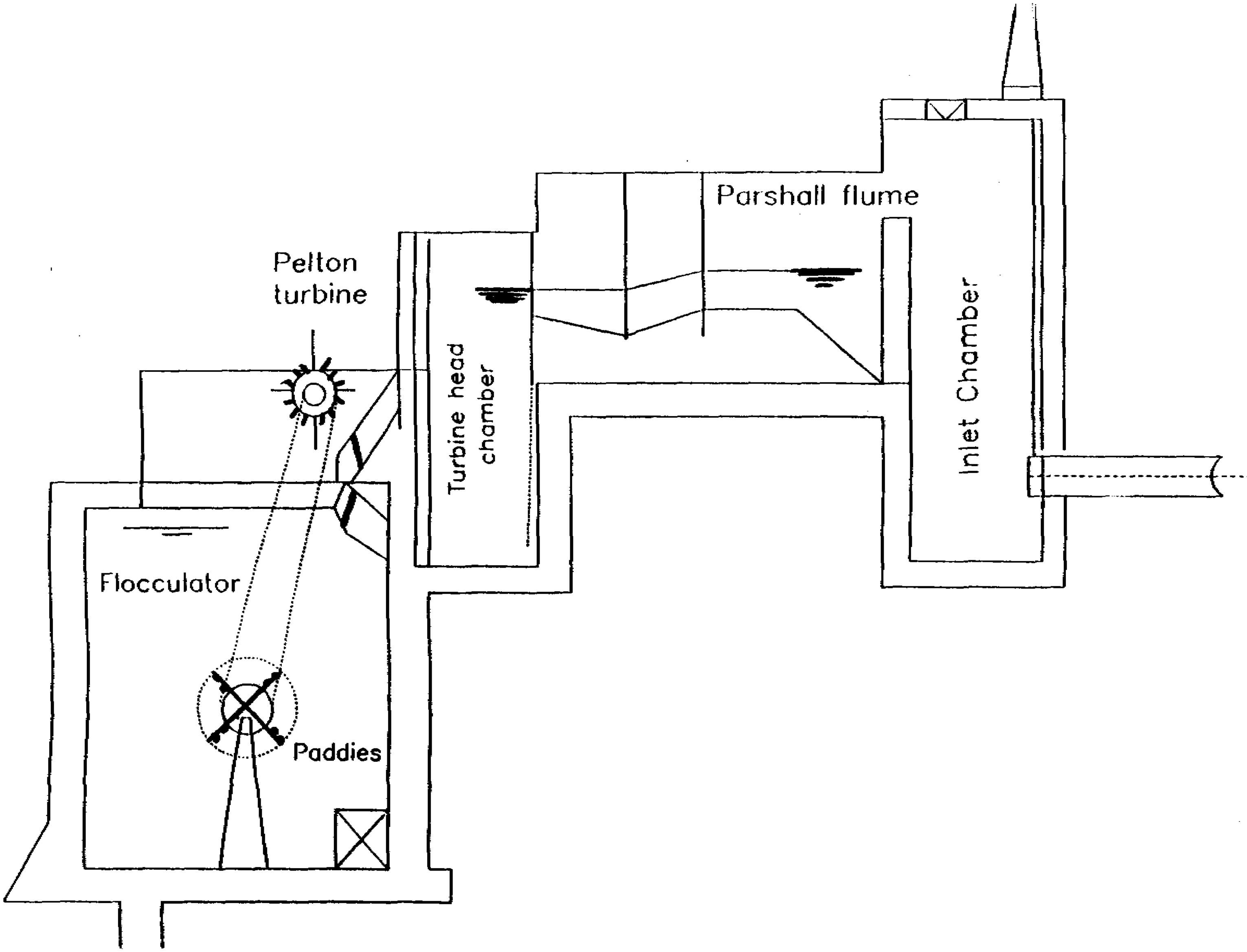


Figure 26.1: Hydromechanical flocculators

(Source: Ref. 21)

26.2 Availability

This technology is widely used in developing countries specially in South America.

26.3 Operation and Maintenance

The treatment process highly depends on the energy input and results can vary in terms of chemical treatment, water temperature, and water characteristics.

26.4 Special Factors

In some cases a hydromechanical flocculator has been used. A Pelton turbine driving a conventional horizontal paddle is introduced into the raw water inflow, inducing rapid mixing at the same time. The head used by the turbine is only 4 ft and can produce velocity gradients up to 60 sec-1. This system can be used only in gravity-fed systems, as shown in Figure 26.1.

26.5 Recommendations

This treatment process is very attractive in terms of cost savings. It can be widely used in Latin America and the Caribbean.

27. LAND APPLICATION OF WASTEWATER BY IRRIGATION

27.1 Description

Land application of wastewater is a potential technique for treatment of municipal wastewater. The technology may be used where an available irrigation site has suitable soil conditions and ground water hydrology, and climate is favorable. Hundreds of efficient systems are currently operating in regions with limited water resources to increase the growth of grass, crops and forests. In addition to this, the natural top soil and soil biota provide filtering and stabilization of the organic matter, nutrients are used by the plants (Figure No. 27.1).

The wastewater is applied by sprinkling to vegetated soil that is moderate to high in permeability (sandy loam to sand gravel mixed loam) and is treated as it travels through the soil matrix by filtration, absorption, ion exchange, precipitation, microbial action and also by plant uptake. Sprinklers are categorized as hand moved, mechanically moved, and permanent set. The selection of sprinkler includes the following considerations: field conditions (shape, slope, vegetation, and soil type), climate, operating conditions, and economics. Vegetation is a vital part of the process and serves to extract nutrients, reduce erosion, and maintain soil permeability.

The renovated water, after passing through the soil filter, may be collected by a drainage system and becomes available for reuse within the groundwater context. In regions with limited water resources, land application by the irrigation process can improve the growth of many crops.

The design of land application of wastewater by irrigation is governed by field area, application rate, BOD_5 loading, soil depth, and crop selection. The general design considerations for land treatment approaches are given in Tables 27.1 and 27.2 (4,

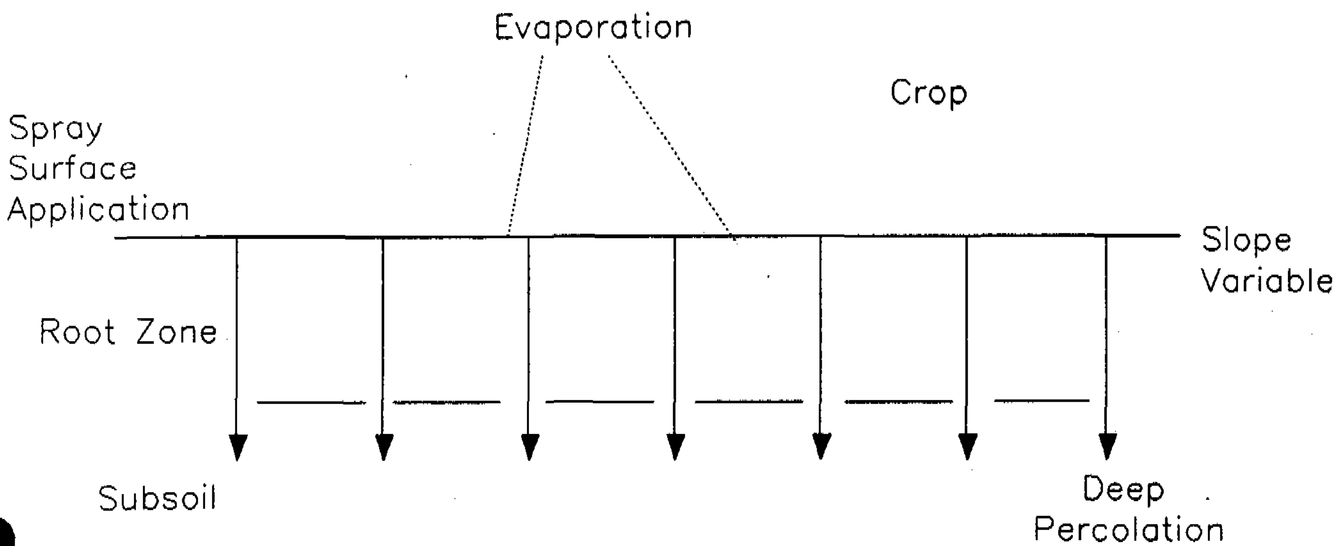


Figure 27.1: Land Application of Wastewater by Irrigation

(Source: Ref. 23) (File:Martin37)

TABLE 27.1

COMPARISON OF DESIGN FEATURES FOR ALTERNATIVE LAND-TREATMENT PROCESSES

Feature	Irrigation	Rapid Infiltration	Overland Flow	Wetland Application	Subsurface Application
Application techniques	Sprinkler or surface[a]	Usually surface	Sprinkler or surface	Sprkinler or	Subsurface piping
Annual application rate, m	0.6-6.0	6-120	3-20	1-30	2-25
Field area required, ha[b]	22-226	1-22	10-44	4-113	5-56
Typical weekly application rate, cm	2.5-10	10-210	6-15[c]	2.5-60	5-50
Minimum preapplication treatment provided	Primary sedimentation[e]	Primary sedimentation	Screening and grit removal	Primary sedimentation	Primary sedimentation
Disposition of applied wastewater	Evapotranspiration and percolation	Mainly percolation	Surface runoff and evapotranspiration with some percolation	Evapotranspiration, percola- and runoff	Percolation with some evapotranspiration
Need for vegetation	Required	Optional	Required	Required	Optional

[a] Includes ridge and furrow and border strip
[b] Field area in hectares not including buffer area, roads, or ditches for 0.044 m^3/s (1Mgal/d) flow.
[c] Range for application of screened wastewater.
[d] Range for application of lagoon and secondary effluent.
[e] Depends on the use of the effluent and the type of crop.

Note: cm x 0.3937 = in
m x 3.2808 = ft
ha x 2.47111 = acre

Source: Reference 4

TABLE 27.2

COMPARISON OF SITE CHARACTERISTICS FOR LAND-TREATMENT PROCESS

Characteristics	Irrigation	Rapid infiltration	Overflow	Wetland Application
Climatic restrictions	Storage often needed for cold weather and precipitation	None (possibly modify operation in cold weather)	Storage often need for cold weather	Storage may be needed for cold weather
Depth to groundwater, m	0.6-0.9 (minimum)	3.0 (lesser depths acceptable where underdrainage provided	Not critical	Not critical
Slope	Less than 20% on cultivated land; less than 40% on noncultivated land	Not critical; excessive slopes require much earthwork	Finish slopes 2-8%	Usually less 5%
Soil permeability	Moderately slow to moderately rapid	Rapid (sands, loamy sands)	Slow (clays, silts, and soils with impermeable barriers)	Slow to moderate

Note: m x 3.2808 = ft

Source: Reference 4

17). Expected effluent quality is shown on Table 27.3. Irrigation should be compared with other land treatment approaches (see Figure 27.2).

27.2 Limitations

Land application is limited by soil type and depth, topography, underlying geology, climate, crop selection and land availability. Crop water tolerance, nutrient requirements, and the nitrogen removal capacity of the soil-vegetation complex limit hydraulic loading rate. Land slopes should be less than 15 percent to minimize runoff and erosion. Pretreatment for removal of solids and oil and grease serves to maintain reliability of sprinklers and to reduce clogging.

27.3 Costs

Typical cost ranges are shown in Table 27.4 and cost comparison of capital and operating costs for irrigation. Overland flow, and infiltration-percolation systems are shown in Table 27.5.

27.4 Availability

This technology has been widely used and successfully utilized for more than 100 years because this treatment process is less expensive and more cost-efficient than conventional treatment processes yielding water of similar quality. The application depends on availability and cost of land.

27.5 Operation and Maintenance

Proper and regular maintenance of sprinklers is essential. It consists of cleaning of spray nozzles, periodic checks of sprinkler piping and drain outlets, and pumping valves.

TABLE 27.3

COMPARISON OF EXPECTED QUALITY OF TREATED WATER FROM LAND-TREATMENT PROCESSES, MG/L

Constituent	Irrigation[a]		Rapid Infiltration[b]		Overland Flow[c]	
	Average	Maximum	Average	Maximum	Average	Maximum
BOD	<2	<5	2	<5	10	<15
Suspended solids	<1	<5	2	<5	10	<20
Ammonia nitrogen as N	<0.5	<2	0.5	<2	0.8	<2
Total nitrogen as N	3	<8	10	<20	3	<5
Total phosphorus as P	<0.1	<0.1	1	<5	4	<6

[a] Percolation of primary or secondary effluent through 1.5 m of soil.

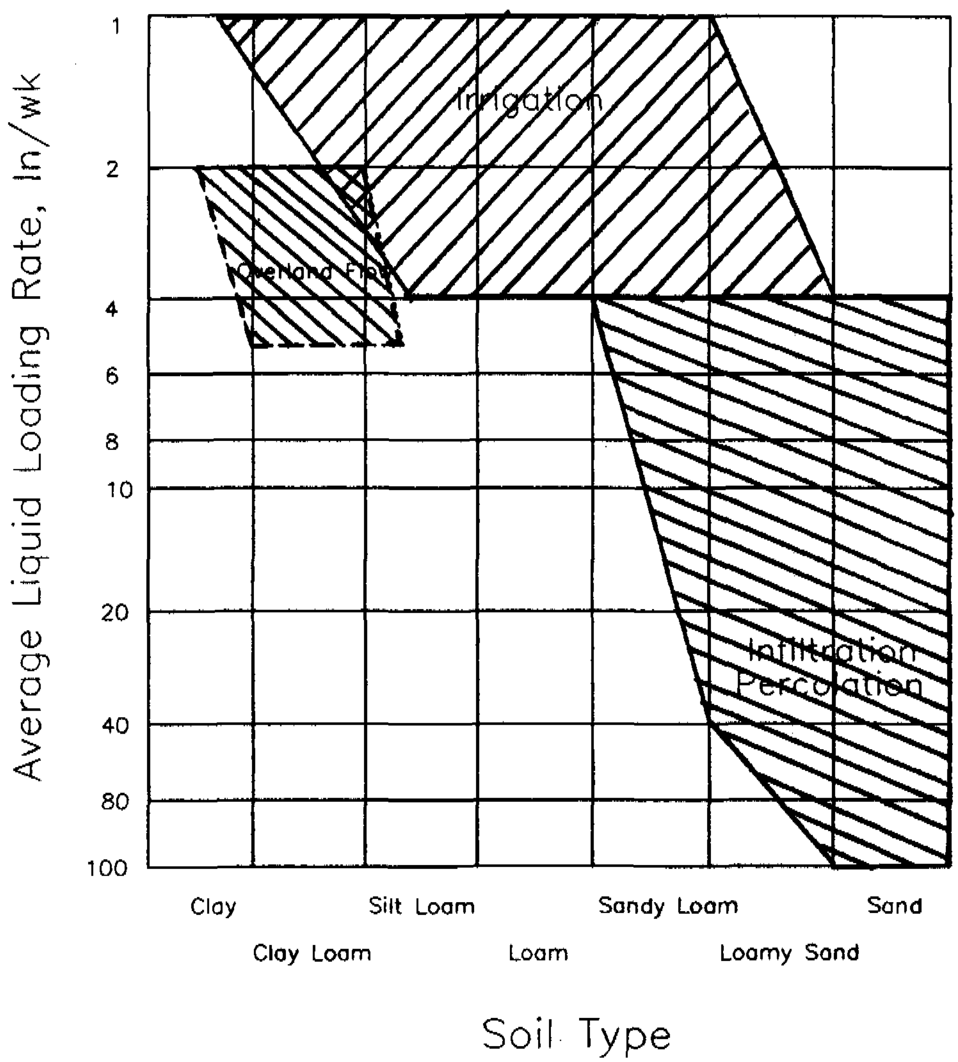
[b] Percolation of primary or secondary effluent through 4.5 m of soil.

[c] Runoff of comminuted municipal wastewater over about 45 m of slope.

Note: m x 3.2808 = ft

Source: Reference 4

Figure 27.2: Soil Type VS. Liquid Loading Rates for Different Land Application Approaches.



(Source: Ref. 23)

TABLE 27.4

TYPICAL COST RANGES

LAND APPLICATION OF WASTE WATER BY IRRIGATION[a]
(Spray Irrigation Systems)

(All Costs Updated to 1984 $)

	.01 MGD Treatment Capacity		.1 MGD Treatment Capacity	
	Capital Cost ($/gpd)	Operating Cost ($/1000 gal.)	Capital Cost ($/gpd)	Operating Cost ($/1000 gal.)
Spray Irrigation System[b]	5.25 - 10.50	.88 - 1.75	1.75 - 5.25	.18 - .50

[a] Cost information based on data presented in the following publications:

1. U.S. EPA (1980) Innovative and Alternative Technology Assessment Manual, MCD-53.
2. U.S. EPA (1980) Construction Costs for Municipal Wastewater Treatment Plants, FRD-11.
3. U.S. HUD (1977) Package Wastewater Treatment Plant Descriptions, Performance, and Cost.

[b] Low rate (.5-4 inches per week) application rate; not including cost of pretreatment facilities, storage lagoons, and land.

Source: Reference 30

TABLE 27.5

COMPARISON OF CAPITAL AND OPERATING COSTS
COSTS FOR ONE-MGD SPRAY IRRIGATION, OVERLAND FLOW,
AND INFILTRATION-PERCOLATION SYSTEMS

Cost Item	Spray Irrigation	Overland Flow	Infiltration-Percolation
Liquid loading rate, inc./wk	2.5	4.0	60.0
Land used, acres	103	64	--
Land required, acres	124	77	5
Capital costs			
Land @ $500/acre	109,000	67,000	4,400
Earthwork	18,000	112,000	17,500
Pumping station	88,000	87,500	--
Transmission	230,000	233,000	230,000
Distribution	250,000	112,000	8,800
Collection	--	11,000	52,500
Total Capital Costs	700,000	620,000	314,000
Capital cost per purchased acre	5,600	8,000	63,000
Amortized cost	65,000	61,000	34,000
Capital cost, ¢/1,000 gal	17.7	16.7	9.3
Operating Costs			
Labor	17,500	17,500	13,000
Maintenance	34,000	21,000	6,100
Power	10,000	10,000	3,200
Total Operating Costs	62,000	49,000	22,400
Operating Cost, ¢/1,000 gal.	16.8	13.3	6.1
Total Cost, ¢/1,000 gal	34.4	30	15.4

Source: Reference 29

27.6 Control

The major concerns and controls in the management of land application of wastewater should be based on the following factors: land availability, site locations, the climate, soil and subsurface conditions, the wastewater characteristics, hydraulic loading, the capacity and utilization of the plant-soil complex to produce a specific water quality, the intended use or reuse of the wastewater, and the ultimate use of the land.

27.7 Special Factors

Toxic substances which are not likely to be removed in the renovation process must be controlled in irrigation water or reduced by pretreatment. Extensive soil water and ground water pollution is more difficult to correct than surface water pollution, and once present, such pollutants can persist for generations within the subsurface.

27.8 Recommendations

Knowledge of wastewater characteristics, treatment process mechanisms, and public health requirements are fundamental to the successful design and operation of the land application of wastewater treatment by irrigation.

Land treatment should be practiced regularly in Latin America. Land is typically available at low cost. Land is available even near large population centers. Also, with water shortages in many areas, land treatment of wastewater can be a very important water recovery technology. Land treatment can be used in a direct (with follow-on treatment) or indirect (groundwater recharge) water reuse mode. Increased consideration could be given in most waste treatment alternative studies so that a detailed examination of the option can be made.

28. LAND APPLICATION OF WASTEWATER BY OVERLAND FLOW

28.1 Description

Overland flow is a land treatment process in which wastewater is applied at the upper end of sloped vegetated terraces and allowed to flow down the terraces in sheet flow to a series of runoff collection ditches. Figure 28.1 (2) is a schematic of the overland flow process. The terraces are constructed on tight, nearly impermeable soils and planted with a mixture of grasses.

The wastewater is renovated by a combination of physical, chemical, and biological processes before reaching the toe of the terrace where it is collected in runoff channels and discharged or recovered. The overland flow can be also used to purify secondary effluents such as flows from an oxidation pond (ditch) or to provide secondary treatment. Under the right conditions it could be used for primary treatment. Table 28.1 provides design parameters of the overland flow process (29). Additional design considerations are given in Table 28.2 (30).

28.2 Limitations

The vegetation on the terraces is very important to achieve the desired smooth sheet flow down the terraces and also to protect the terraces from erosion. Therefore a water-tolerant, tuff grass is required. The most commonly used grasses in warm climates have been Bermuda, Reed canary grass, mixture of several grasses.

28.3 Costs

Typical cost range of construction cost and operations and maintenance cost are shown in Tables 28.3 and 28.4 (18, 19).

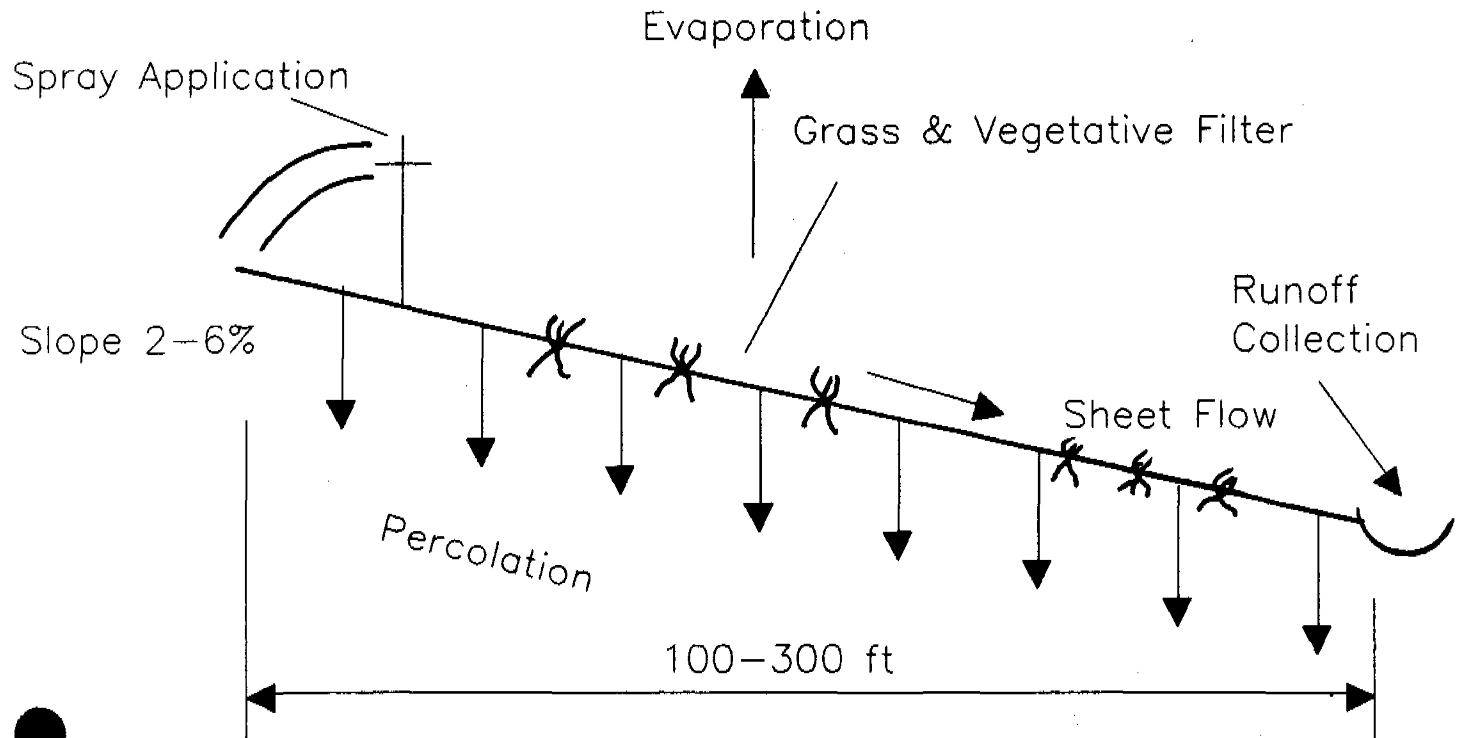


Figure 28.1: A Schematic Diagram of Overland Flow Treatment

(Source: Ref. 2)

TABLE 28.1

SUGGESTED OVERLAND FLOW DESIGN RANGES

Preapplication Treatment	Application Rate $m^3/h.m$	Hydraulic Loading Rate cm/d
Screening/Primary	0.07 - 0.12[a]	2.0 - 7.0[b]
Aerated Cell (1 day detention)	0.08 - 0.14	2.0 - 8.5
Wastewater Treatment Pond[c]	0.09 - 0.15	2.5 - 9.0
Secondary[d]	0.11 - 0.17	3.0 - 10.0

[a] = $m^3/h.m \times 80.5$ = gal/h.ft

[b] = cm/d X 0.394 = in./d

c = Does not include removal of algae

[d] = Recommended only for upgrading existing secondary treatment

Source: Reference 29

TABLE 28.2

OVERLAND FLOW DESIGN FEATURES

- Application Technique	o Sprinkler o Surface
- Process Objectives	o Wastewater Treatment o Crop Production o Augment Surface Streams
- Annual Application Rate	o 10 to 70 feet
- Field Area Required	o 16 to 110 acre/MGD
- Precipitation Treatment (minimum)	o Screening and grit removal
- Slope	o Finish Slope 2 to 8%
- Soil Permeability	o Slow (clay, silt, etc.)
- Depth to Groundwater	o Not critical
- Climatic restrictions	o Storage usually required during extreme cold weather
- Vegetation	o Grass crop required
- Fate of Applied Wastewater	o Surface runoff o Evaporation o Minimal Percolation

Source: Reference 30

TABLE 28.3

TYPICAL COST RANGES FOR OVERLAND FLOW SYSTEMS[a]
(All Costs Updated to 1984 $)

	10,000 gpd Capacity	100,000 gpd Capacity
Capital Cost[b] ($/gpd)	6.90 - 13.80	1.40 - 2.80
Operating Cost[b]	1.40 - 2.80	.35 - .70

[a] Based on cost data from the following sources:

1. U.S. EPA (1980), Innovative and Alternative Technology Assessment Manual, MCD-53.
2. U.S. EPA (1980), Construction Costs for Municipal Wastewater Treatment Plants, FRD-11.
3. U.S. HUD (1977), Package Wastewater Treatment Plant Descriptions, Performance and Cost.

[b] Complete system including disinfection and discharge; capital costs do not include land costs.

Source: Reference 19

TABLE 28.4

CAPITAL AND OPERATING COSTS
FOR ONE-MGD OVERLAND FLOW[a]

Cost Item	Overland Flow
Liquid loading rate, inc./wk	4.0
Land used, acres	64
Land required, acres	77
Total Capital Costs	620,000
Capital Cost, ¢/1,000 gal	16.6
Total Operating Costs	49,000
Operating Cost, ¢/1,000 gal	13.3
Total cost, ¢/1,000 gal	30

[a] Estimated for 1973 dollars, Engineering News-Record construction cost (ENRCC) index 1860 and sewage treatment plant construction cost (STPCC) index 192.

Source: Reference 18

28.4 Availability

Relatively new. Very few municipal plants in operation, and most are in warm, dry climates.

28.5 Operation and Maintenance

Operation of overland flow is basically a surface phenomenon, soil clogging is not a problem. High BOD and suspended solids removal will be achieved with the application of raw wastewater. However, poor design and mismanagement, particularly overloading of wastewater, can result in health risks, nuisance factors, subsurface water contamination, and the potential toxicity of metals buildup in the soils and changes in the soil structures. The success or failure of the process operation primarily depends on various loading rates, climate, soil type, and depth of groundwater.

The maintenance depends on the type and size of the effluent distribution system, the nature of the vegetation cover, and geographical conditions of the system.

28.6 Control

The major concern and controls in the management of overland flow application should be based on the following factors: land availability, site location, the climate, soil and subsurface conditions, various loading rates, wastewater characteristics, the capacity and utilization of the plant-soil complex to produce a specific water quality, the intended use or reuse of the wastewater, and the ultimate use of the land.

28.7 Special Factors

The concentration of potential toxic metals and trace elements which are not likely to be removed in the renovation process must be controlled in overland flow water. Extensive soil, water, and groundwater pollution is more difficult to correct than surface water pollution, and once present, such toxic pollutants persist for generations within the subsurface.

28.8 Recommendations

The overland flow process requires long term commitment of large land areas. Potential odor and vector problems exist, but careful design and operation is highly recommended to control them.

All land application technologies should be explored for use in Latin America. See the discussion in Section 27.

29. LAND APPLICATION OF WASTEWATER BY INFILTRATION-PERCOLATION METHOD

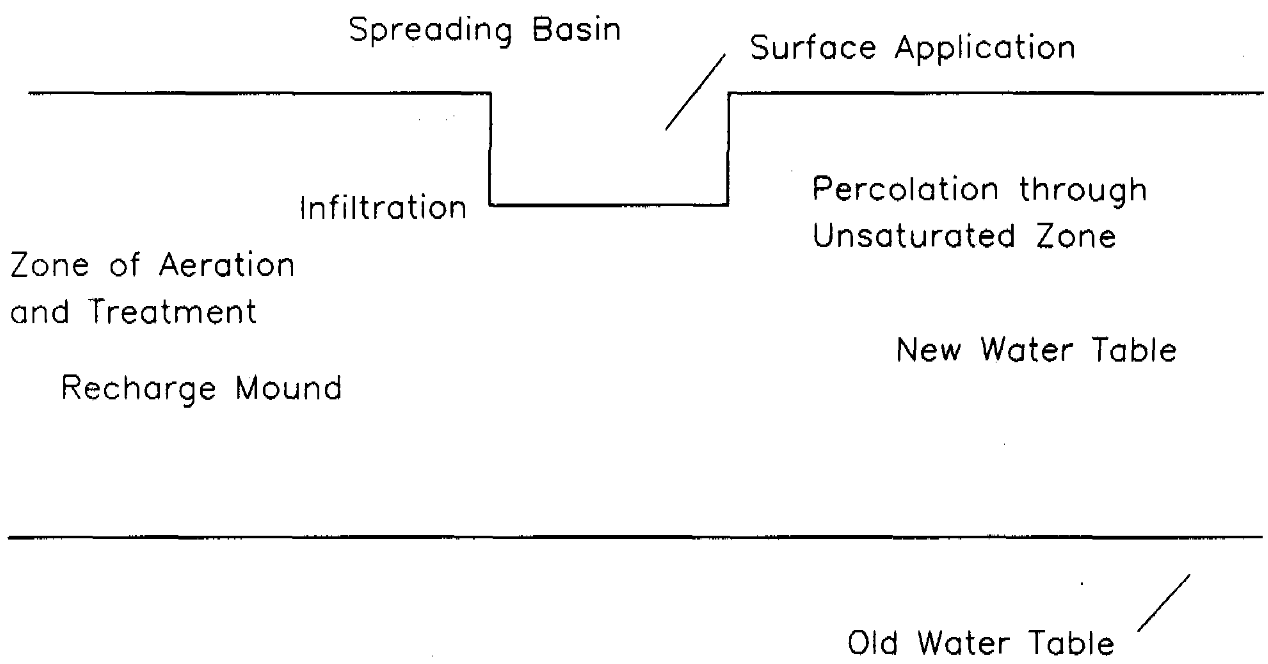
29.1 Description

In the Infiltration-Percolation process, sometimes referred to as Rapid-Infiltration, most of the applied wastewater percolates through the soil, and the treated effluent eventually reaches the groundwater. This process is the most applicable land treatment technology for indirect water reuse. The wastewater is applied to rapidly permeable soils such as sandy and loamy soils, by spreading in basins or by sprinkling, and is treated as it travels through the soil matrix. Vegetation is not usually used, but grass cover helps to remove suspended solids and organic matters.

The typical hydraulic pathway for this process is shown in the schematic view in Figure 29.1 (23). A much greater portion of the applied wastewater percolates to the groundwater than with irrigation. There is little or no consumptive use by vegetation, and there is less evaporation in proportion to a reduced surface area. The recovery of renovated water is accomplished by using underdrains or wells as shown in Figure 29.2 (4). The principal design parameters for infiltration-percolation are shown in Table 29.1 (2).

Spreading basins are constructed by removing the fine textured top soil from which shallow banks are constructed. The underlying sandy soil serves as the filtration media. Underdrainage is provided by using plastic or clay tile pipes. The distribution system applies wastewater at a rate which constantly floods the basin throughout the application period of several hours to two weeks. The spreading basin water drains uniformly away, driving air downward through the soil and fresh air from the above. A cycle of flooding and drying maintains the infiltration capacity of the soil material.

Figure 29.1: Schematic view of Land Application of Wastewater by Infiltration – Percolation Method.



(Source: Ref. 23)

(File: Martin70)

Figure 29.2: Recovery of Renovated Water by Wells

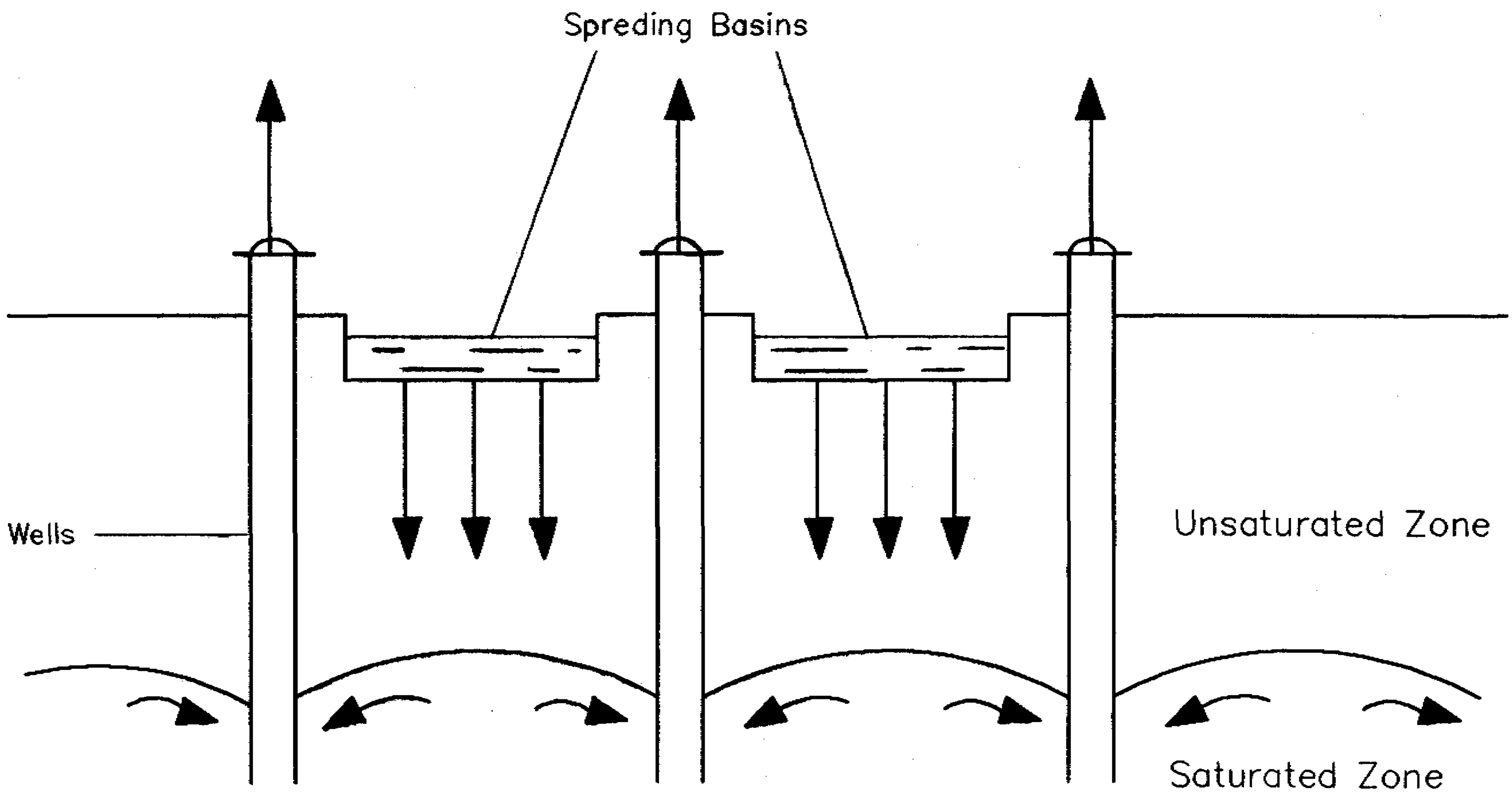


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TABLE 29.1

TYPICAL DESIGN PARAMETER FOR LAND APPLICATION OF WASTEWATER BY INFILTRATION - PERCOLATION METHOD

Parameter	Range Values
- Field Area	3 to 56 acres/Mgal/day
- Application rate	20 to 400 ft/yr 4 to 92 in/week
- BOD_5 loading rate	20 to 100 lb/acre/d
- Soil depth	10 to 15 ft or more
- Soil permeability	0.6 in/hr or more
- hydraulic loading cycle	9 hr to 2 weeks application period 15 hr to 2 weeks resting period
- Soil Texture	Sands, sandy loams
- Basin Size	1 to 10 acres, at least 2 basins/site
- Height of dykes	4 ft
- Underdrains	6 or more feet deep well or drain spacing site specific
- Application techniques	Flooding or sprinkling
- Preapplication treatment	Primary or secondary

Source: Reference 2

29.2 Limitations

Process is limited by soil type, soil depth, the hydraulic capacity of the soil, the underlying geology, and the slope of the land. Adverse conditions cause improper treatment results. Nitrate and nitrite removals are low unless special management practices are used.

29.3 Costs

The capital and operating costs for infiltration and percolation processes are given in Table 29.2 (1).

29.4 Availability

This process was developed approximately 100 years ago and has remained unaltered since then. It has been widely used for municipal and certain industrial wastewaters throughout the world.

29.5 Operation and Maintenance

This process is required to maintain wastewater spreading basins from clogging, due to suspended solids in the wastewater, therefore, occasional tillage of the surface layer is necessary.

29.6 Control

Preapplication treatment of sewage wastewater is essential to remove solids, which improves distribution system reliability, reduces nuisance conditions, and may reduce clogging rates. Common preapplication treatment practices include the following: primary treatment for isolated locations which restricted public access; biological treatment for urban locations with controlled public access.

TABLE 29.2

CAPITAL AND OPERATING
COSTS FOR ONE-MGD INFILTRATION-PERCOLATION SYSTEMS[a]

Cost Item	Infiltration-Percolation
Liquid loading rate, inc./wk	60.0
Land used, acres	--
Land required, acres	5
Total Capital Costs	314,000
Capital cost, ¢/1,000 gal	9.3
Total Operating Costs	22,400
Operating Cost, ¢/1,000 gal.	6.1
Total Cost, ¢/1,000 gal	15.4

[a] Estimated for 1973 dollars, Engineering News-Record construction cost (ENRCC) index 1860 and sewage treatment plant construction cost (STPCC) index 192.

Source Reference 1

29.7 Specific Factors

This treatment process has potential for contamination of groundwater by nitrates and heavy metals. The heavy metals are eliminated by pretreatment techniques as necessary. Monitoring for metals and toxic organics is needed where they are not removed by pretreatment. Requires long term commitment of relatively large land area, although small by comparison to other land treatment systems. Surface water resources are diverted to groundwater. Crops grown and harvested from basins and groundwater below basin require monitoring for heavy metal content, unless metal removal was practiced in pretreatment step.

29.8 Recommendations

With the continuous usage of spreading basins the infiltration rate diminishes slowly with time due to clogging. Full infiltration is readily restored by occasional tillage of the surface layer and, when appropriate, removal of several inches from the surface of the basin.

See the discussion in Section 27 related to application options for land treatment.

30. ANAEROBIC PONDS/LAGOONS

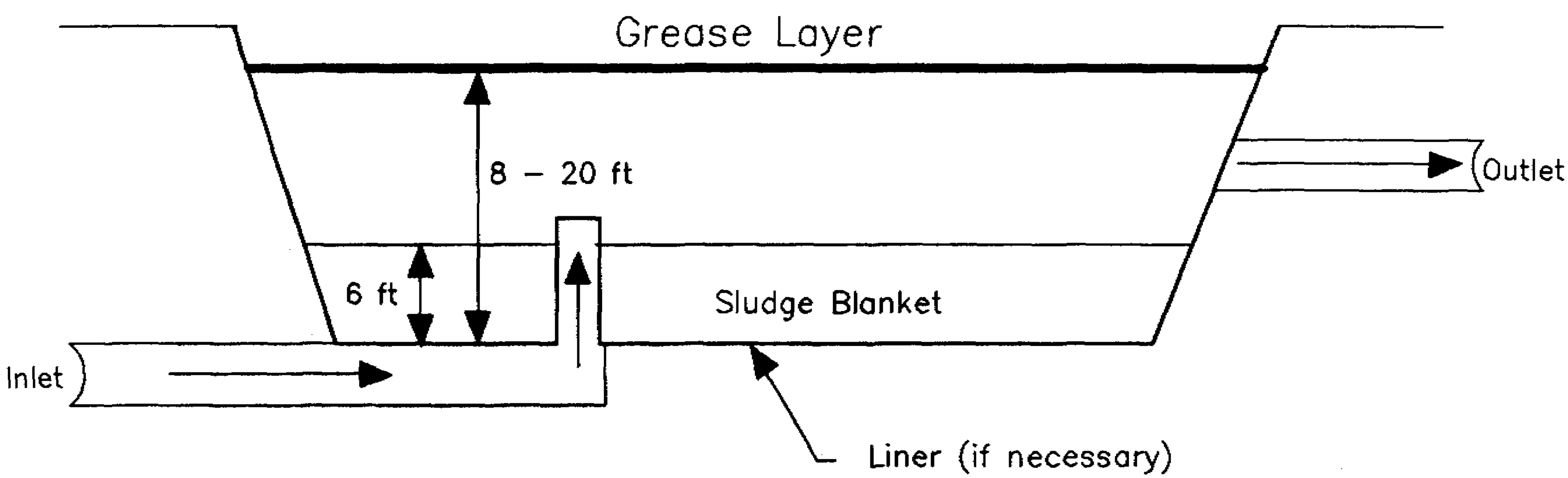
30.1 Description

Anaerobic ponds are relatively deeper (up to 20 feet) than aerobic and facultative ponds, with steep sidewalls. Anaerobic conditions are maintained by keeping loading so high that complete deoxygenation is prevalent. Although some oxygenation is possible in shallow surface zones, once greases form an impervious surface layer, complete anaerobic conditions develop. The stabilization/treatment results from thermophilic anaerobic digestion of organic wastes. During the treatment process the acid forming bacteria will break down organics in the untreated anaerobic digestion of sludge. The resultant acids are then converted to carbon dioxide, methane, and other end products.

These lagoons are constructed in parallel or series. The typical detention time 20 to 50 days, depth 8 to 20 feet (2.4 to 6.1 m), water temperature range 35 to 120 degrees F (Optimum 86 degrees F), and organic loading 200 to 2200 lb BOD5/acre/day (kg/ha/day). In the typical anaerobic pond, wastewater enters near the bottom of the pond (see Figure 30.1)(2) and mixes with active microbial mass in the sludge blanket, which is usually six feet (1.8 m) deep. The discharge is located near one of the sides of the pond, submerged below the liquid surface. Excess undigested grease floats to the top, forming a heat retaining and relatively air tight cover (2).

Anaerobic and facultative ponds have been shown to be effective in Brazil (64). Tables 30.1, 30.2, and 30.3 show results of studies in Campina Grande in northeast Brazil. So-called maturation ponds which are aerobic during the day from algal activity, may be used as post-treatment for effluent from facultative and anaerobic ponds.

Figure 30.1: Typical Anaerobic Lagoon



(Source: Ref. 2)

TABLE 30.1

EXPERIMENTAL RESULTS FROM THE SERIES OF FIVE PONDS MEAN RESULTS DURING JUNE 1988 - MAY 1979

Pond Temperature, 26°C; overall retention time, 28.1 days

	Retention time (days)	BOD_5 (mg/l)	SS (mg/l)	FC (na/100ml)
Raw Sewage	--	240	205	4.6×10^7
Effluent from pond				
1	6.8	63	56	2.9×10^6
2	5.5	45	74	3.2×10^5
3	5.5	25	61	2.4×10^4
4	5.5	19	43	450
5	5.8	17	45	30

TABLE 30.2

EXPERIMENTAL RESULTS FROM THE FOUR FACULATIVE PONDS

Selected mean results obtained in three experiments during June 1977 - December 981; temperature, 26°C

BOD_5 loading (kg/ha/d)	Retention time (days)	BOD reduction (%)
162	18.9	84
255	12.0	79
322	9.5	77
425	6.8	73
529	6.8	74
577	6.3	74

TABLE 30.3

EXPERIMENTAL RESULTS FROM THE ANAEROBIC PONDS
MEAN RESULTS DURING JUNE 1977 - MARCH 1979
(Pond Temperature, 26°)

	Retention time (days)	BOD_5 (mg/l)	SS (mg/l)	FC (na/100ml)
Raw Sewage	--	245	310	4.7×10^7
Effluent from pond				
1*	0.8	59	82	8.1×10^6
2*	0.4	46	64	5.0×10^6
3	1.9	49	57	4.7×10^6

* Ponds 1 and 2 in series; ponds 1 and 3 raw sewage.

30.2 Limitations

Anaerobic ponds may generate odors. Relatively large land area is required. For efficient operation, water temperature above 75 degrees F should be maintained.

30.3 Costs

Cost information is given in Figures 30.2 and 30.3 (2, 11).

30.4 Availability

The process is well demonstrated for stabilization of highly concentrated organic wastes.

30.5 Operation and Maintenance

These ponds are larger in size and relatively deeper than aerobic and facultative ponds. The principal advantages in developing countries are that high loadings of organics are possible at a much lower cost than any other form of treatment. There are minimum operating and maintenance requirements.

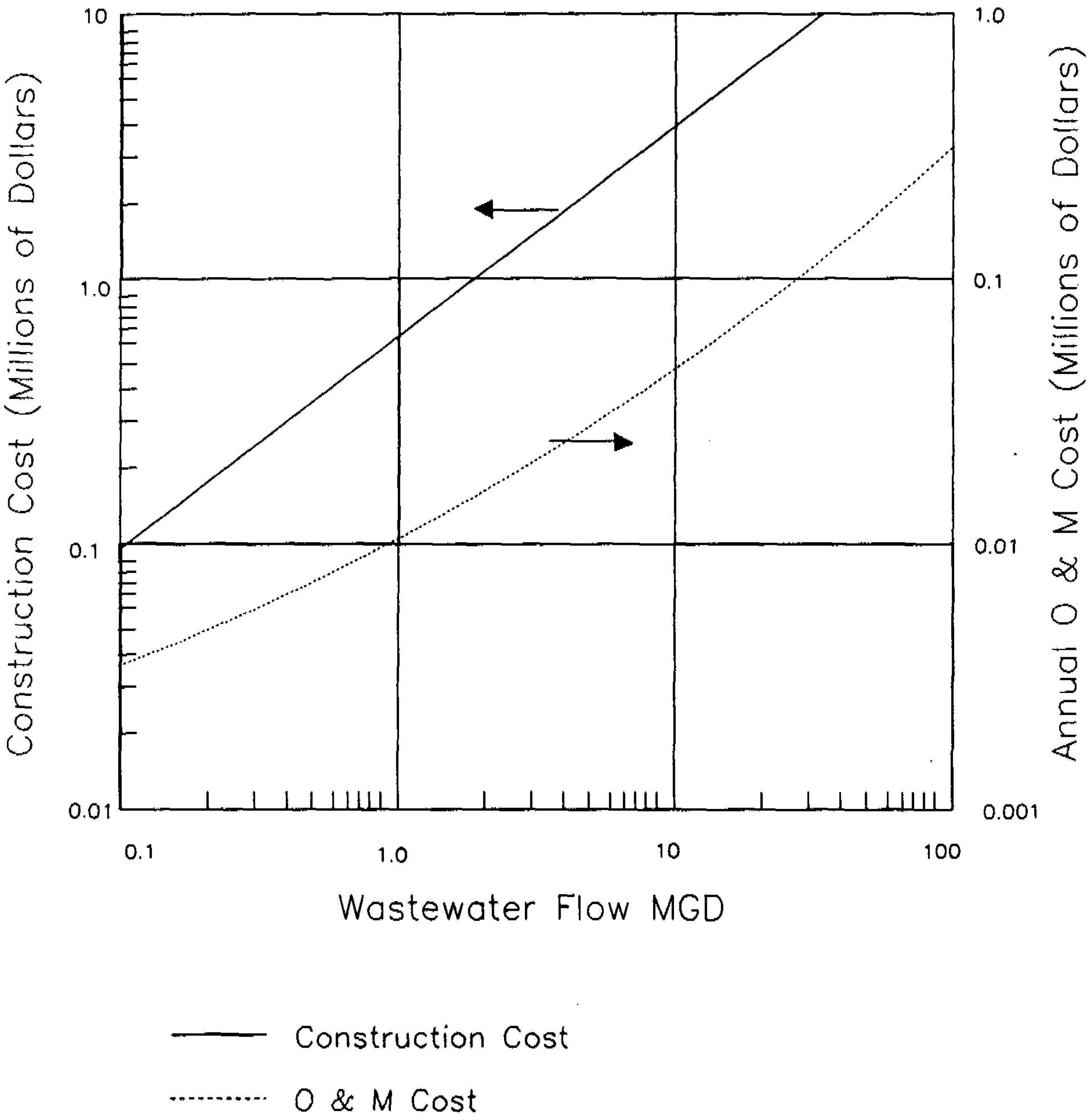
30.6 Control

The major concern in the anaerobic process to maintain oxygen free conditions. For efficient operation, water temperature should be maintained above 75 degrees F.

30.7 Special Factors

High concentrations of organic matter and bacteria / pathogens decomposition may result in odor problems. Also, there is potential for seepage of wastewater into groundwater unless ponds are lined with clay or impervious material.

Figure 30.2: Construction, Operation and Maintenance Costs for Anaerobic Lagoons



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30.8 Recommendations

Ponds and lagoons are simple and efficient waste treatment technologies. There are several sections in this handbook devoted to these. Waste treatment should be considered for extensive application in Latin America, and lagoons and ponds will play an important role.

31. OZONE DISINFECTION

31.1 Description

Ozone (O_3) may be used for disinfection in water and wastewater treatment process. As a disinfectant (dosages of 3 to 10 mg/l are common), ozone is an effective agent for deactivating common forms of bacteria, bacterial spores and vegetative microorganisms, as well as eliminating harmful viruses. Additionally, ozone acts to chemically oxidize materials found in the water and wastewater and can reduce the BOD_5 and DOC, forming oxygenated organic intermediates and end products. Further, ozone treatment reduces water and wastewater color, odor and taste. (2,48,73).

Ozone injection into the flow may be accomplished via mechanical mixing devices, countercurrent or co-current flow columns, porous diffusers or jet injectors. Ozone acts quickly and consequently requires a relatively short contact time. (See Figure 31.1).

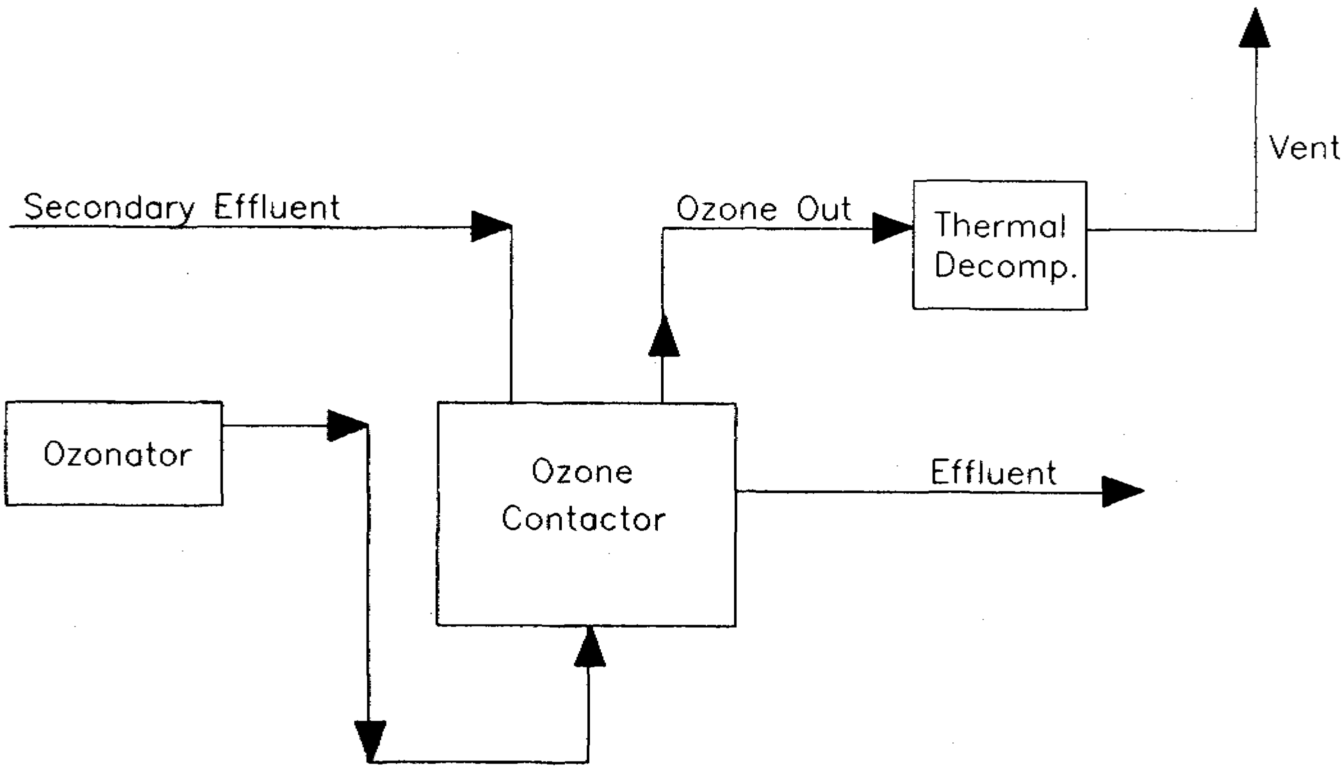
Results of disinfection by ozonation and design criteria are providing in Table 31.1

Table 31.1

DESIGN CRITERIA FOR OZONATION

Influent	Dose, mg/l	Contact Time Minutes	Effluent Residual
Secondary effluent	5.5-6.0	Less than or equal to 1	Less than 2 fecal coliforms/100 ml
Secondary effluent	10	3	99% inactivation of fecal coliform
Secondary effluent	1.75-3.5	13.5	Less than 200 fecal coliform/ 100 ml
Drinking water	4	8	Sterilization of virus

Figure 31.1: Ozone Disinfection: Flow Diagram



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Ozone has been found to be a good oxidant for removal of cyanide, phenol and other dissolved toxic organic materials. Combination of ozonation and activated carbon treatment can achieve 95 percent chloroform and other trihalomethanes removals.

31.2 Limitations

Ozonation may not be economically competitive with chlorination under local conditions.

Although ozone is effective in disinfecting water, its use is limited by its solubility. The temperature and pressure of water being treated regulate the amount of ozone that can be dissolved in the water. These factors tend to limit the disinfectant strength that can be made available to treat the water.

Many scientists claim that ozone destroys all microorganisms. Unfortunately, significant residual ozone does not guarantee that a water is safe to drink. Organic solids may protect organisms from the disinfecting action of ozone and increase the amount of ozone needed for disinfection.

In addition, ozone residuals cannot be maintained in metallic conduits for any period of time because of ozone's reactive nature. The inability of ozone to provide a residual in the distribution system is a major drawback to its use.

31.3 Costs

See Figure 31.2 and 31.3 for Construction and Operation and Maintenance Costs, respectively.

31.4 Availability

Ozone has been used in the water industry since the early

Figure 31.2: Construction Cost for Ozone Disinfection

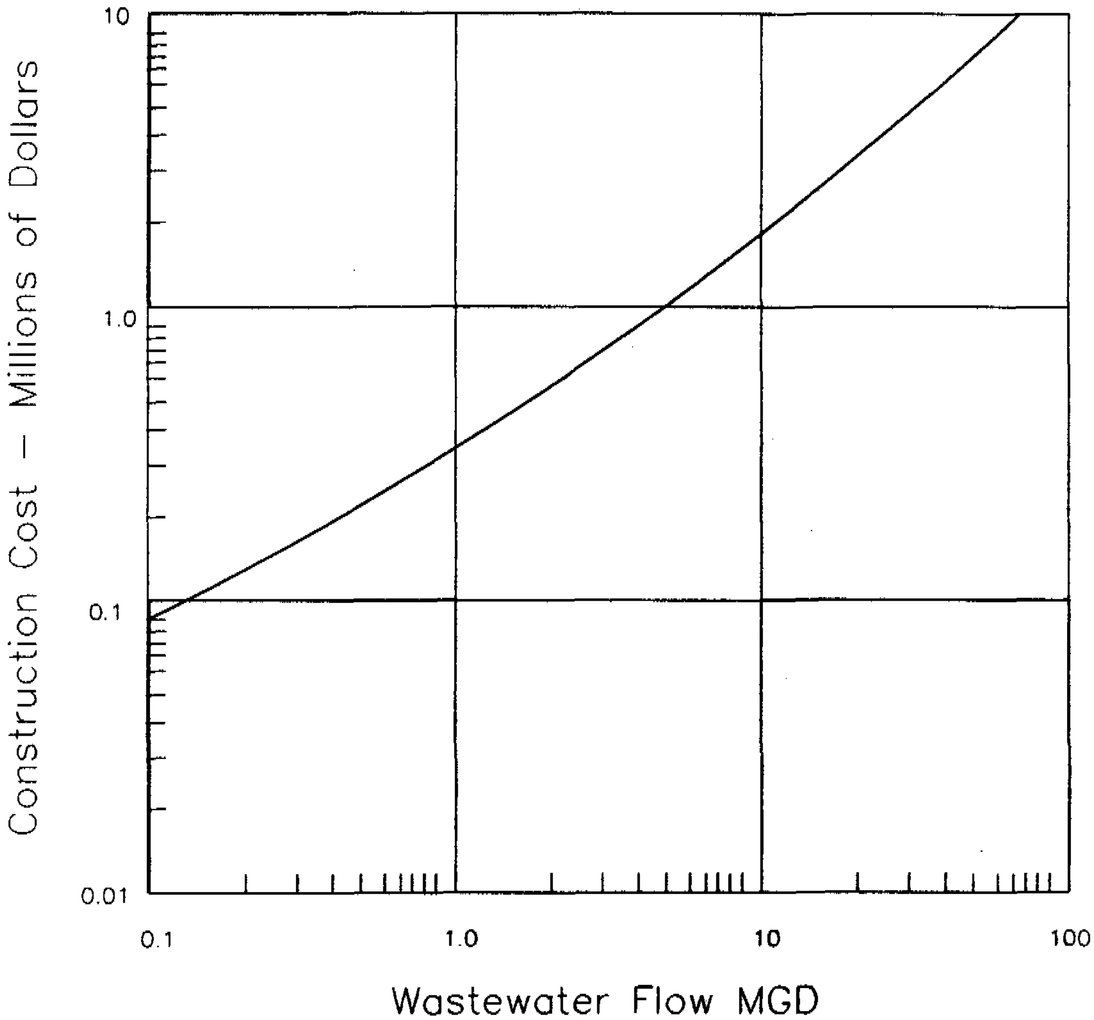

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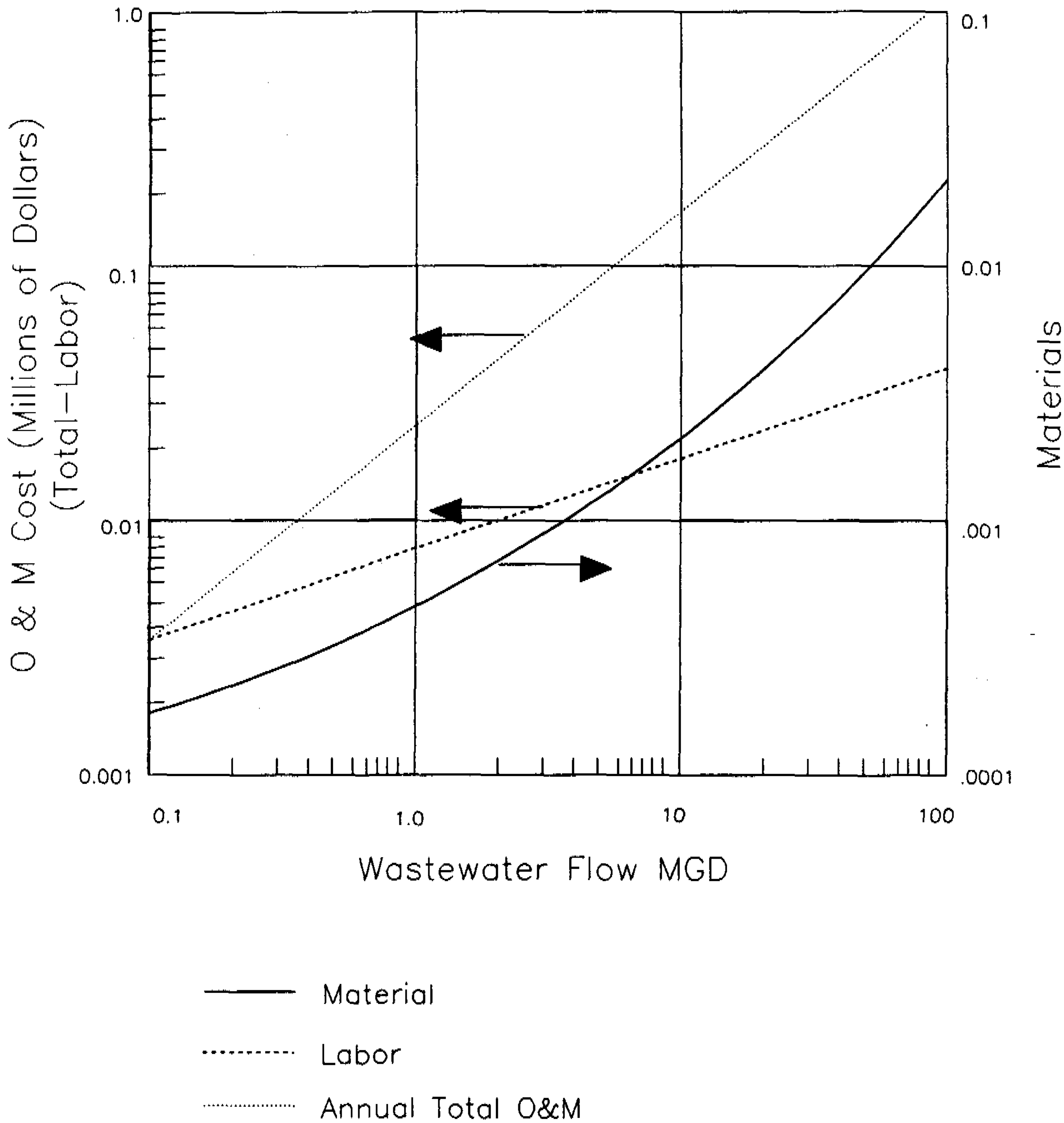
Figure 31.3: Annual Operation & Maintenance Cost for Ozone Disinfection



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1900's, particularly in France and it has been fully demonstrated but not widely used in the United States because of the relatively high cost of ozone. Recent developments in ozone generation have lowered the cost and thus make it more competitive with other disinfection methods.

31.5 Operation and Maintenance

The ozone disinfection system is a complex series of mechanical and electrical units, requiring substantial maintenance, and is susceptible to a variety of malfunctions. Since data on long-term experience are relatively unavailable, it is not possible to assess maintenance requirements on air cleaning equipment, compressors, cooling and drying equipment, and contractors. It is estimated that 8 to 10 kWh/lb of ozone generated will be required. Monitoring requirements are similar to those for UV disinfection, including occasional bacterial analyses and routine ozone monitoring.

31.6 Control

Ozone breaks down to elemental oxygen in a relatively short period of time (half life about twenty minutes) therefore it is difficult to store. Consequently, it is generated on site using air as the raw material. The ozone generation process utilizes a silent electric arc or corona through which air or oxygen passes yielding a certain percentage of ozone. Automatic devices are commonly applied to control voltage treatment, frequency, gas flow and moisture, all of which influence the ozone generation rate.

Easily oxidizable waste water organic materials consume ozone at a faster rate than disinfection; therefore, effectiveness of disinfection is inversely correlated with effluent quality but directly proportional to ozone dosage. When sufficient ozone is introduced, ozone is a more complete disinfectant than chlorine.

31.7 Special Factors

Ozone is an air pollutant which can discolor or kill vegetation coming in contact with it. Residual ozone in off-gas streams must be processed for ozone decomposition prior to release. Ozone is toxic when inhaled in sufficient concentration.

Organic removal is improved with ultraviolet radiation. It is postulated that the UV activates the O_3 molecule and may also activate the substrate. Ozone-UV is effective for the oxidative destruction of pesticides to terminal end products of CO_2 and H_2O.

Effluents containing high levels of suspended solids may require filtration to make ozone disinfection more cost-effective.

31.8 Recommendations

Ozonation is applicable where chlorine is deficient or where chlorine disinfection may produce potentially harmful chlorinated organic compounds (such as Trihalomethanes). If oxygen-activated sludge is employed in the system, ozone disinfection is economically attractive, since a source of pure oxygen is available facilitating ozone production.

In many ways, the desirable properties of ozone and chlorine as disinfectants are complementary. Ozone provides fast-acting germicidal and viricidal potency, commonly with beneficial results regarding taste, odor, and color. Chlorine provides sustained, flexible, controllable germicidal action that continues to be beneficial during distribution. Thus, it would seem that a combination of ozonation and chlorination might provide an almost ideal form of water supply disinfection.

32. WATER COLLECTOR

32.1 Description

This collector is used where underground water supply is available. Specifically, the Ranney collector is used primarily for water supply development and treatment. The Ranney method consists of sinking a reinforced concrete caisson from the bottom of which screens are projected horizontally like the spokes of a wheel; as much as 3000 lineal feet from a single unit is practicable. The caisson is used as a clear well from which the water can be pumped. Refer to Figure 32.1 (30, 63). Figure 32.2 shows the application with a pump facility for direct water supply.

The screens which may be 8, 12, 18 or 24 inches (1 inch = 2.54 cm) in diameter are fabricated from heavy steel and perforated with longitudinal slots. One of the chief advantages of the horizontal collector system is the fact that the entire depth of the aquifer is utilized. Another major advantage is that any required area of screen openings can be developed so as to control the entrance velocity of water into the laterals. The design capacity is usually in the range of 2 - 50 MGD (0.09 to 2.2 m3/sec).

The type of underground water that is considered in this case is infiltrated water which is formed as a result of percolation of river water as it flows over sand and gravel formations that are adjacent to the river. The sand and gravel formation is generally very permeable thus allowing for free flow of water from the body to the formation. This water then settles underground. In the course of percolation of the water in the formation natural filtration occurs thus producing very clean and odorless underground water.

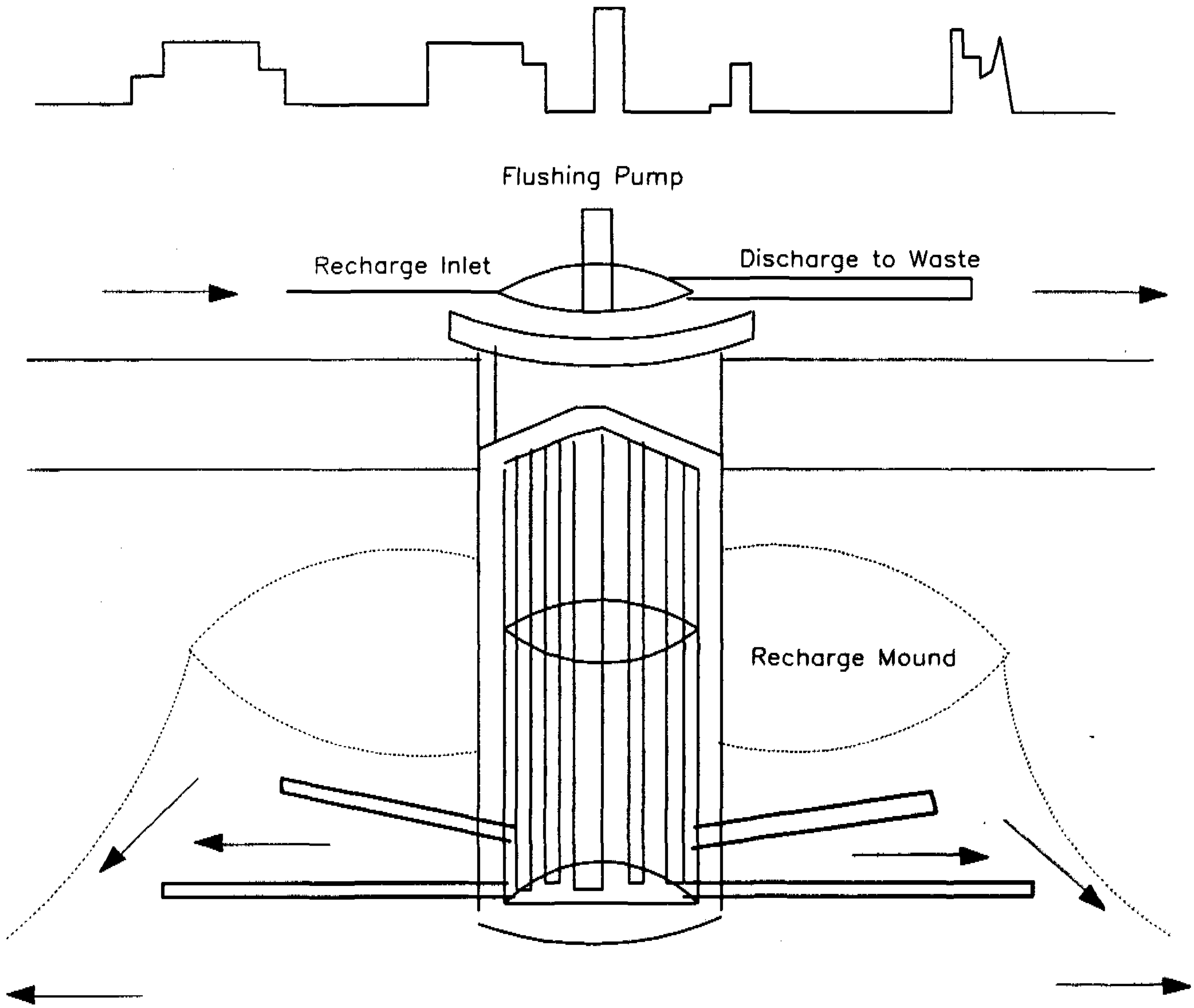
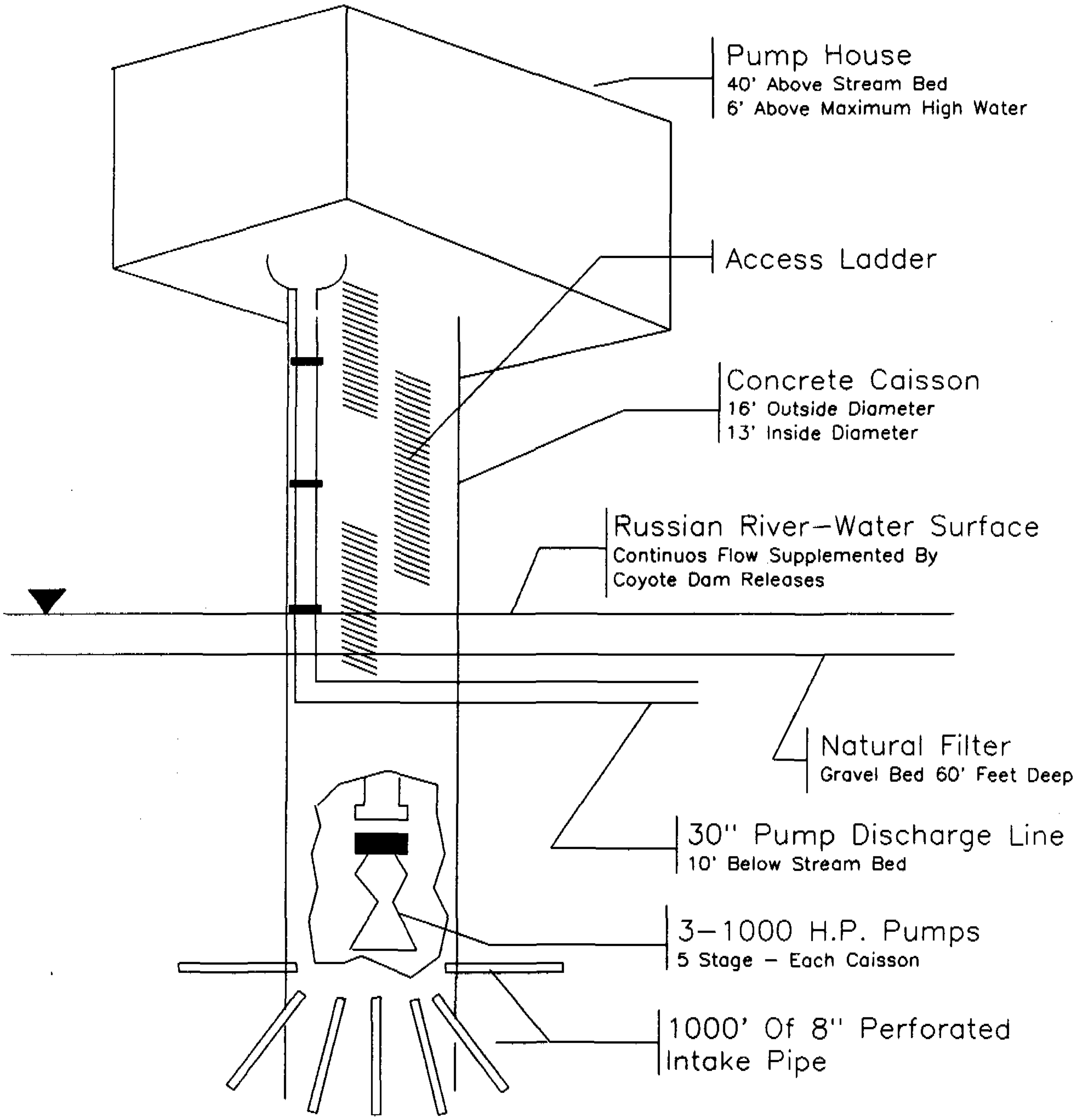


Figure 32.1: Flushing Pump Showing Recharge Mound Mound Developed by Ranney Western Kennewick, WA. 1985.

(Source: Ref. 30)

(File:Martin38)

Figure 32.2: Wohler Pumping Plant
Ranney Water Collectors



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The caisson has a hollow inside, therefore, making it look like a well from which water could be pumped. The spoke-like laterals may be up to twenty-four inches in diameter and are pre-fabricated from dense perforated steel with longitudinal slots. When the lateral is projected into the sand-gravel aquifer, sand is removed in order to prevent resistance to permeability if the lateral just presses on or pushes the sand causing it to compress.

The caissons are usually designed to rise a little above the highest flood level of the stream or lake. But sometimes the requirement may be that the design should not destroy the shorelines natural character in which case the caisson would stay below the surface.

The method by which the caisson collects water is as follows: pressure induced by the groundwater forces water through the lateral screens into the caisson until the water in the caisson reaches the level of the sea or river water. When the water is withdrawn from the caisson, the difference in pressure, as indicated by height levels, causes more water to flow in.

A pump house usually sits on top of the caisson and houses the pump controls, as its name indicates. There is a structure on the shore of the water body that houses the chlorinator, the flow meters and the electrical controls.

32.2 Limitations

This system is effective as a community water supply system only where there is sufficient underground water supply. Alternate supplies may be required on an intermittent basis. Contaminated water may require additional treatment.

32.3 <u>Costs</u>

Construction data is presented in Figure 32.3. It should be kept in mind that the comparison shown to surface water treatment plant costs is only valid where the screening provided in the water collector system is comparable to that required in an alternate type treatment plant. Additional treatment may be required and often is.

32.4 <u>Availability</u>

This system should be used where a study related to groundwater resource availability has been done. The same would hold for any groundwater development project. The collectors have been installed in many locations in North America and Europe.

32.5 <u>Operation and Maintenance</u>

Operational costs for this system may be small when quality and supply of the resource is consistent, and where the automatic cleaning function of the system is maintained.

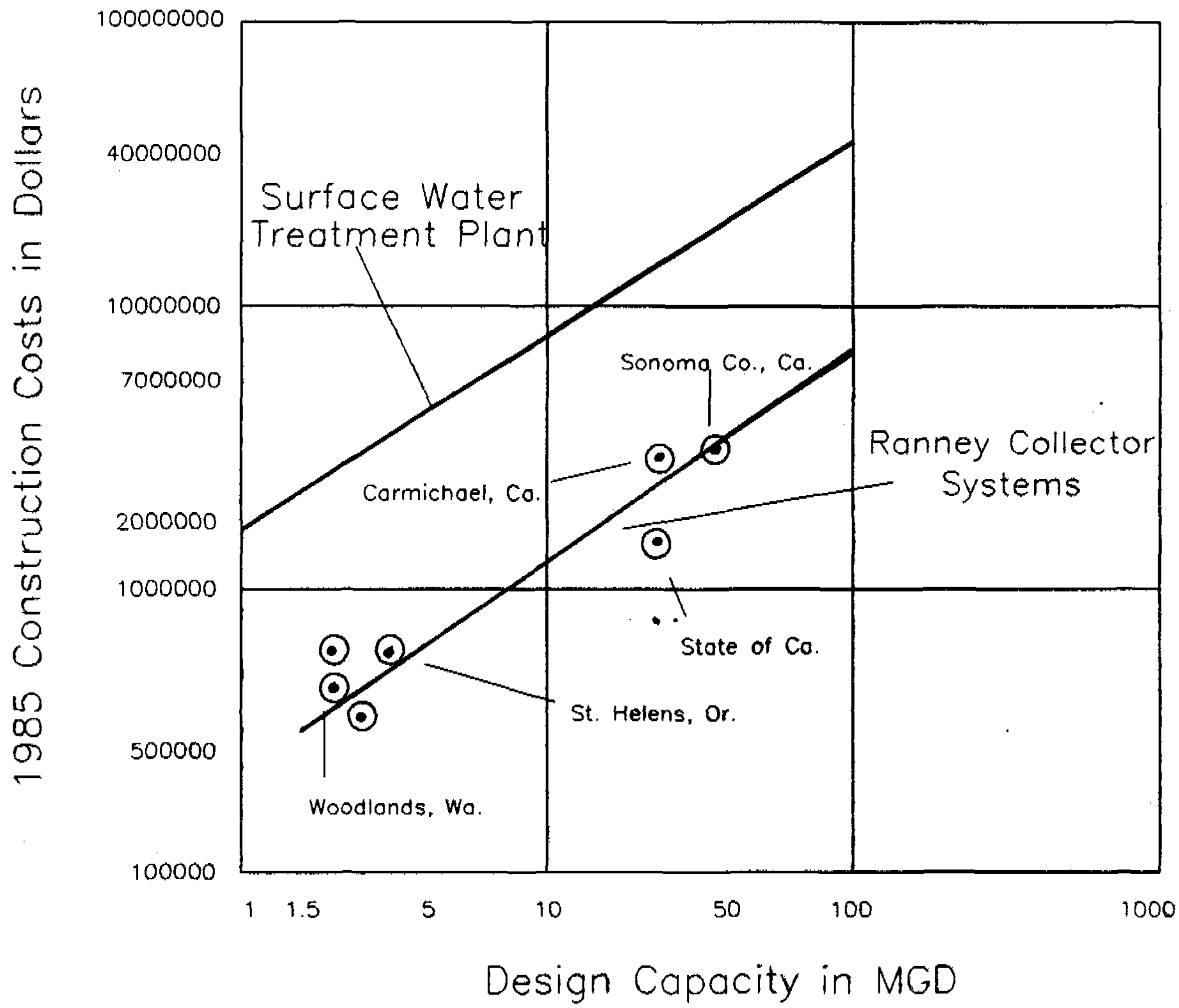
An automatic control mechanism determines the amount of water that is extracted from the aquifer by controlling the surface are of the laterals available for water transmission. There is also automatic control on the amount of water that is pumped to the user.

The system may require some specialized labor to monitor system operation to control the chlorinator. Also, fuel is necessary to run the pumps and other mechanical equipment.

32.6 <u>Control</u>

This is a closed loop control system that is used for the smooth running of this system. There is a control located in

Figure 32.3: Construction Costs for Surface Water Treatment Plant and Ranney Collector Systems.



(Source: Ref. 30)

the onshore structure that constantly determines demand and controls necessary pumping. The control system can be expanded to control everything from the chlorinator feed to the area used for water collection in the laterals.

32.7 Special Factors

An artificial recharge method can be carried out without necessarily contaminating the water that with collected down below. This is done by connecting laterals at a certain distance just below the surface. These laterals will discharge water into the ground. Studies should be done on using the collector with treatment systems for water recovery and reuse.

32.8 Recommendations

This system can produce anywhere from as little as half a million gallons per day to twenty million gallons per day from a single unit. It may be considered for application directly where groundwater treatment requirements are met by the collector system (screening). In other applications mat require additional treatment.

33. CHEMICAL ADDITION FOR WATER AND WASTEWATER TREATMENT

33.1 Description

Introduction

Colloidal solids and very finely divided suspended matter cannot be effectively removed from wastewater by plain sedimentation unless they are rendered settleable by the addition of chemicals. Three theories have been proposed which explain the effects of chemical addition to wastewater:

a) Certain heavy metal salts when treated with alkaline materials form heavy precipitates which enmesh and carry down colloidal suspensions by mechanical entrapment. Salts of iron and aluminum fall into this category.

b) It has been proven that colloidal particles possess an electrical charge, which because they are alike repel each other keeping the particles in suspension. If a precipitant with an opposite charge is added the charges neutralize each other and settling is enhanced. This explains the greater effectiveness of multivalent ions such as Ferric Chloride.

c) Insoluble substances which have a large surface area can effectively adsorb pollutants and act as the nuclei for the start of flocculation/precipitation. Activated carbon is a large surface area material.

To be effective, the chemicals must be distributed evenly through the wastewater being treated. Rapid mixing which can be done with the in-line devices, turbulence in channels, paddles, propellers or diffused air will achieve this result with a minimum tank volume. A common rapid mixer consists of a constant speed motor driving a propeller through a speed reducer.

Detention periods for mixers vary from 0.5 to 3.0 minutes and G values from 200 to 300 per sec.

Once floc growth has begun, maximum opportunity for contact between floc and colloidal should be encouraged by controlling the rate of mixing. This process is called flocculation and takes place in flocculation tanks. Flocculation tanks are usually duplicated and designed with detention periods of from 10 to 30 minutes. In general, G values are maintained between 10 and 75/sec and Gt values between 10,000 and 100,000.

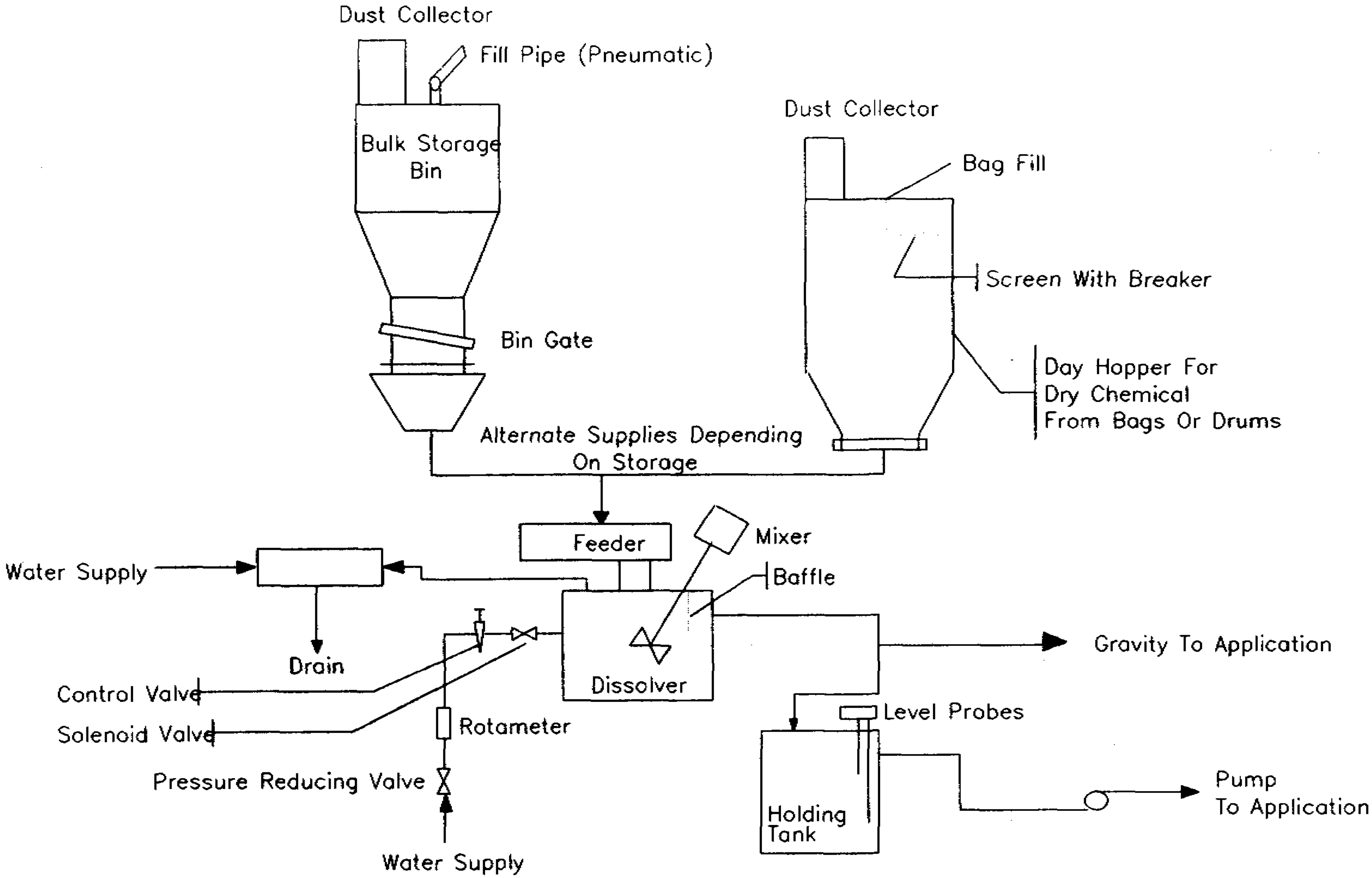
Normally settling tanks are designed with detention periods of from 1-4 hours and settling rates of no more than 1,000 gal/ft^2/day. (2, 69, 70, 71, 72)

Commonly Used Chemicals

The most common/economical chemical additives are Alum, Ferric Chloride, Lime, Polymers, and Powered Activated Carbon.

Alum: Alum is used for suspended solids/phosphorous removal. It is added directly to the wastewater which is mixed, flocculated, and settled. Also, alum has been used as a filter aid in tertiary filtration. Alum is marketed as a solid (lump, ground, rice, or powdered forms) or liquid (50% solution). The choice between liquid or dry (solid) alum is dependent on economics. In general, dry alum is utilized in wastewater treatment. Alum weighs 40 to 75 lb per cu ft depending on its form, and its solubility in water varies from 50 lb/gal at 32°F to 66 lb/gal at 76°F. Alum is best fed dry as ground rice. The dry feeder supplies a measured quantity to a dissolver tank and feeding of the alum solution is accomplished by gravity or pumping (see Fig. 33.1). For a minimum detention period of 5 minutes, 2 gallons of water per pound of alum are required. Powdered alum is slightly hygroscopic, dusty and cakes and arches in hoppers. Dry alum is stored in mild steel or concrete bins.

Figure 33.1: Typical Chemical Mixing And Feed System



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Required storage space for dry alum is between 30 and 55 cu ft per ton. Alum solution is corrosive, hence the dissolving tank, pumps and all piping in contact with it must be constructed of resistant materials, e.g., rubber, 316 stainless steel, FRP, or plastic.

Ferric Chloride: Ferric chloride is used for suspended solids and/or phosphorous removal. In these applications, solids contact or separate mixing/flocculation tanks are used in treatment of raw wastewater or secondary (trickling filter/activated sludge) effluent. Ferric chloride (also called ferrichlor) is available in either liquid or dry (hydrated crystal or anhydrous powder) form. The commercial solution which dissolves completely in water, is supplied in concentrations of from 35-45% $FeCl_3$ and weighs between 11.2 and 12.4 lb/gal. Crystalline ferric chloride comes in lumps or sticks, which weigh 60 to 64 lb per cu ft and contain 60% $FeCl_3$. Their solubility is 5.4 lb/gal at 50°F and 7.6 lb/gal at 68°F. The powder weighs 85 to 90 lb/cu ft and contains 96 to 97% $FeCl_3$. Its solubility in water is 6.2 lb/gal at 32°F. All forms of $FeCl_3$ are best fed as solutions in concentrations up to 45% $FeCl_3$. Dry $FeCl_3$ is dissolved on site before use in treatment. The stored solution is transferred to a "day tank" and controlled quantities are fed to the mixer by gravity or pumping. Ferric chloride solutions are staining and corrosive, hence must be handled with care. Also, all tanks, pipes, pumps and other equipment coming in contact with the solution must be constructed of corrosion resistant materials such as rubber, Saran, Teflon, plastic, FRP or vinyl.

Lime: Lime addition is used for improved removal of suspended solids, removal of toxic metals and the removal of phosphates. The primary use of this process is for phosphorous removal. Lime is available in many forms, however quicklime (CaO) and hydrated lime ($CaOH_2$) are the most prevalent forms marketed. Quicklime or unslaked lime is available in lumps, pebbles, crushed or ground. When water is added it slakes to hydrated lime with the evolution of heat. Quicklime weighs 55-75 lb/cu ft and contains 70-90% CaO. It is best fed dry and from 0.4-0.7 gal of water is required for continuous solution. Final dilution should be about 10%. Steel and concrete are suitable materials for handling. Quicklime should not be stored for more than 60 days even in the tightest container.

Hydrated lime is fine, white powder which weighs 35-40 lb/cu ft and has a commercial strength of 82-99% $Ca(OH)_2$. Its solubility in water is low, 1.5 lb/100 gal at 32°F and 1.3 lb/100 gal at 68°F. Hydrated (slaked) lime may be fed at a maximum rate of 0.5 lb/gal for continuous dissolving or at a rate of 0.93 lb per gal as a slurry. Hydrated lime is caustic, irritating and dusty and should be stored dry. Steel and concrete are suitable materials for handling lime.

Polymer: Polymers (or polyelectrolytes) are high molecular weight compounds (usually synthetic) which can be used as coagulants, coagulant aids, filter aids or sludge conditioners. Polymers are utilized alone or in conjunction with other chemicals such as lime, alum, or ferric chloride to improve performance. Polymers are also used to strengthen flocs to facilitate effluent filtration. Polymers are available in liquid or dry forms. Dry polymers are supplied in relatively small quantities (100 lb bags) and must be dissolved prior to use in treatment. A solution of from 0.2-2.0 percent concentration is used. Many competing polymer formulations with differing characteristics are supplied as stock solutions ready for feeding to the treatment process. Manufacturers of these should be consulted as to their use in the treatment of wastewater. Polymer solutions are fed using equipment similar to that commonly used for coagulant (Alum-Ferric Chloride) addition. Stock polymer solutions may be very viscous and special attention must be paid to the diameter of pipes and sizes of orifices used in the feed system. Corrosion resistant materials such as 316 stainless steel, FRP, or plastic should be used in the handling of polymer solutions.

Powdered Activated Carbon: Powdered activated carbon is used in water and wastewater treatment to adsorb soluble organic materials and as an aid in the settling process. Over the past several years a new technology has been developed in which powdered carbon is added to the aeration basins of biological treatment systems. This application achieves high BOD and COD reductions, improved settling in final clarifiers and adsorption of color, toxic organic compounds and detergents. Powdered carbon is marketed in bags or bulk, weighs 15-30 lb/cu ft and requires from 72-135 cu ft of storage space per ton. It is insoluble in water and is usually fed dry or as a suspension (slurry). Powdered carbon is fed using chemical feed equipment (either dry of slurry feeders) similar to those used for feeding other chemicals. Spent carbon is removed with the sludge and either discarded or regenerated. Regeneration can be accomplished with a furnace or wet air oxidation process. Because powdered carbon is combustible, storage should be isolated because of the possibility of carbon fires. Suitable materials for handling powdered carbon include for dry carbon; iron, and mild steel, and for wet carbon; rubber, silicon iron, and 316 stainless steel.

Handling and Storage of Chemicals

Suitable provision must be made for the handling and storage and feeding of chemicals. Depending on plant size, bags, drums, barrels, or bulk via truck, rail or barge should be considered. They must be unloaded, conveyed to and from storage, weighed and added to the water treatment process in measured amounts. Dry chemicals are transported by hand, on belt conveyors, bucket elevators, pneumatic tubes or screw conveyors. Liquids are pumped or flow by gravity. Storage containers and/or hoppers, solution/slurry tanks, conveyors, buckets, tubes, pipes, valves, fittings orifices and pumps must be constructed of materials resistant to attack by the chemicals being handled. Pipes and pneumatic tubes should be straight and provided with clean-outs to relieve clogging.

The amount of chemical fed is measured by dry (Fig. 33.1) or solution feed apparatus (Figs. 33.2 and 33.3) depending on the nature of the chemical applied. Dry feed machines control chemical dosage by regulated volumetric or gravimetric displacement of dry chemicals (accuracy range 107%). The latter is more accurate, reliable, and amenable to automatic control, but is more expensive. For handling certain chemicals, agitators and dust control are provided. The measured dry chemical is usually dissolved/diluted in water before being added to the treatment process.

Solution feed devices control chemical dosage by regulating displacement of raw liquid chemicals or of chemical solutions made up to known strength. Measurement is by orifices, flow tubes, meters or positive displacement, plunger or diaphragm pumps.

Chemical Doses

The amount of chemical that must be added to the treatment process varies (1) with the quality of the water, (2) with the degree of treatment (removal) required, (3) with treatment conditions, and (4) with the quantity of water to be treated. Optimal chemical dosage is determined by repeated bench scale testing and laboratory analysis. In most cases, frequent "jar" testing is necessary to determine proper chemical dosages. Results of "jar" tests should be compared to, or correlated with, actual plant performance. Bench scale analyses should include testing to determine optimum pH, G and Gt values for chemical addition. Plant economy depends upon careful control of chemical

Figure 33.2: Alternative Liquid Feed Systems For Overhead Storage

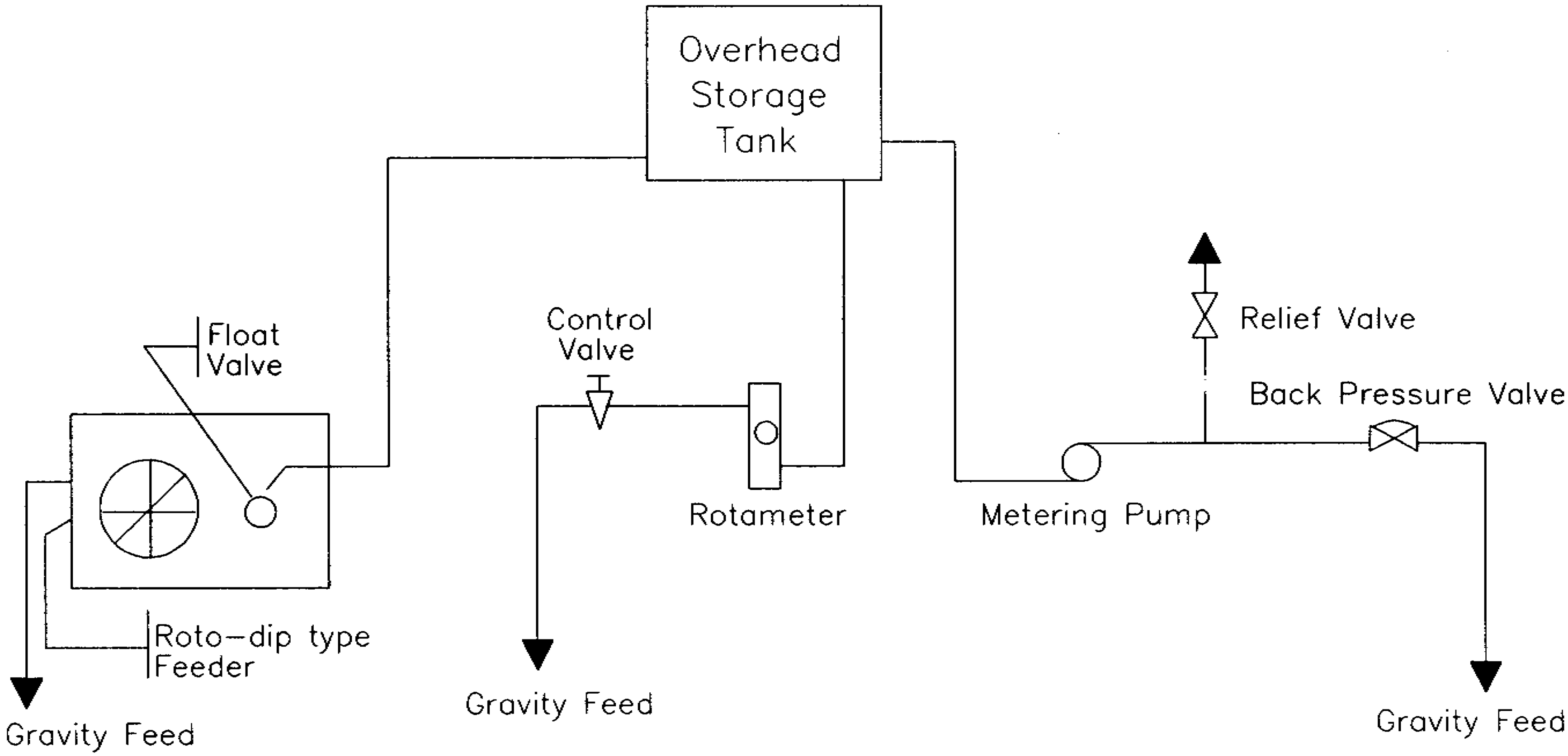

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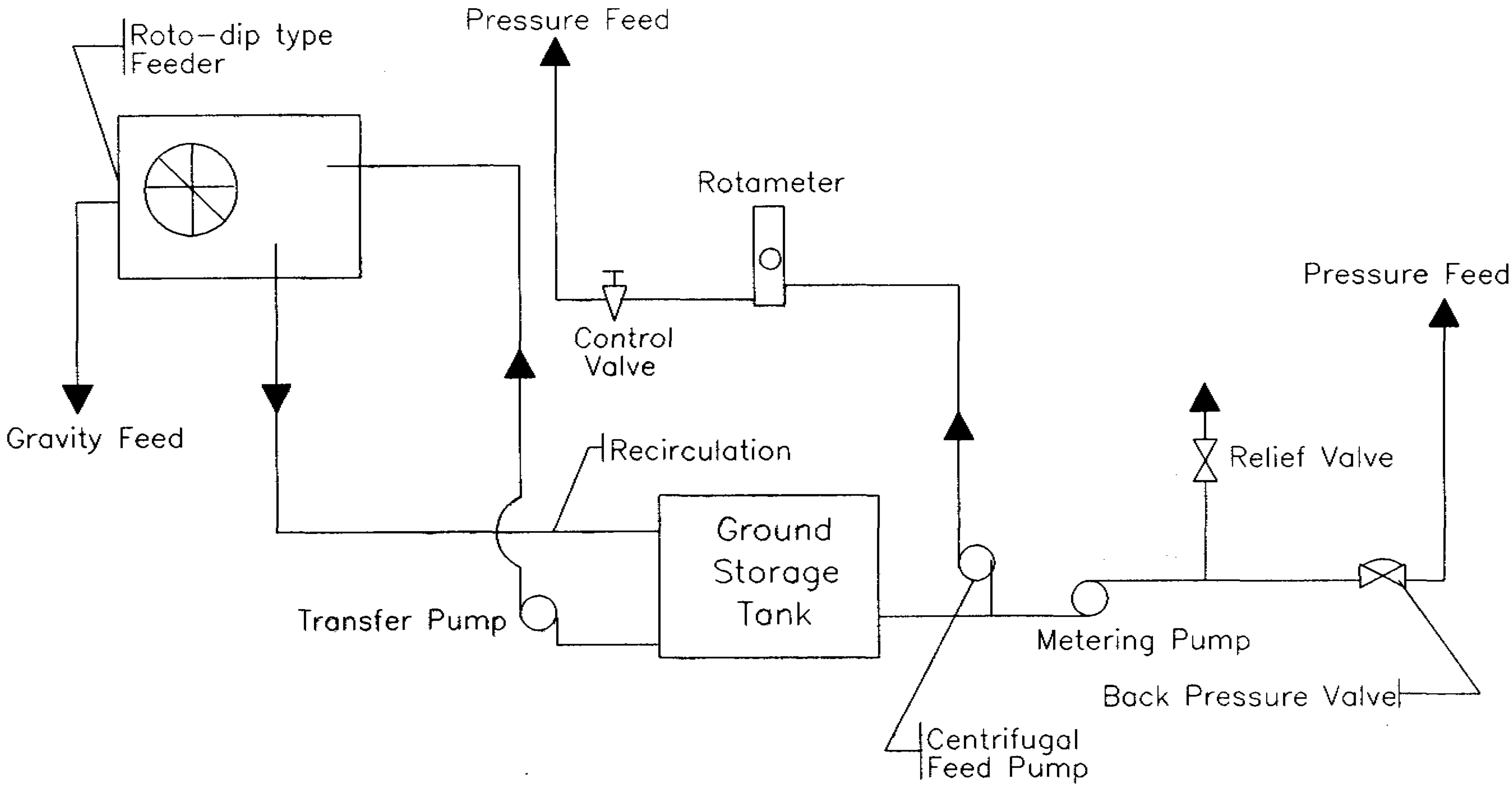
Figure 33.3: Alternative Liquid Feed Systems For Ground Storage



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doses. Too small an amount can be as wasteful as too large a dose.

Dosages commonly employed in the design of chemical treatment at municipal wastewater treatment plants are as follows:

Alum - 5 to 20 mg/l for SS removal; +/- 100 mg/l for P removal
Ferric chloride - 20 to 200 mg/l
Lime - 100 to 500 mg/l @ CaO
Polymer - 1 to 10 mg/l
Powdered Carbon - 50 to 300 mg/l

Effectiveness

As a treatment process, addition of chemical coagulants (alum, lime, ferric chloride, polymer) accomplishes removal of BOD and solids about midway between plain sedimentation and secondary biological treatment. Removal of about 80% of the suspended solids and 65% of the BOD can be achieved with "normal" doses. If doses are doubled, removals as high as 90% of the suspended solids and 80% of the BOD may be reached. The addition of such chemicals has little or no effect on the concentration of soluble BOD or COD. The addition of lime, alum, or ferric chloride to either effluent from a biological treatment process or untreated wastewater can achieve phosphorous removals in the range of 80-95%. Phosphorous concentrations as low as 0.1-0.5 mg/l can be achieved utilizing such chemical addition processes.

It has been shown that a wastewater treatment process utilizing the addition of powdered carbon along with coagulating chemicals can achieve SS, BOD and P removals of over 90%.

33.2 Advantages and Disadvantages

The advantages of chemical addition in wastewater treatment include:

- o Reliability, with proper operational control, capable of producing consistently high effluent quality.
- o Flexibility, treatment efficiencies can be varied as needed, e.g., seasonally to protect streams during periods of low runoff or beaches during summer months.
- o Process is suitable for treatment/pretreatment of wastewaters containing toxic or otherwise objectionable industrial wastes which would be harmful to or render biological treatment processes inactive.
- o Removes phosphorous from untreated wastewater and treated effluents.
- o Removes heavy metals from raw wastewater and treated effluents.

The disadvantages of the chemical addition process are:

- o Relatively high life cycle if operated on a full time basis.
- o Increases amount of sludge generated, thereby increasing sludge treatment and disposal costs.
- o High concentrations of chemicals and/or heavy metals or other toxic materials in the sludge may render it unsatisfactory for use as a fertilizer and/or soil conditioner.

- Skilled labor required for operation and maintenance of systems.

- Certain of the mechanical equipment and chemicals required in the process may not be readily available in developing countries.

33.3 Costs

Estimated capital and operation and maintenance costs for chemical treatment including alum, ferric chloride, lime, polymer and powdered carbon addition are presented in Figures 33.4 and 33.5 respectively. The curves presented in the aforementioned figures include the costs for handling, storage, and feeding of chemicals, but do not include provision for the cost of mixing, flocculating and/or settling facilities.

33.4 Availability

The technology and equipment and appurtenances typically utilized in the simpler chemical application systems, e.g., solution tanks, water level controllers (float controlled valves), piping, valves, and orifices are available in most areas of the world. Similarly, alum, lime and ferric chloride are readily available in many parts of the world.

Gravimetric and volumetric dry feed machines, slakers, mechanical/pneumatic conveyors, piston and diaphragm pumps, and remote/automatic controls may not be readily available in developing countries. Such equipment may have to be imported, hence cost prohibitive. Similarly, chemical additives such as polyelectrolytes and powdered carbon may not be available within developing nations and may be costly to import.

Figure 33.4: Construction Costs For Chemical Addition

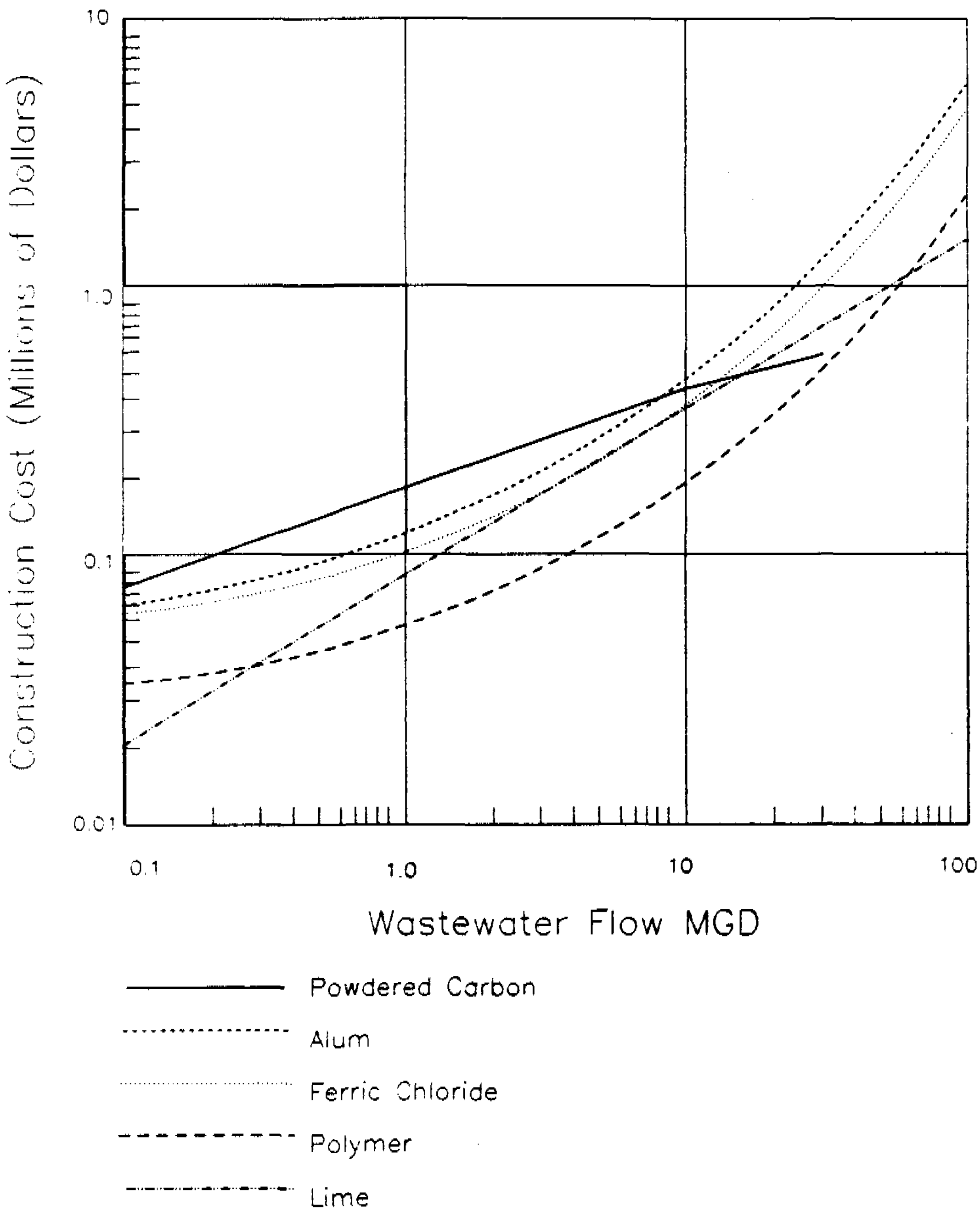

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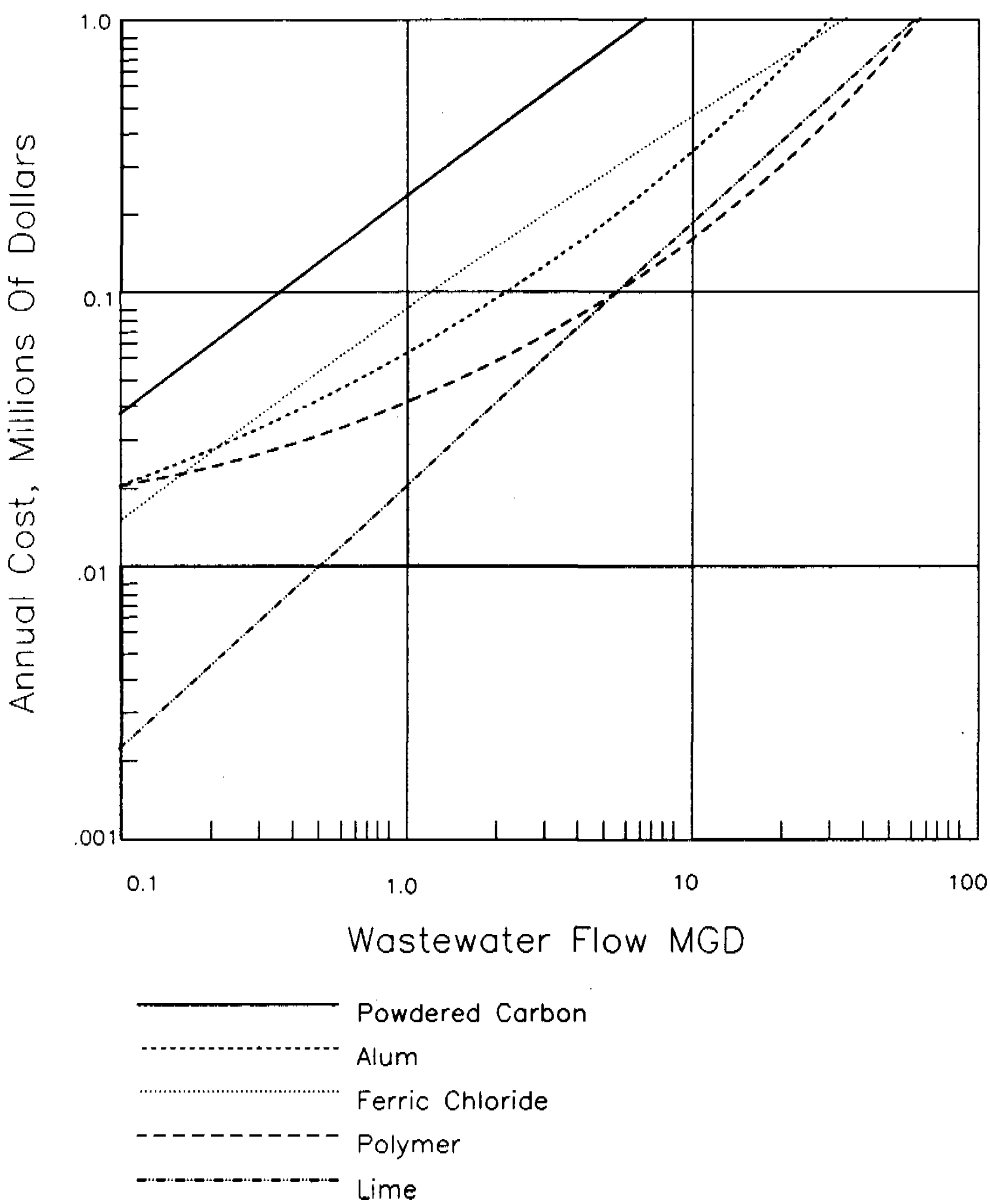
Figure 33.5: Operation & Maintenance Cost for Chemical Addition



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33.5 Operation and Maintenance

The effectiveness of chemical treatment is a function of the concentration of chemical used, the contact time, and conditions during the contact. Thus, for best results, dosage, time of contact and contact conditions must be optimized. In plants which employ mechanical systems, the speed of mixers and flocculator paddlers must be adjusted to optimize dispersion of chemicals and opportunity for floc build-up and contact. In tanks which employ turbulence and baffles to accomplish mixing and flocculation, opportunity to control contact time and conditions is limited unless baffles are movable.

In plants which use relatively simple solution feeder systems, the makeup of chemical solutions of known strength and the adjustment of solution flow rates to the point of application are critical to successful operation of the process.

In plants employing more sophisticated chemical feed systems, all mechanical equipment must be periodically and systematically inspected, lubricated and overhauled. Flow metering devices should be checked at least yearly. Similarly, feeders must be checked for accuracy. With dry feeders it is desirable to catch the feed for a short period of time, then weigh it on scales. Solution feeders can be checked by comparing their rated output with the time-volume displacement of the solution in a tank of known geometry.

Routine operation and maintenance includes unloading and storage of chemicals, dust control, conveyor, pipeline, solution tanks and storage hopper cleaning.

A very important aspect of plant operations is the promulgation of information concerning the safe handling of wastewater treatment chemicals. Careless handling of most chemicals used can cause eye injury, throat irritation and other problems.

33.6 Control

The amount of chemical added to the treatment process is a function of the dosage required to achieve the requisite degree of treatment and the quantity of flow. The dosage required varies with raw water quality and the degree determined by repeated bench/laboratory testing of the untreated water. As noted above, the results of the laboratory "jar" tests are compared to actual plant performance and these results used to select concentrations used in the treatment process. Also, data concerning variations in certain untreated critical water characteristics are recorded which are useful in determining actual dosage amounts. These include turbidity, BOD, suspended solids and settleable solids concentrations and pH. Flow is determined by taking repeated flow meter readings in conjunction with the laboratory tests.

Chemicals are fed in either dry or solution form. In either case, adequate controls are available to set proper feed rates. Manufacturers of dry feeders have equipped these units with various means to control feed rate including loss-in-weight type hoppers, adjustable speed belts, screws, scrapers and plungers and revolving disks. Solution feeders control chemical dosage rates by regulating the flow of a solution of known strength. In gravity feed systems, flow measurement is achieved using a constant head tank and a primary meter such as an orifice. In pumped systems, dosage rates are controlled by varying pump speed and/or displacement. Varying solution strength offers another means of control.

Automatic change in dosage with varying wastewater flow, varying strength, and effluent quality is possible but expensive.

33.7 Special Factors

The cost of sludge treatment and disposal is a very important consideration in the planning and design of treatment systems in which the use of chemical addition is contemplated. Chemical addition will increase the volume of sludge generated. For example, lime addition will generate from 1.0 to 1.5 lbs of dry solids per lb of lime added; powdered carbon will produce 1.0 lb per lb of carbon added; and the addition of ferric chloride will produce about 1.0 to 1.3 lbs of solids per lb of $FeCl_3$ added. Certain chemical sludges, i.e., alum sludges, in addition to being voluminous are difficult to dewater. Also, these sludges may contain high concentrations of the chemical used in treatment or of certain metals, e.g., cadmium, chromium, lead or copper, removed during treatment. This could render them unsatisfactory for use as fertilizers or soil conditioners. Care must be used in the siting of landfills for chemical sludges to insure that pollution of underground water sources does not occur. Recovery (regeneration) of chemicals used in wastewater treatment such as alum, lime and powdered carbon is practiced in the USA, but appears feasible only at large scale facilities (plant capacities > 50 mgd).

33.8 Recommendations

The process of addition of chemical coagulants to wastewater can provide BOD and SS removals of 65-80% and 80-90% respectively. Chemical addition will raise the efficiency of existing treatment facilities without appreciable construction cost. However, O&M cost may be high if the process is used on a full-time basis. The process is attractive where seasonal variations in treatment efficiency are required to meet receiving water quality standards. Also, if nutrient removal is required the process can produce an effluent with phosphorous concentrations as low as 0.1 mg/l. Removals of heavy metals is possible with lime addition and significant reductions in toxic organic compounds can be achieved using powdered activated carbon. The process is extremely well suited to the treatment/pretreatment of wastewater containing toxic or otherwise objectional industrial wastes.

34. GRANULAR ACTIVATED CARBON ADSORPTION

34.1 Description

The granular activated carbon (GAC) system is generally utilized for the removal of suspended and/or colloidal matter in wastewater and the removal of tastes and odors in water supply treatment. Generally, the applications for water supply use powered activated carbon. The GAC can also be used either as a tertiary treatment process in advanced wastewater treatment plants or as a secondary treatment process. This system is also used in conjunction with biological treatment processor or in independent physical/chemical treatment plants. See Figures 34.1 and 34.2 (2, 31).

Granular activated carbon can be used to upgrade water quality in existing sand filtration systems. Used as a complete replacement for sand or coal, activated carbons function as dual purpose media; providing both filtration and adsorption.

The advantages of this process are (31):

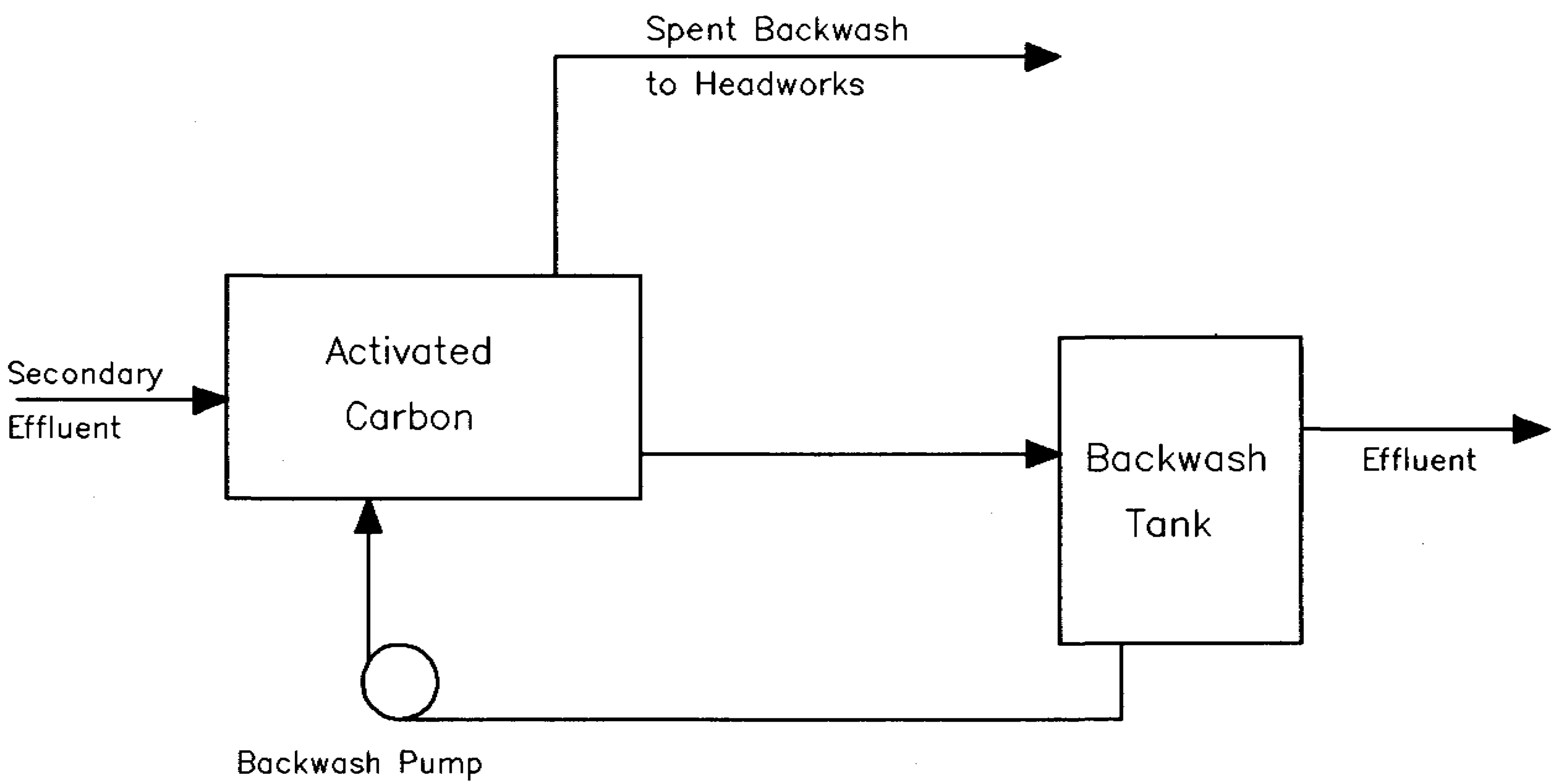
(A) Granular carbon adsorption is the most reliable taste and odor removal process.

(B) The reserve capacity of granular carbon can effectively control sudden water quality fluctuations and unexpected contamination.

(C) Granular carbon filter beds are generally more economical compared to powdered carbon.

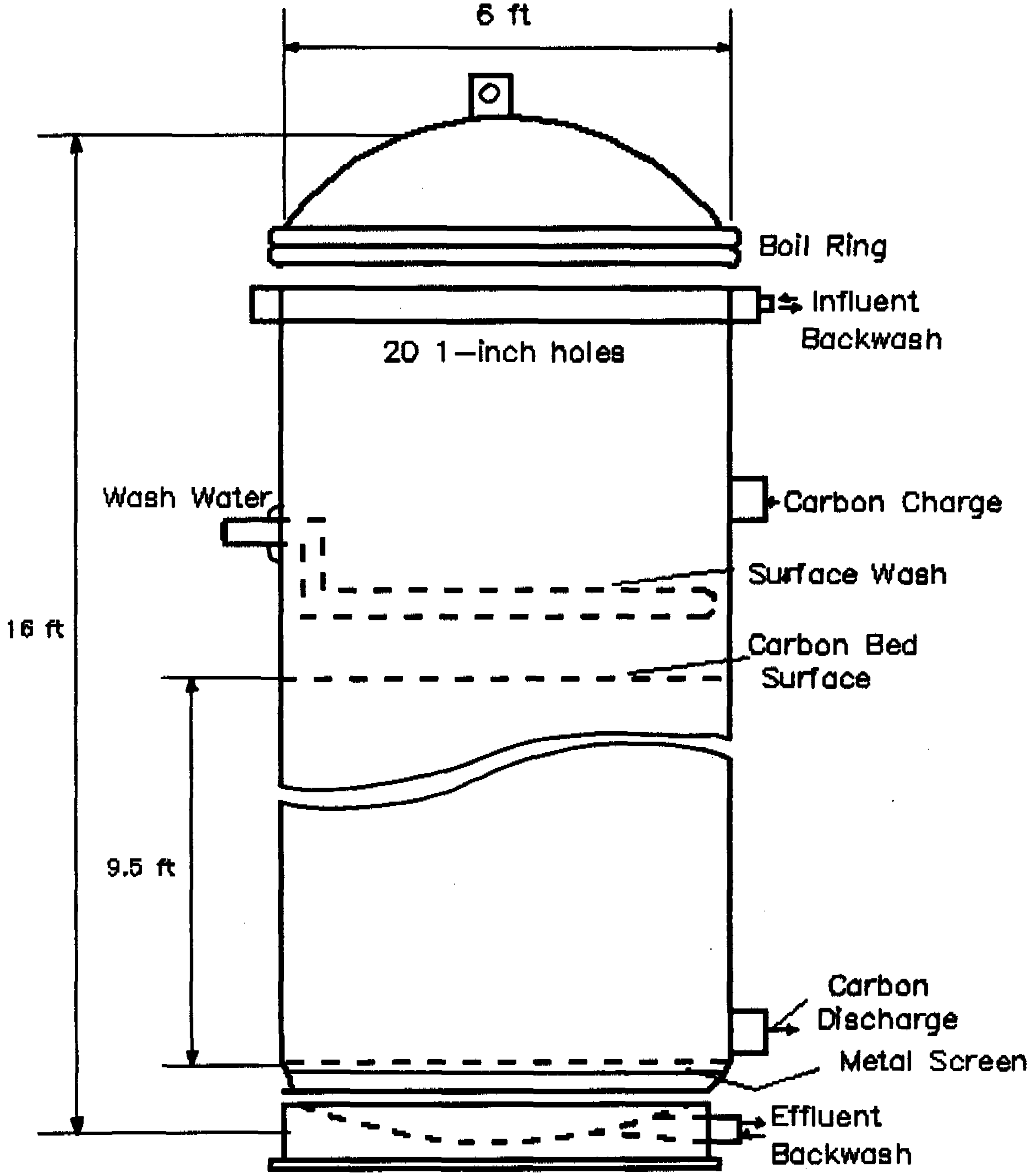
(D) Can be used for polishing organics in industrial wastewaters, accidental spills of toxic and hazardous chemicals and municipal water.

Figure 34.1: Flow Diagram of Granular Activated Carbon Adsorption



(Source: Ref. 2)

Figure 34.2: Downflow Type Granular Activated Carbon System.



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Water treatment with GAC consists of the carbon contact system and the carbon regeneration system. Activated carbon removes soluble suspended and colloidal matter from water in three steps:

(A) The first step is the transport of the dissolved substances to be removed (solute) through a surface film to the exterior surface of the carbon.
(B) The next step is the diffusion of the solute within the pores of the activated granular carbon.
(C) The last step is the adsorption of the solute on the interior surfaces bounding the pore and capillary spaces of the activated carbon.

Typically three to four alternative configurations for carbon contacting systems are used for treating wastewater -

(A) Downflow or upflow of the wastewater through the carbon bed.
(B) Parallel or series operation (single or multistage).
(C) Pressure or gravity operation in downflow systems (Figure 34.2).
(D) Packed or expanded bed operation in upflow systems.

Design Criteria

Typical granular activated carbon material performance and design criteria are listed in following tables:

Performance	Influent	Effluent
BOD mg/l	10 to 50	5 to 20
COD mg/l	20 to 100	10 to 50
FSS mg/l	5 to 10	2 to 10

Typical design parameters of granular activated carbon are as follows (2):

Size = vessels 2 to 12 ft diameter commonly used
Area loading = 2 to 10 gal/min/ft2
Organic Loading = 0.1 to 0.3 lb BOD5 or COD/lb carbon
Backwash = 12 to 20 gal/min/ft2
Bed depth = 5 to 30 ft.
Contact time = 10 to 50 min
Land area = minimal

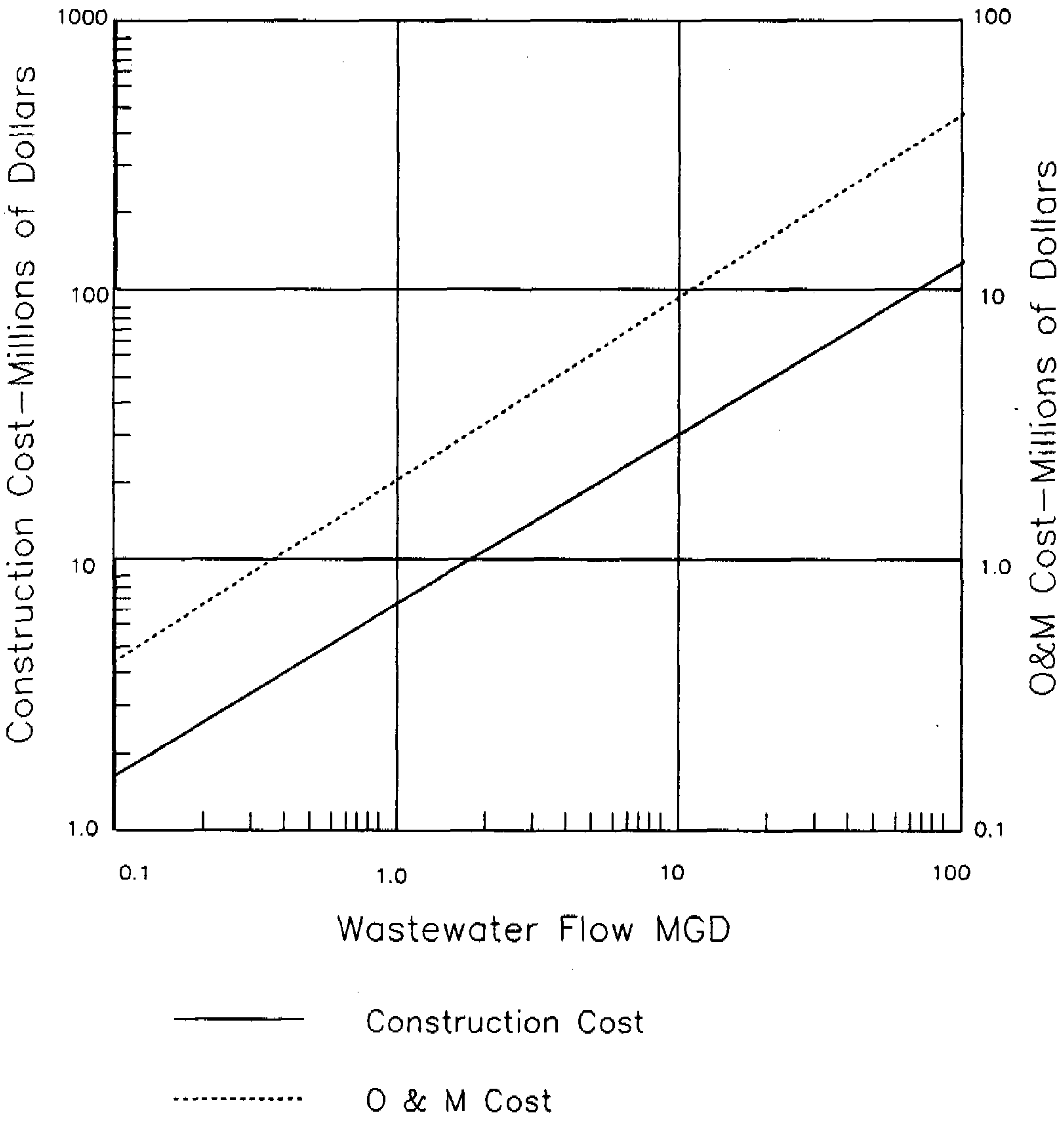
34.2 Limitations

Activated carbon adsorption systems are complicated to operate. Column operation involves cycling different columns into operation as carbon adsorption capacity is exhausted. Carbon regeneration is required for intermediate size systems for economical operation cost levels. Adsorption may be required for water reuse systems, where less costly systems such as lagoons, have been used beforehand. Also, adsorption may be required where toxic materials such as pesticides are in common use. Carbon adsorption have been used successfully for removal of halomethanes after disinfection.

34.3 Costs

Figure 34.3 (2, 11) shows construction costs (includes vessels, media, pumps, carbon storage tanks, controls, and operation building; loading rate 30 pounds carbon per Mgal, contact time = 30 minutes).

Figure 34.3: Construction, Operation & Maintenance Costs for Granular Activated Carbon Adsorption.



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34.4 Availability

Large shipments may lead to problems when the material has to be delivered to remote areas.

34.5 Recommendations

Carbon should only be used where trained personnel are available for operation continually. Removal of organics, especially toxic compounds is becoming a more important requirement universally.

35. DISSOLVED AIR FLOTATION

35.1 Description

Dissolved air flotation is a clarification process in which the raw liquid is pressurized to 60-80 psig. Air is dissolved into the liquid which is released to atmosphere pressure in the flotation cell. As the microscopic air bubbles come out of solution they attach to the suspended solids causing them to float to the surface. The concentrated solids impurities are scooped from the surface of the flotation cell. Dissolved air flotation can be used for municipal and industrial effluent treatment and for potable water treatment.

There are several commercially available clari-flocculators on the market. One is shown in Figure 35.1 (65). These systems include an advanced clarification system, using combined chemical flocculation, dissolved air floatation and rapid sand filtration in one unit.

Design Criteria

Retention time = 10 minutes

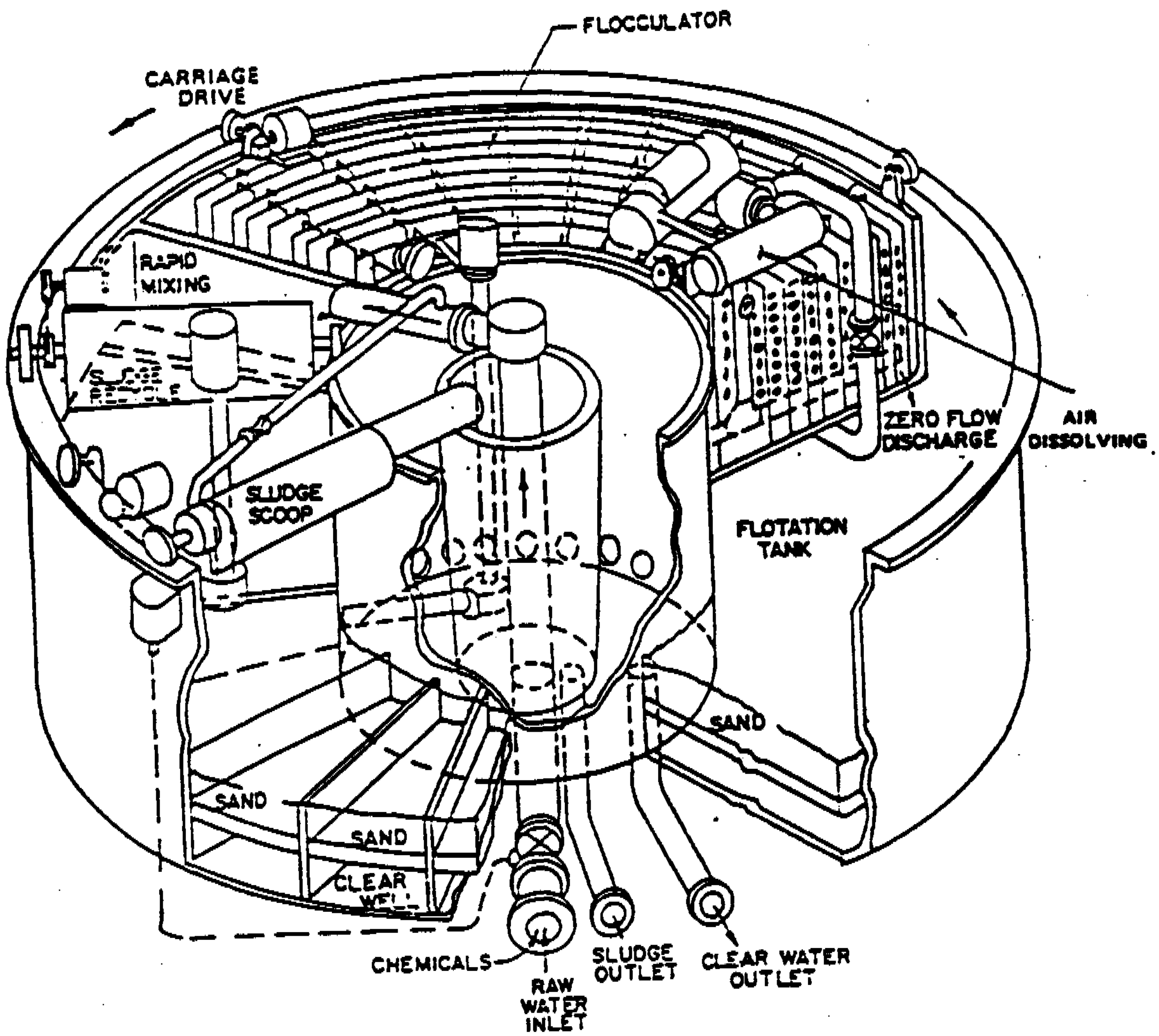
Discharge sludge concentration = 0.5 - 1.0%

A - Inside tank diameter	15 - 20 ft
B - Height of tank	6 ft
C - Height of carriage walkway	8 ft
D - Maximum height of equipment	13 ft
E - Minimum height of head clearance	14.5 ft
F - Tank depth	5 ft

35.2 Limitations

Additional dissolved air filtration thickener required for implant thickening and filterpress required for producing disposable sludge of 15-30% dryness.

FIGURE 35.1



Sandfloat Type SAF 22

35.3 Costs

See Figures 35.2 and 35.3 for construction and operation and maintenance costs, respectively.

35.4 Control

The operation is generally completely automatic. The outlet is connected to the distribution piping system and varying demand is automatically adjusted by the inlet flow regulating valve which maintains a constant level in the flotation tank. The Backwashing system in automatically operated by a clear well level control and a timer. The
SANDFLOAT installation is equipped with an alarm system that transmits a beeper signal via telephone and radio to an on call maintenance man. Normally one person visits the SANDFLOAT installation once a day for approximately one hour to perform water test and inspection.

35.5 Recommendations

Package treatment facilities are generally easy to operate and maintain. Often they are less costly (48). The effectiveness is usually dependent on the system internal operational reliability and not on separate systems which must operate together. On the other hand, the package systems generally have less flexibility for treatment of highly variable waste characteristics. Tests should be done using actual conditions.

Figure 35.2: Construction Cost for Dissolved Air Flotation

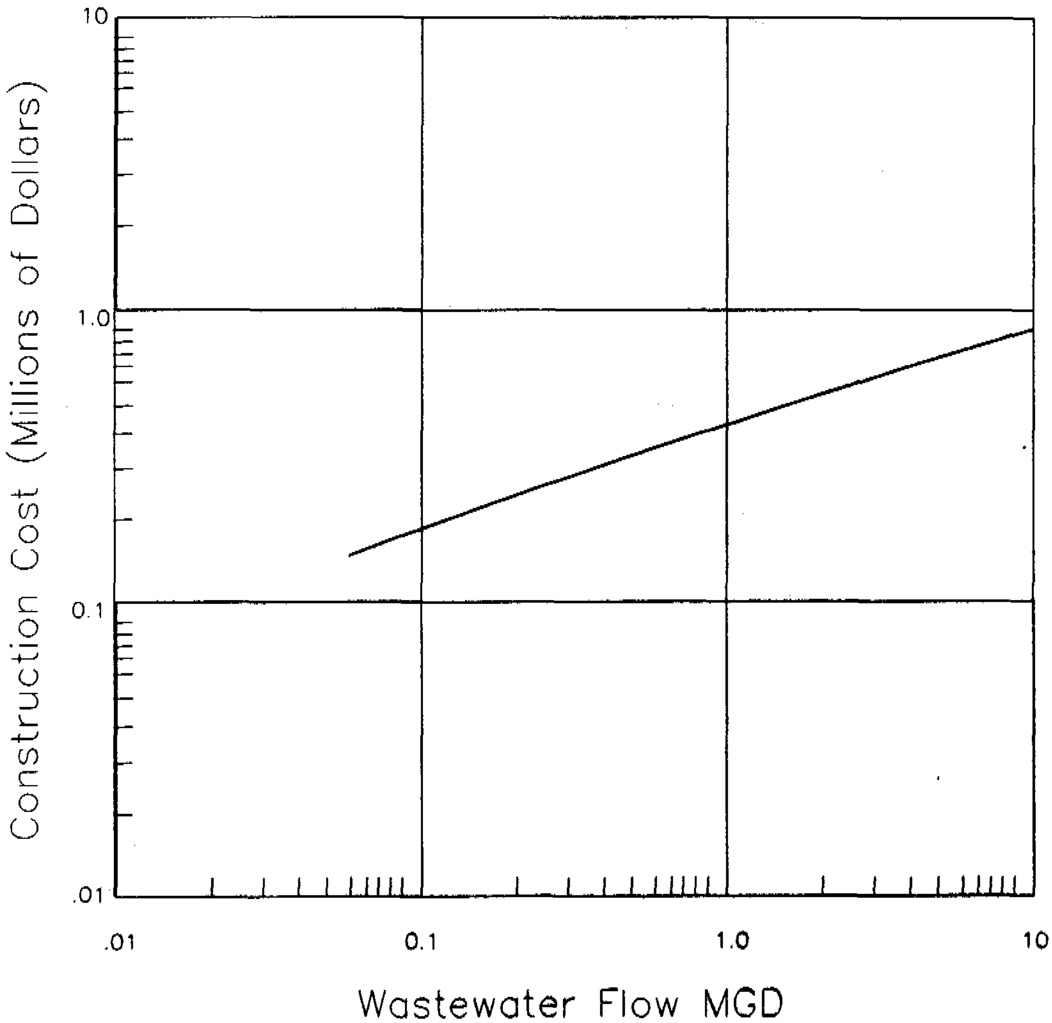


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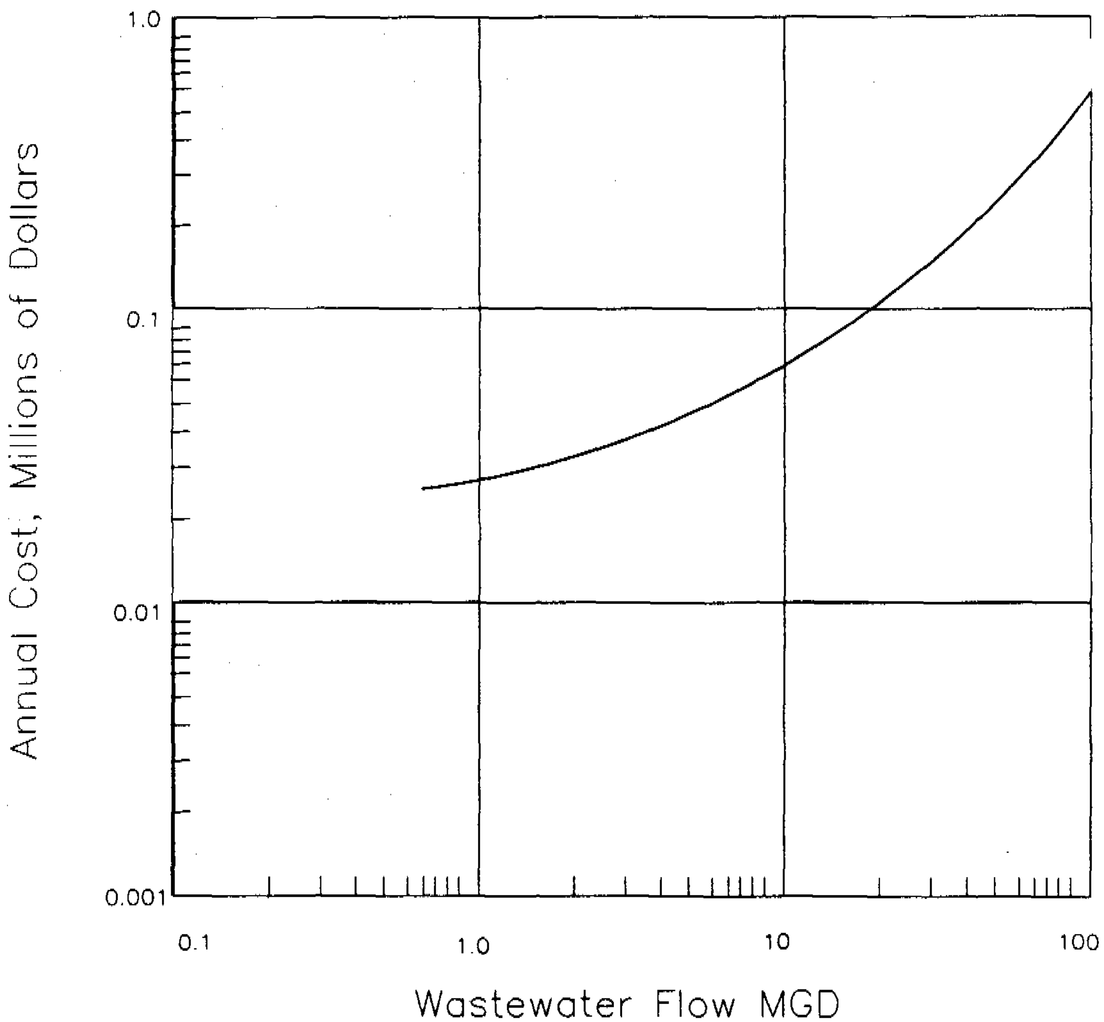
Figure 35.3: Operation & Maintenance Cost for Dissolved Air Flotation



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36. IMHOFF TANKS

36.1 Description

Imhoff tanks are two-storied tanks in which the lower story provides for sludge storage, and the upper one for sludge settling. The tanks may be either round or rectangular. Settling particles slide from the upper into the lower compartment (Figure 34.1). The two stories are so constructed that rising gas bubbles and sludge particles cannot enter the settling compartment. Rigid separation of sewage and sludge is enforced in Imhoff tanks. Thus, purtrescible solids are removed from the flowing sewage and stay removed. The entire sedimentation chamber is effective. Cleaning mechanisms are not needed.

Although the sludge chamber can be designed to hold sludge for but a few days, it is generally designed to serve as a digestion chamber and is given sufficient capacity to hold the sludge until it is well digested or until it can be dried or otherwise disposed of.

Sedimentation and digester compartments of Imhoff tanks are sized on the basis of conventional units. Normally multiple tanks are provided.

Sedimentation Chamber

Generally the sedimentation compartment of Imhoff tanks is designed with a maximum surface rating based on average dry-weather flow of 600 gpd per sq. ft. (25 $m^3/m^2/day$); the sloping bottom of the sedimentation compartment has 1.4 vertical to 1.0 horizontal slope, and the slot between upper and lower compartments has a minimum opening and a minimum overlap of 6 in. (15 cm). The sludge slots must overlap or otherwise bar the

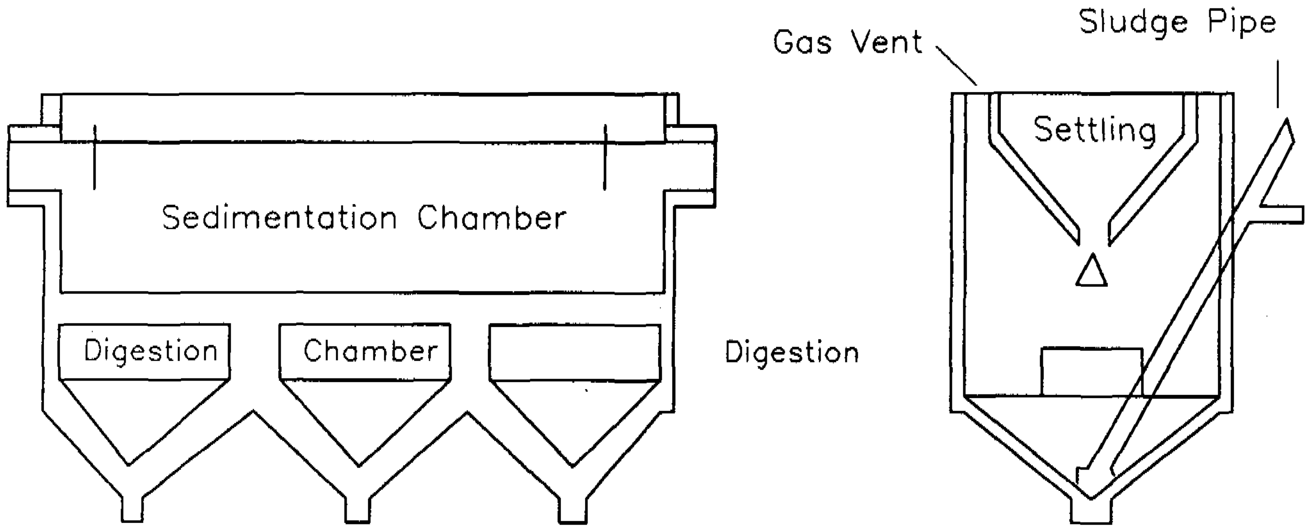


Figure 36.1: Imhoff Tank

(File: Mart66)

passage of rising sludge gases or sludge particles into the settling chamber.

Where gas is not collected, gas vents are given a minimum width of 18 in. (46 cm), an area equal to about 20% of the total superficial area of the tank, and a free board of 18 to 24 in. (46 cm to 61 cm) above the flow line of the tank. At least one vent should be 24 in. (61 cm) in diameter to facilitate entrance for tank repairs.

Baffles across the ends of the sedimentation chambers of Imhoff tanks are used to distribute the flow more uniformly and to serve as a trap to collect floating solids. Where the tanks are long, an extra beam is placed across the center of the tank to help support the sedimentation chamber and act as an intermediate baffle, collecting scum and preventing surface currents.

Inlet and outlet arrangements are particularly important because of the desirability of reversing the direction of flow through the tanks to give an even distribution of solids in the digestion chamber. A weir running the width of the sedimentation chamber is the most common arrangements as it serves effectively in distributing the flow and controlling outlet velocity. It is desirable to keep a relatively constant water level in the sedimentation chamber; therefore, the weir crest is set above the maximum flow line of the effluent channel.

A walkway along the tank to facilitate cleaning the sides of the sedimentation chamber and slot is usually provided for the operator.

Submerged drawoff pipes are installed in the flowing-through chambers and gas vents of some recent Imhoff tanks for the removal of skimmings and scum to sludge beds or lagoons, and have reduced the manual attention required.

Digestion Chamber

Capacity of anaerobic digestion chambers is the same as for unheated separate tanks.

The standards for digestion-tank loading vary between 1 and 4, cubic feet (0.11 m^3) per capita, depending on digestion temperature, climate, and operating conditions. Where data on the solids are not available, the foregoing unit per capita capacities are given for plants treating domestic sewage.

The digestion chamber (lower story)is generally subdivided by cross walls into several compartments. These walls make for structural economy and keep sewage from following a path of low resistance through the digestion chamber and fouling the effluent. Openings in the cross walls for equalization of sludge storage in successive compartments must lie below the normal sludge level. Each settling chamber should have its own sludge chamber. Otherwise, there will be cross currents in the lower story owing to unequal distribution of flow between parallel sedimentation chambers. These cross currents will drive shocks of septic digestion-tank liquor into the effluent. Longitudinal dividing walls should, therefore, be built into tanks that have more than one settling compartment.

The bottom slope of sludge compartments is generally 1 vertical to 1 or 2 horizontal.

Sludge is withdrawn through a central riser pipe controlled by its own valve. The pipe should have a free outlet at which the sludge can be seen and sampled. An irrigating or flushing ring at the mouth of the riser will improve sludge withdrawal.

Sludge drawoff from the digestion chamber is usually accomplished by utilizing the hydrostatic head, a differential of at least 6 ft. (1.8 m) being necessary.

Digestion compartments need no outlets to surface other than the gas stacks. The entire tank surface is effective for sedimentation. If sludge slots are kept high, tank depth is utilized to the fullest extent for digestion and tank surface for settling.

As noted above, an area equal to 20% of the total surface area of the tank is normally provided for venting gas from the digestion compartment. Large Imhoff tanks are sometimes equipped for gas collection. This adds little to their cost of construction, because the slabs that separate the upper and lower stories form the necessary gas-collecting covers (Figure 36.1). Vertical stacks penetrate to the digestion compartment and house gas domes. These end about 1 ft. below the water surface. Scum is kept out of them by porous concrete or wooded slabs. Scum can then be broken up by water or mechanically. Heavy layers of scum can be drawn off through gated, lateral openings or skimmed off.

Variation in Design

A variation of the conventional type is a two-story mechanized clarifier superimposed over a mechanized digester with concrete tray between the two compartments. A one-way seal permits sludge to pass downward from the settling to the digestion compartment but prevents gas, scum, or warm liquor from passing upward. Gas from the digestion compartment is led off through an opening at one side to a gas dome and take-off located above the water level. Heating of the digestion compartment is practiced in colder climates utilizing outside fuel and circulating hot water through internal pipe coils or sludge through an external heat exchanger. Radiation losses through the concrete tray are moderate because in operation a layer of scum packs against the underside and acts as an insulating medium. Digestion temperatures of 85° to 90° F have been maintained.

36.2 Advantages/Disadvantages

Advantages for developing countries are: relatively low construction cost using manual labor, simplicity of design and operation, unskilled maintenance labor, no chemicals required, power is not required, large quantities of wash water are not required, sludge disposal is simpler. Disadvantages are: impracticability of heating digestion compartment in colder climates because heat is lost through slot and gas vents; tanks must be isolated from populated areas because digestion gases and other odors are vented to the atmosphere; tanks are required deep which is more costly construction than modernized tanks.

36.3 Cost

Estimated capital and O&M costs for Imhoff Tanks, exclusive of land and special foundations are presented in Figure 36.2.

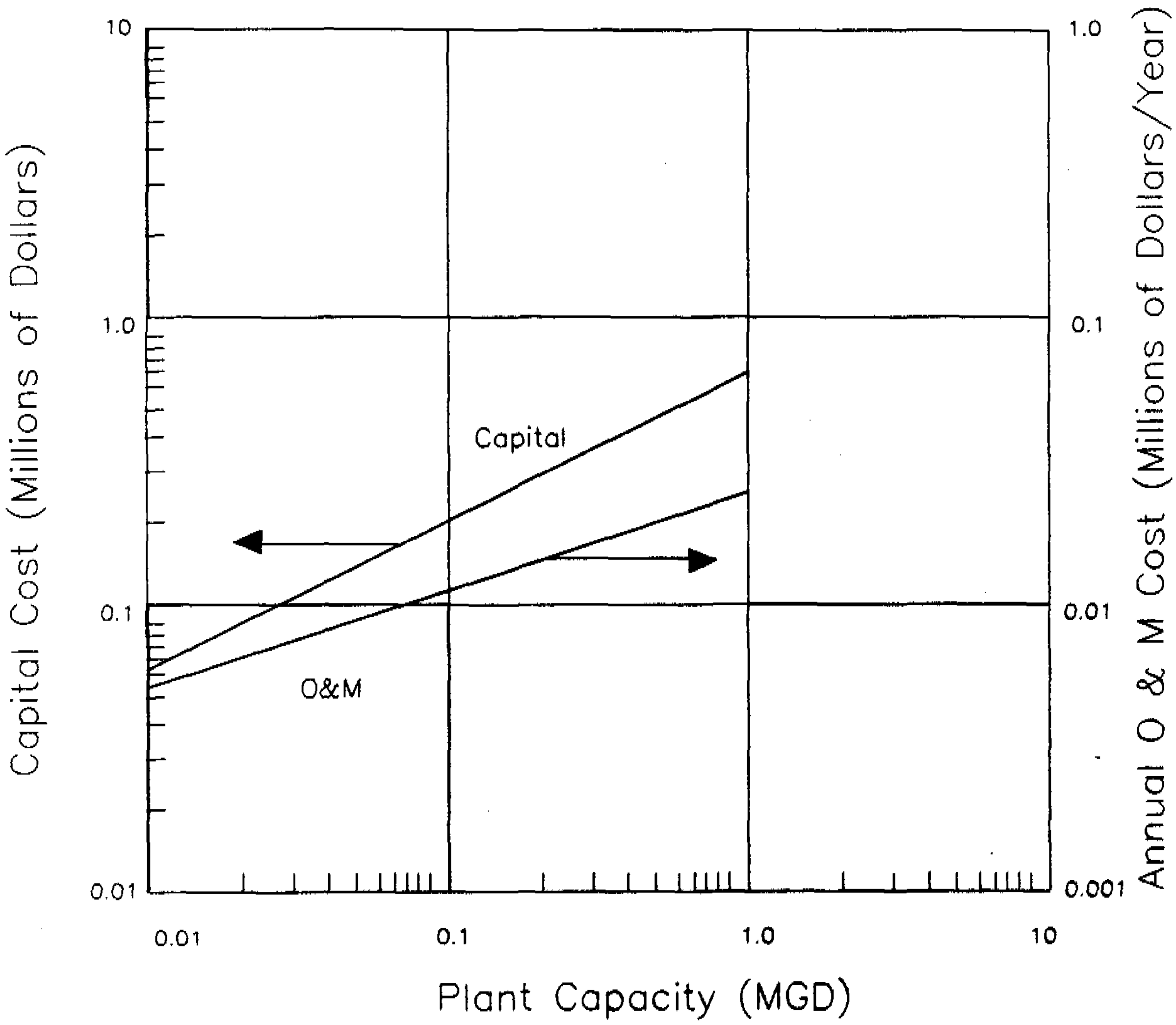
36.4 Availability

Technology/components readily available in developing countries.

36.5 Operation and Maintenance

Digesting sludge or scum must not be permitted to reach the slots. There should be a neutral zone of at least 18 in. both below and above them. The upper neutral zone offers protection against excessive scum formation. To maintain the neutral zone, sludge and scum must be withdrawn, even if they are not fully digested. Appearance of a row of gas bubbles on the sewage surface above the slots is a sign that sludge has encroached on them. Sufficient sludge must be left in the tank as a seeding and buffering reserve. Repeat breaking-in periods should be avoided.

Figure 36.2: Imhoff Tank Costs *



* Costs Are Exclusive of Land & Special Foundation

(File: Mart64)

Sludge-digestion tanks will foam when there are too many foaming substances in the sludge liquor. Rising gas bubbles then drive the foam into the gas stacks, sometimes to overtop them. As a rule, foaming is confined to breaking-in periods. It can be controlled by flushing the stacks with clean water or sewage and so diluting the foaming sludge liquor. Sludge withdrawal has much the same effect, because it lowers the sludge level and pulls in fresh sewage from the settling chamber. The use of anti-foaming agents is a possibility.

36.6 Control

Opportunities process control in a conventionally designed Imhoff tank are somewhat limited. Maintenance of a clear zone above and below the sedimentated compartment slots is essential for good clarifier performance. Thus apparatus/devices/means for locating the top of the sludge and bottom of the scum layers would improve the sedimentated process performance. One approach to locating the limits of the clear zone is by lowering a pitcher pump suction into the digestion compartment.

Controls to govern the withdrawal rate and amount of sludge removed from the digestion compartment will improve that process performance. A means of tracking the location of the top of sludge layer will insure that enough sludge is left for seeding thereby improving system performance.

Simple controls to govern the rate of sludge removal will help to maximize the solids content of the sludge withdrawn from the tank.

36.7 Special Factors

Imhoff tanks should be preceded by grit removal to reduce the frequency of digester compartment cleaning as the result of excessive accumulation of grit in these units. Also, measures for sludge diverting (drying) and disposal must be provided as part of the treatment system.

36.8 Recommendations

Imhoff tanks with ample design capacity can provide SS and BOD removal efficiencies of from 50-75% and 25-30% respectively. Imhoff tanks are suitable for use as a primary treatment step preceding oxidation ponds, intermittent sand filters, subsurface filters, leaching systems and trickling filters. They could be suitable for pretreatment before ocean discharge. They could be used for treatment of some industrial wastes.

37. ROUGHING FILTERS

37.1 Description

Roughing filters are used predominantly for wastewater treatment, but can also be used in water treatment (see Figure 37.4). They allow deep penetration of suspended materials into a filter bed, and have a large silt storage capacity. The solid materials retained by the filters are removed by either flushing or excavating the filter media, washing it, and replacing it. Roughing filtration uses much larger media (more than 2.0mm diameter) than either slow or rapid filter media (0.15 to 0.35 and 0.4 to 0.7 mm diameter). In this sense the roughing filter operates much like a trickling filter. Indeed, the biological activity has been observed to be much the same, including a potential for nitrification of wastewater (4). The rate of filtration can be as low as those used for slow sand filters or higher than those used for rapid filters, depending on the type of filter, the nature of the turbidity, and the desired degree of turbidity removal. There are basically two types of roughing filters, which are differentiated by their direction of flow: vertical flow and horizontal flow (Figs. 37.2 and 37.3). Vertical flow roughing filters are further subdivided into upflow (Fig. 37.1) and downflow units. (4, 25, 34)

Vertical Flow

Design Criteria and performance expectations for two types of wastewater vertical flow roughing filters are provided in Tables 37.1, 37.2, and 37.3.

Gravel media upflow units consist usually of several gravel layers tapering from a coarse gravel layer located directly above the underdrain system, to successively fine gravel layers to permit deep penetration of suspended solids into the filter bed.

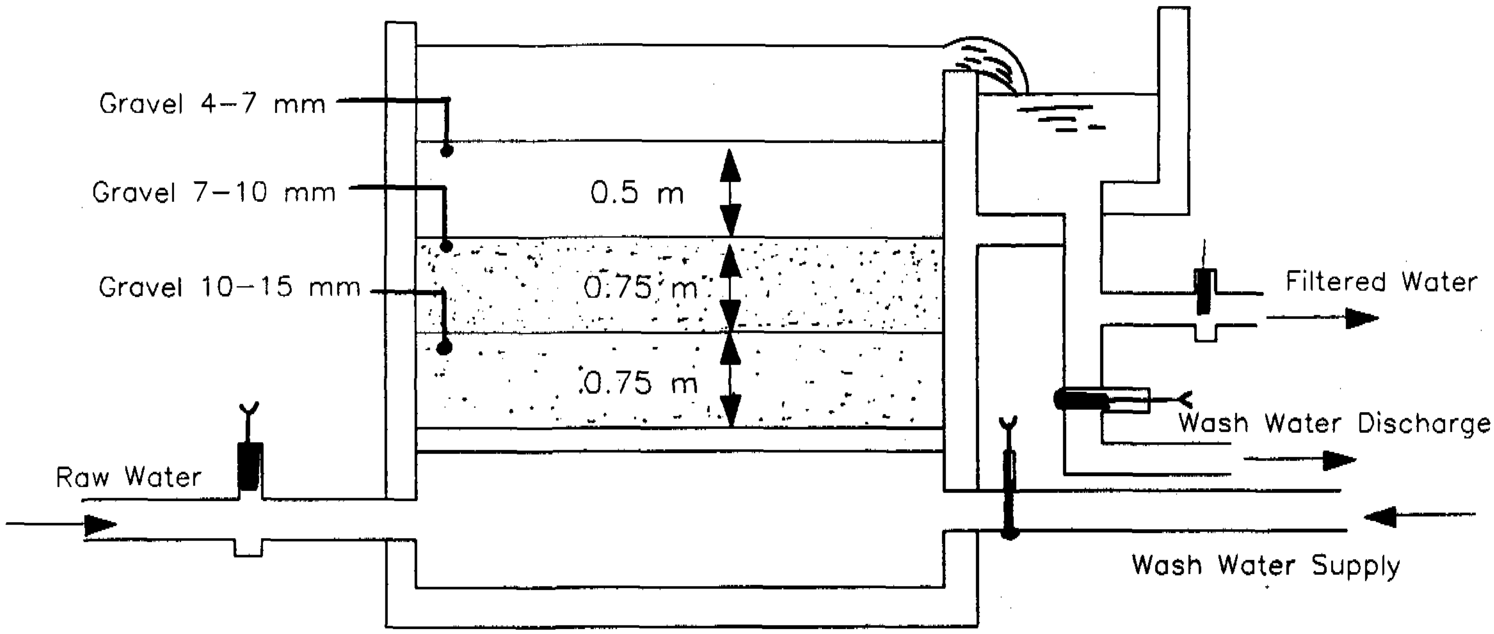


Figure 37.1: Gravel Upflow Roughing Filter.

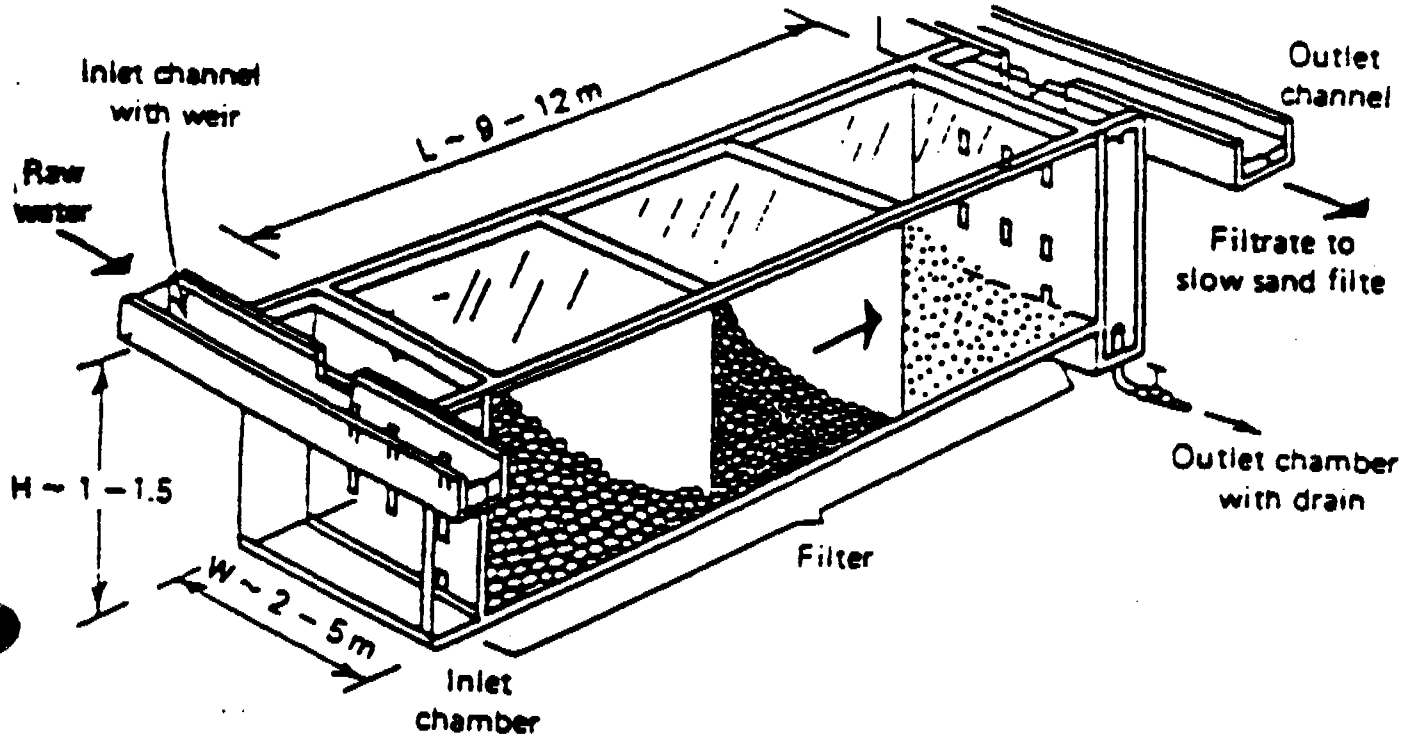
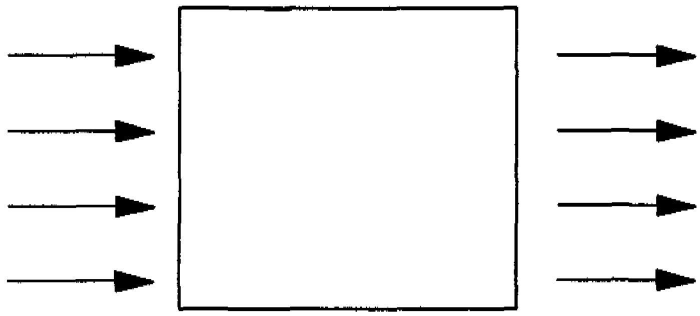


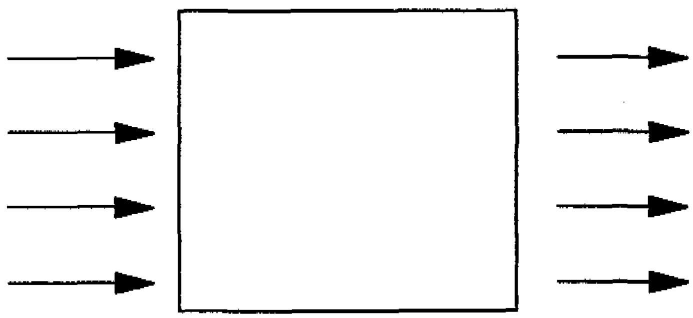
Figure 37.2: Basic Features of a Horizontal–Flow Roughing Filter.

(Source: Ref. 25)

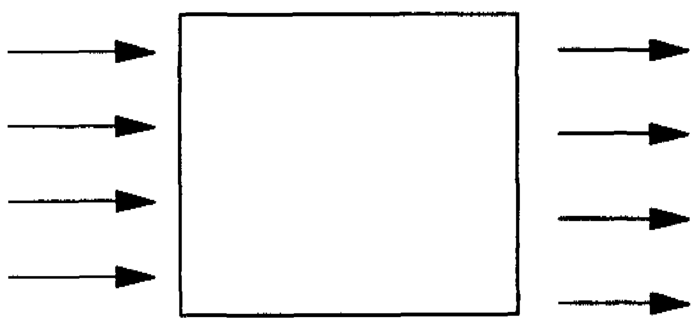
Mechanism of Horizontal–Flow Roughing Filter
Figure 37.3



HRF acts as a multistore sedimentation tank



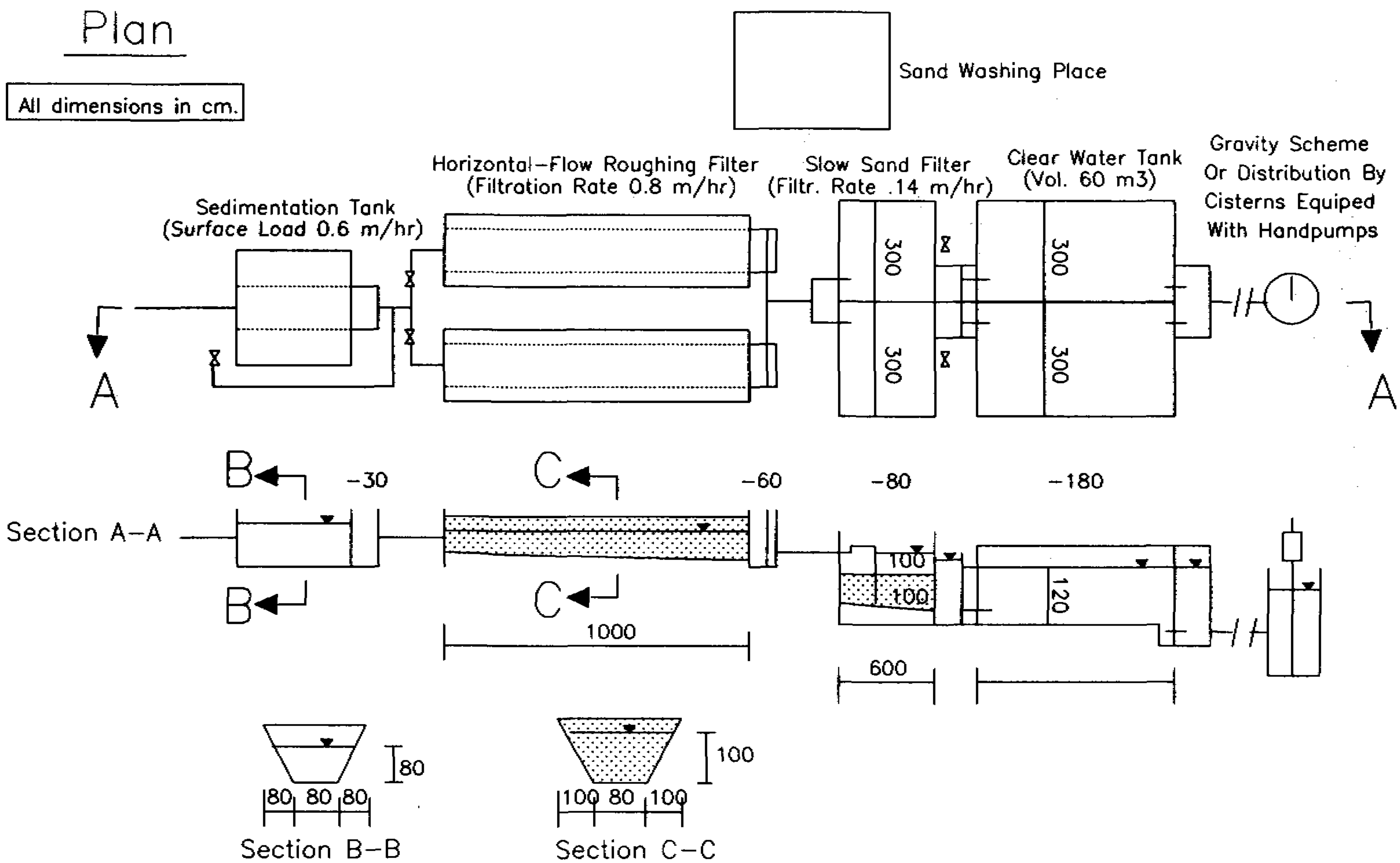
Accumulation of solids on the upper collector surface



Drift of separated solids to the filter bottom

(Source: Ref. 34) (File: Martin62)

Figure 37.4: Lay-Out Of Water Treatment Plant For 60 M3/Day (2000 People at 30 l/c.d.)



(Source: Ref. 34)

(File:Martin63.drw)

TABLE 37.1

TYPICAL WASTEWATER DESIGN PARAMETER FOR GRAVEL MEDIA ROUGHING FILTERS

Parameter	Filter Type: Vertical (Fig. 37.1)	Horizontal (Fig. 37.2)
Filtration rate (m/hr)	Up to 20	0.5 - 4.0
Filter Thickness (mm):		
Coarse	10 - 15	-
Medium	7 - 10	-
Fine	4 - 7	-
Total Filter Thickness (mm)	21 - 32	4-40
Sequence of filter arrangement	Vertical	Longitudinal
Length of Filter Media (m):		
Coarse	-	4.5 - 6.0
Medium	-	3.0 - 4.0
Fine	-	1.5 - 2.0
Total Length of filter (m)	-	9 - 12
Filter side wall height (m)	-	1 - 1.5
Filter Slope	-	In direction of flow (1% grade)
Discharge measurement	-	V-notch weir
Removal Efficiency (%)	60-80	60 - 70
Influent water turbidity	<150 best if <50	30 - 100

Source: Reference 25

TABLE 37.2

TYPICAL WASTEWATER DESIGN PARAMETERS FOR COCONUT FIBER MEDIA ROUGHING FILTERS

Parameters	Vertical Only
Filtration rate (m/hr)	1.25 - 1.5
Filter thickness (mm)	60 - 80
Sequence of filter arrangement	vertical
Filter side wall height (m)	-
Filter slope	-
Discharge measurement	-
Removal efficiency (%)	60 - 90
Influent water turbidity (NTU)	<150

Source: Reference 25

TABLE 37.3

Performance of Horizontal-Flow Settling Basins in Columbia (1959)

Location of Water Treatment Plan	Detention Period (hr)		Settling Velocity (m/day)		Temperature (°C)		Influent Turbidity (NTU)		Effluent Turbidity (NTU)	
	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.
Cali	5.05	3.92	22.2	17.3	18	2	600	5	8	2
Pasto	3.96	3.1	27.2	21.3	14	14	120	5	4	0.9
Pereira	2.66	2.13	35.4	28.2	18.5	18.5	130	7	9	4
Santa Marta	3.31	1.83	50.2	27.6	28	28	4690	59	6	2

Source: Reference 19

Where coconut is abundant, a low cost filter medium of shredded coconut fibers has been employed and proven successful. Small villages in Thailand and Southeast Asia have found the following very useful:

Shredded coconut fiber may be prepared manually by soaking the husk for 2 to 3 days in water, then shredding the husk by pulling off the individual fibers and removing the solid particles which bind the biers. Shredded coconut fibers may also be purchased directly from upholstery stores or coir (coconut fiber) factories. The shredded fiber should be immersed in water for about three days, until the fiber does not impart any more color to the water.

Several small filter plants ranging in capacity from 24 to 360 m^3/day were constructed from 1972 to 1976 in the Lower Mekong River Basin countries (Thailand, Viet Nam, Cambodia) and in the Philippines. Two-stage filtration, using shredded coconut fibers and burnt rice husks for the roughing and polishing filter, respectively, was typical for all filter plants. (25)

Horizontal Flow

Horizontal flow roughing filter have a large silt storage capacity because of their coarse filter media and long filter length. For overall efficiency it is best to use a graded gravel scheme for the filter medium. The horizontal flow filter is usually divided into several zones, each with its own uniform grain size, tapering from large sizes in the initial zone to small sizes in the final zone. In this way, penetration of suspended solids will more easily take place over the entire filter bed and result in longer filter runs.

Horizontal flow roughing filters may also be constructed adjacent to a stream bed so as to allow raw water to flow through a porous-stone wall and into a gravel bed. The drain system is made of a perforated PVC pipe that leads to a junction box. To avoid the infiltration of surface runoff, an impermeable layer of clay or a polyethylene liner can be placed over the gravel bed. This particular design has a capacity ranging from 85 to 860 m^3/day, and is intended to operate at a filtration rate of 0.5 m/hr. It can treat waters of turbidities less than 150 NTU prior to slow sand filtration. The length of the filter is variable.

37.2 Limitations

Roughing filters are limited, because higher turbidities (greater than 150 NTU) will result in frequent clogging of filter media and cause discontinuous operation. Higher solids loadings may be used if cleaning is practiced regularly. In the case of wastewater, treatment performance is not considered equivalent to say activated sludge.

37.3 Costs

The construction costs, and operation and maintenance cost of vertical flow and horizontal flow roughing filters are roughly the same as trickling filters, depending on the type of construction.

37.4 Availability

Roughing filters are a very economical method of sewage treatment for smaller communities. Several water treatment plants exist in South and Central America (See Table 37.2). They may be used for high rate and intermittent treatment applications of any kind, including stormwater treatment, sewage and industrial waste pretreatment, and other applications.

The availability of the raw coconut husks at low cost, as well as the elimination of backwash pumps and ancillary equipment, combine to make this manual filter bed regeneration process economical in areas where coconut trees are common. The use os such indigenous materials for filter media is also a practical alternative to conventional filter design.

37.5 Operation and Maintenance

Filtration rates in gravel upflow filters are relatively high because of the large pore spaces in the filter bed that are not likely to clog rapidly. Low backwashing rates are used because no attempt is made to expand the bed; but longer time periods for adequate cleaning of the gravel are usually necessary (about 20 to 30 minutes).

There are no backwashing arrangements for cleaning the coconut fibers, because the fibers do not readily relinquish entrapped particles due to their fibrous nature. Instead, waster is drained from the filter box and the dirty fibers are removed and discarded. Coconut fiber stock, which has been properly cleaned, is then packed into the filter. The filter medium generally must be replaced every three or four months.

For horizontal roughing filters, frequent cleanings are required when the filter lengths are short. Short filter lengths could be used, especially in areas of cheap labor costs, such as Latin America. On the other hand, if filter beds are long enough, 2-5 years can go by before cleaning.

Structural constraints limit the depth of the filter bed in VF filters, but higher filtration rates and backwashing of the filter media are possible. On the other hand, HF filters enjoy practically unlimited filter length, but normally are subject to lower filtration rates, and they generally require manual cleaning of the filter media.

37.6 Special Factors

To use roughing filters effectively, the raw wastewater characteristics and treatment objectives should be clearly defined.

The filter length is the most critical dimension in the design of horizontal flow roughing filters and should be selected after considering an appropriate balance between construction costs and the frequent cleanings required when filter lengths are short.

Upflow filters are used predominantly in upflow-downflow type filtration to replace the unit process of flocculation and sedimentation found in conventional rapid filtration plants. They are similar in design and construction to gravel bed flocculators.

Roughing filters are often used before slow sand filters because of their effectiveness in removing suspended solids.

37.7 Recommendations

Roughing filters are very effective treatment of sewage wastewater when turbidity of 20 to 150 NTU, so that it can prevent too frequent clogging and to ensure continuous operation for an extended period of time. Roughing filters can also be applied to water treatment. They should be considered for wide application in Latin America.

38. ROTATING BIOLOGICAL CONTACTORS (RBC)

38.1 Description

The process is a fixed film biological reactor consisting of plastic media mounted on a horizontal shaft and placed in a tank. A general layout is shown in Figure 38.1 (2). Common media forms are discs made of styrofoam and a denser lattice type made of polyethylene. While wastewater flows through the tank, the discs are slowly rotated. The discs are about 40 percent immersed in contact with the wastewater. Organic matter is removed by the biological film that develops on the media. The removal is similar to any other fixed film media biological unit such as trickling filter. Rotation results in exposure of the film to the atmosphere as means of aeration. Excess biomass on the media is stripped. Solids are maintained in suspension by the mixing action of the rotating media. Multiple staging of RBC's increases treatment efficiency and could aid in achieving nitrification year round. A complete system generally consists of two or more parallel trains with each train consisting of multiple stages in series (See Figure 38.1)

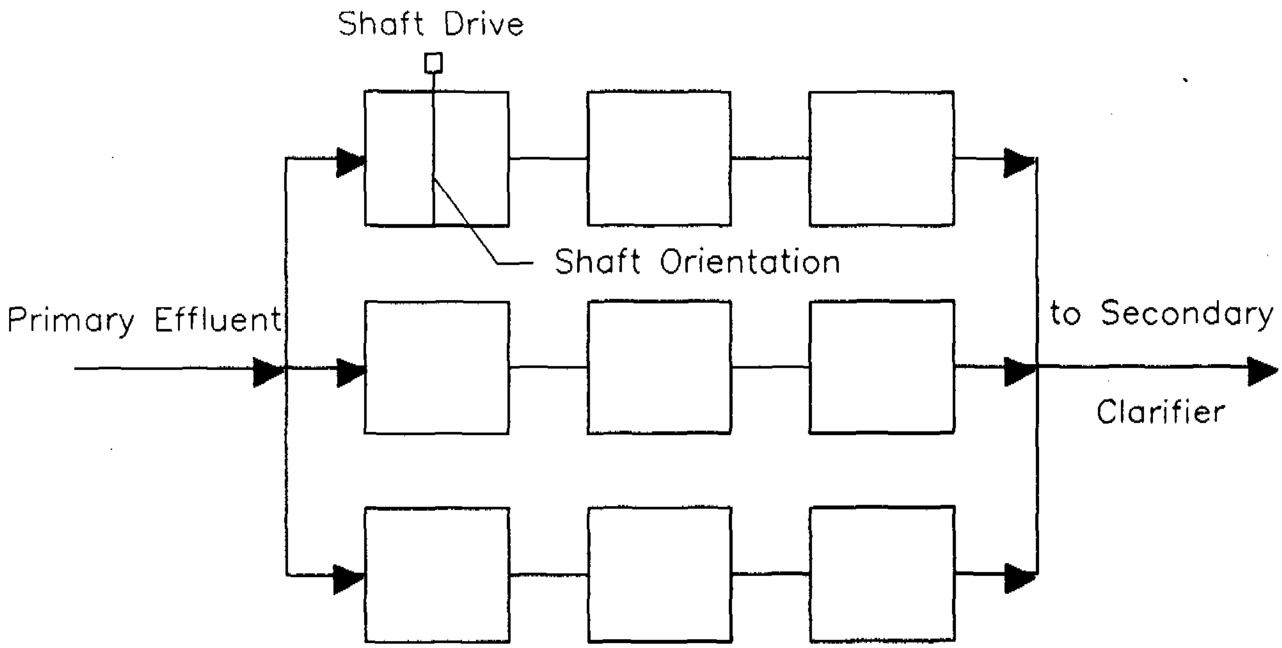
The typical design criteria for RBC technology is shown in Table 38.1 (2, 35, 36, 37, 38, 39).

38.2 Limitations

Performance of this process may fall off significantly at temperature below 55 degrees F, if the biological media is exposed directly to the ambient air. The high organic loadings can result in first stage septicity and supplemental aeration may be required. Use of dense media for early stages can result in media clogging.

Alkalinity deficit can result from nitrification; supplemental alkalinity source may be required.

Figure 38.1: Typical Staged RBC Configuration.



(Source: Ref. 2)

TABLE 38.1

A TYPICAL DESIGN PARAMETER OF RBC

Parameter		Typical Valve Range
Organic loading		
(without nitrification)	=	30 to 60 lb BOD_5/d/1000 ft^2 media
(with nitrification)	=	15 to 20 lb BOD_5/d/1000 ft^2 media
Hydraulic loading		
(without nitrification)	=	0.75 to 1.5 gal/d/ft^2 media
(with nitrification)	=	0.3 to 0.6 gal/d/ft^2 media
Number of stages/train	=	1 to 4 depending upon treatment objectives
Number of parallel trains	=	Recommended at least 2
Rotational Velocity Peripheral velocity	=	60 ft/min for mechanically drivers 30 to 60 ft/min for air drivers
Typical media surface area		
(Disc type)	=	20 to 25 ft^2/ft^3
(Standard Lattice type)	=	30 to 40 ft^2/ft^3
(High density Lattice type)	=	50 to 60 ft^2/ft^3
Percent media submerged	=	40%
Tank volume	=	0.12 gal/ft of disc area
Detention time based on 0.12 gal/ft^2		
(without nitrification)	=	40 to 120 minutes
(with nitrification)	=	90 to 250 minutes
Secondary clarifiers overflow rate	=	500 to 800 gal/d/ft^2
Hourne power	=	3 to 5 consumed/25ft shaft 5 to 7.5 connected/25 ft shaft

Source: References 2,35,36,37,38,39).

38.3 Costs

Costs are based on including RBC shafts (standard media 100,000 ft squared/shaft), motor drives (5 hp/shaft), molded fiberglass covers, and reinforced concrete basins. Operation and maintenance cost includes power, labor, material. See Figure 38.2

38.4 Availability

The process has been in the use in the United States since 1969 and may thus be considered ready for wide spread application. It should work well in warm climates. Because of its characteristic modular construction, low hydraulic head loss and shallow excavation, which make it adaptable to new or existing treatment facilities, its use is growing (38).

38.5 Operation and Maintenance

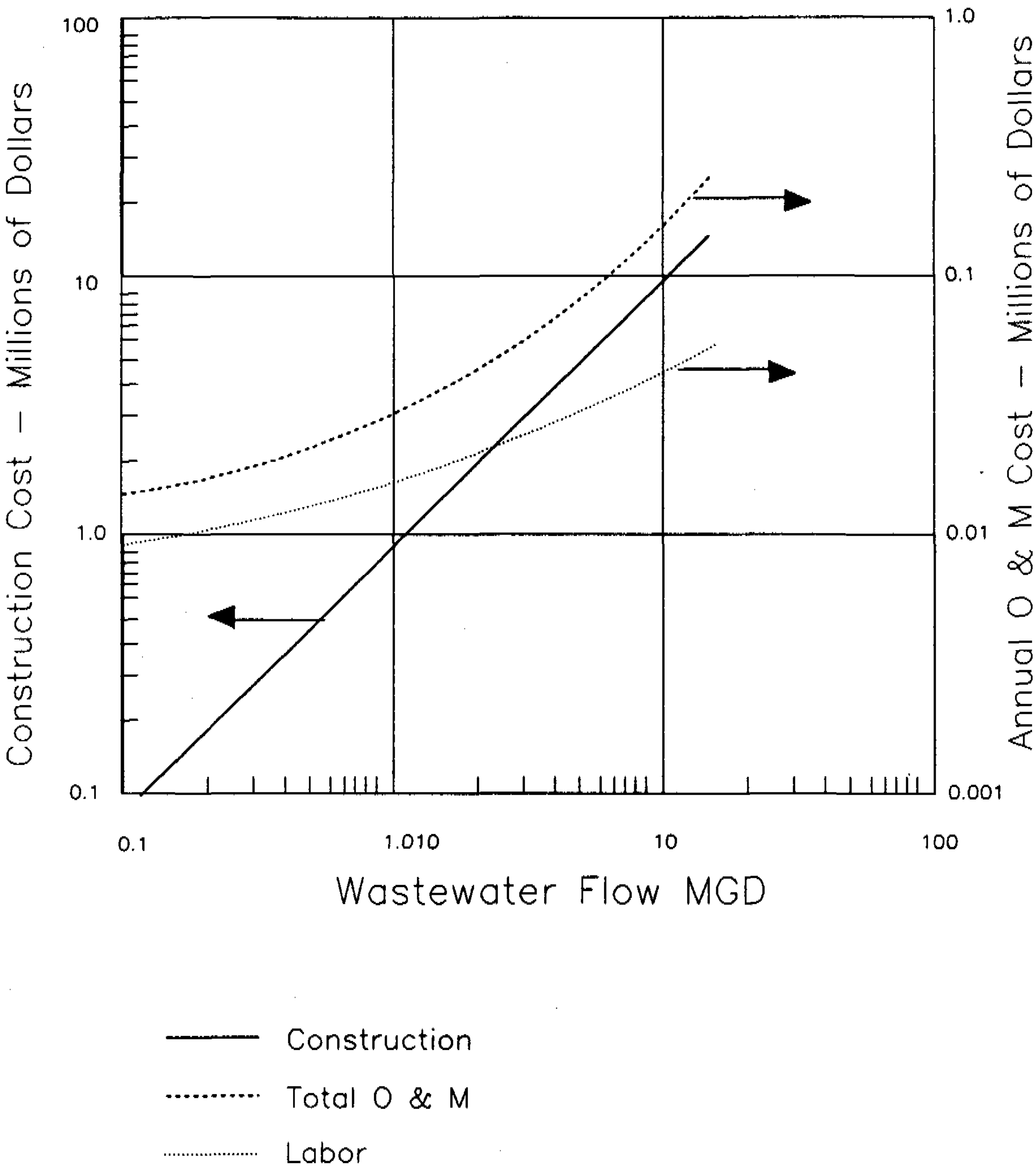
Ease of maintenance should be provided by considering the following:

1) Ease of access to shaft, media, and other mechanical equipment needing inspection/maintenance and possible periodic removal/replacement.

2) Use of self-aligning, moisture resistant bearings.

Rotating biological contactors have few moving parts and require minor amounts of preventive maintenance. Chain drives, belt drives, sprockets, rotating shafts and any other moving parts should be inspected and maintained in accordance with manufacturers' instructions. All exposed parts, bearing housing shaft ends and bolts should be painted or covered with a layer of grease to prevent rust damage. motors, speed reducers and all other metal parts should be painted for protection.

Figure 38.2: Construction, Operation & Maintenance Costs for Rotating Biological Contactors



(Source: Ref. 2 & 5)

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Maintenance also includes the repair or replacement of broken parts. A preventive maintenance program that keeps equipment properly lubricated and adjusted to help reduce wear and breakage requires less time and money than a program that waits for breakdowns to occur before taking any action. The frequency of inspection and lubrication is usually provided by manufacturer's instructions.

Properly designed systems have sufficient turbulence so solids or sloughed slime growths should not settle out on the bottom of the bays. If grease balls appear on the water surface in the bays, they should be removed with a dip net or screen device.

38.6 Control

RBC units perform most effectively under conditions of low hydraulic loadings and organic loadings, compared to other biological treatment processes. For instance, four stage system with final clarifier and preceded by primary treatment, the percent removal of BOD5 = 80 to 90%, suspended solids = 80 to 90%, phosphorous = 10 to 30%, and NH_3-N up to 95%.

38.7 Special Factors

There are many special factors to be considered when considering the RBC system which include longer retention time (8 to 10 times longer than trickling filters), and less susceptibility to upset from changes in hydraulic or organic loading than conventional activated sludge. While loadings must be lower, unit costs are significantly lower as well.

38.8 Recommendations

RBC technology may be considered for applications in warm climates. Construction costs are lower per unit of treatment capacity purchased. Operational efficiency is more consistent than other biological processes and maintenance is simpler.

39. ACTIVATED SLUDGE TREATMENT

39.1 Description

Activated sludge treatment is used to remove dissolved and colloidal biodegradable organics. The technology is a continuous flow, biological treatment process characterized by a suspension of aerobic micro-organisms, maintained in a relatively homogeneous state by the mixing and turbulence induced aeration. The micro-organisms are used to oxidize soluble and colloidal organics to carbon dioxide (CO2) and water (H2O) in the presence of molecular oxygen. The process may or may not proceed primary sedimentation process. The mixture of micro-organisms and wastewater (called mixed liquor) formed in the aeration basins is transferred to secondary classifiers following treatment for liquid and solid separation. The major portion of the micro-organisms settling out in the secondary clarifiers is recycled to the aeration basins to be mixed with incoming water, while the excess which constitutes the waste sludge, is sent to the sludge handling facilities. The rate and concentration of activated sludge returned to the aeration basins determines the mixed liquor suspended solids (MLSS) level developed and maintained in the basins. During the oxidation process, a certain amount of the organic material is synthesized into new cells, some of which then undergoes auto-oxidation (endogenous respiration) in the aeration basins, the remainder forming net growth or excess sludge. Oxygen is required in the process to support the oxidation and synthesis reactions. Volatile compounds are partly removed in certain extent in the aeration process. Metals will also be partially removed, with accumulation in the sludge.

Activated sludge systems are classified generally as high rate, conventional, or extended aeration (low rate), based on organic loading. In the conventional activated sludge plant, the wastewater is commonly aerated for a period of 4 to 8 hours

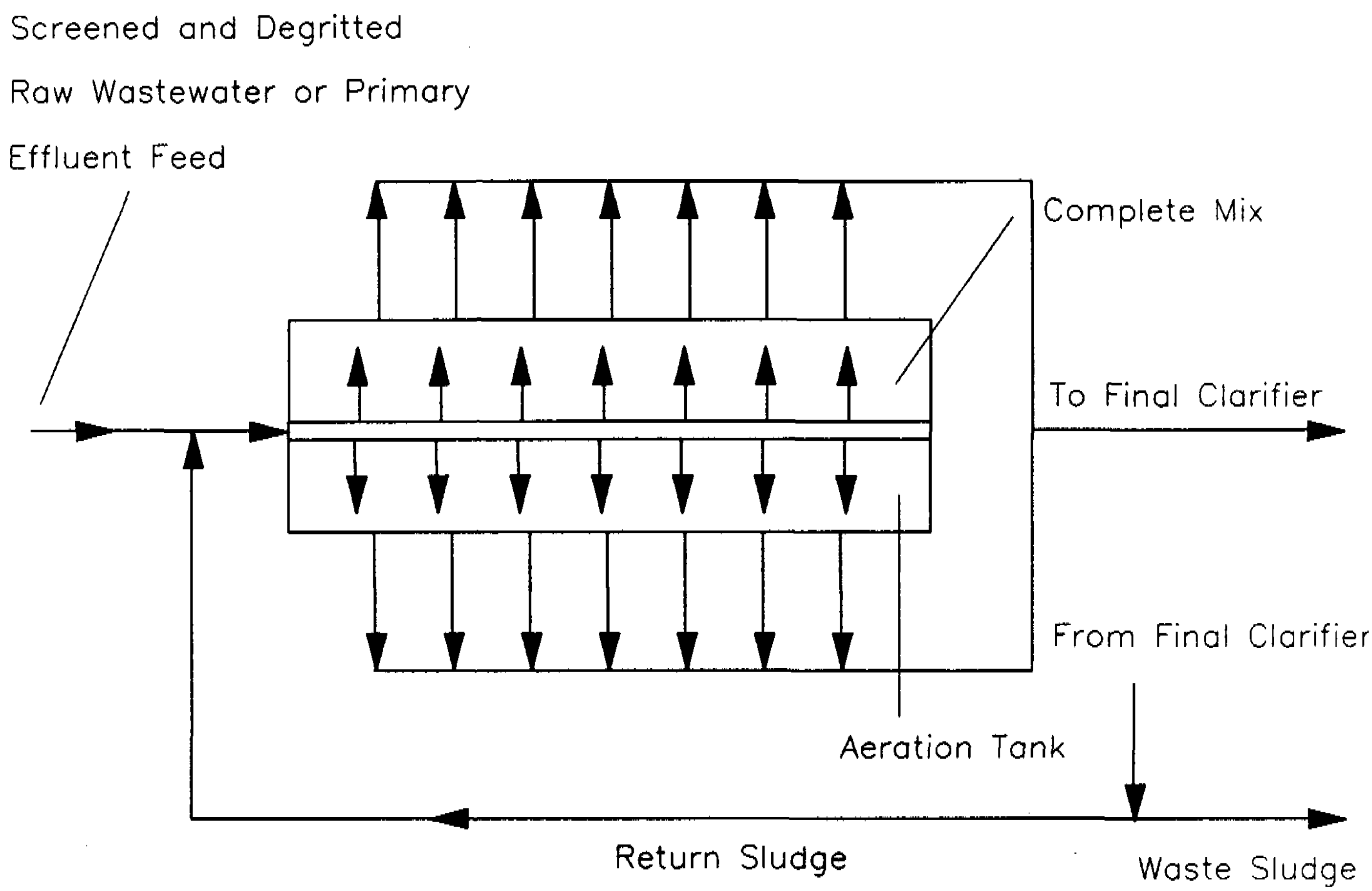


Figure 39.1: Typical Completely Mixed Activated Sludge Treatment.

(Source: Ref. 2)

(File: Martin67)

(based on average daily flow) in a plug flow hydraulic mode. Either surface or submerged aerators may be used. Compressors are used to supply air to submerged systems. Common modifications to the process are step aeration, contact stabilization, and complete mix flow regimes. The oxidation ditch is a variation of the activated sludge process. Alum or ferric chloride may be added for phosphorus removal. Design parameters for a wide range of process modifications are shown in Table 39.1 (4). The typical design parameters for high rate activated sludge treatment are given in Table 39.2 (2).

39.2 Limitations

This process alone does not produce an effluent with BOD5 and suspended solids concentrations suitable for discharge into many surface water bodies. The concentration of BOD5 and suspended solids in final clarified effluent are in the range of 15 to 30 milligrams per liter.

39.3 Costs

The construction cost includes aeration basins, air supply equipment and piping. Clarifiers and recycle pumps are not included. See Figure 39.2 (2, 11). Costs shown are for completely mixed conventional loading rates. High rate systems may be as much as 20% cheaper in construction cost. There is however usually a trade off in that O&M costs are higher for high rate systems. The decision about which option to select should be made based on local conditions and waste treatment requirements.

39.4 Availability

This technology is the most versatile and widely used biological process in municipal wastewater treatment, in the world.

TABLE 39.1
DESIGN PARAMETERS FOR ACTIVATED-SLUDGE PROCESSES

Process modification	Solids Retention Time Days	F/M, kg BOD_5 applied/ kg MLVS.d	Volumetric loading kg BOD_5 applied/m^3.d	MLSS,mg/l	Hydraulic Retention Time	Recir-culation Ratio
Conventional	5-15	0.2-0.4	0.3-0.6	1,500-3,000	4-8	0.25-0.5
Tapered aeration	5-15	0.2-0.4	0.3-0.6	1,500-3,000	4-8	0.25-0.5
Continuous-flow stirred-tank reactor	5-15	0.2-0.6	0.8-2.0	3,000-6,000	3-5	0.25-1.0
Step aeration	5-15	0.2-0.4	0.6-1.0	2,000-3,500	3-5	0.25-0.75
Modified aeration	0.2-0.5	1.5-5.0	1.2-2.4	200-500	1.5-3	0.05-0.15
Contact stabilization	5-15	0.2-0.6	1.0-1.2	(1,000-3,000)[a] (4,000-10,000)[b]	(0.5-1.0)[a] (3-6)[b]	0.25-1.0
Extended aeration	20-30	0.05-0.15	0.1-0.4	3,000-6,000	18-36	0.75-1.50
Kraus process	5-15	0.03-0.8	0.6-1.6	2,000-3,000	4-8	0.5-1.0
High-rate aeration	5-10	0.4-1.5	1.6-1.6	4,000-10,000	0.5-2	1.0-5.0
Pure-oxygen systems	8-20	0.25-1.0	1.6-3.3	6,000-8,000	1-3	0.25-0.5

[a] Contact unit
[b] Solids stabilization unit

Note: $kg/m^3 \cdot d \times 62.4280 = lb/10^3 f^3 \cdot d$
$kg/kg \cdot d \times 1.0 = lb/lb \cdot d$
$mg/L = g/m^3$

Source: Reference 4

TABLE 39.2

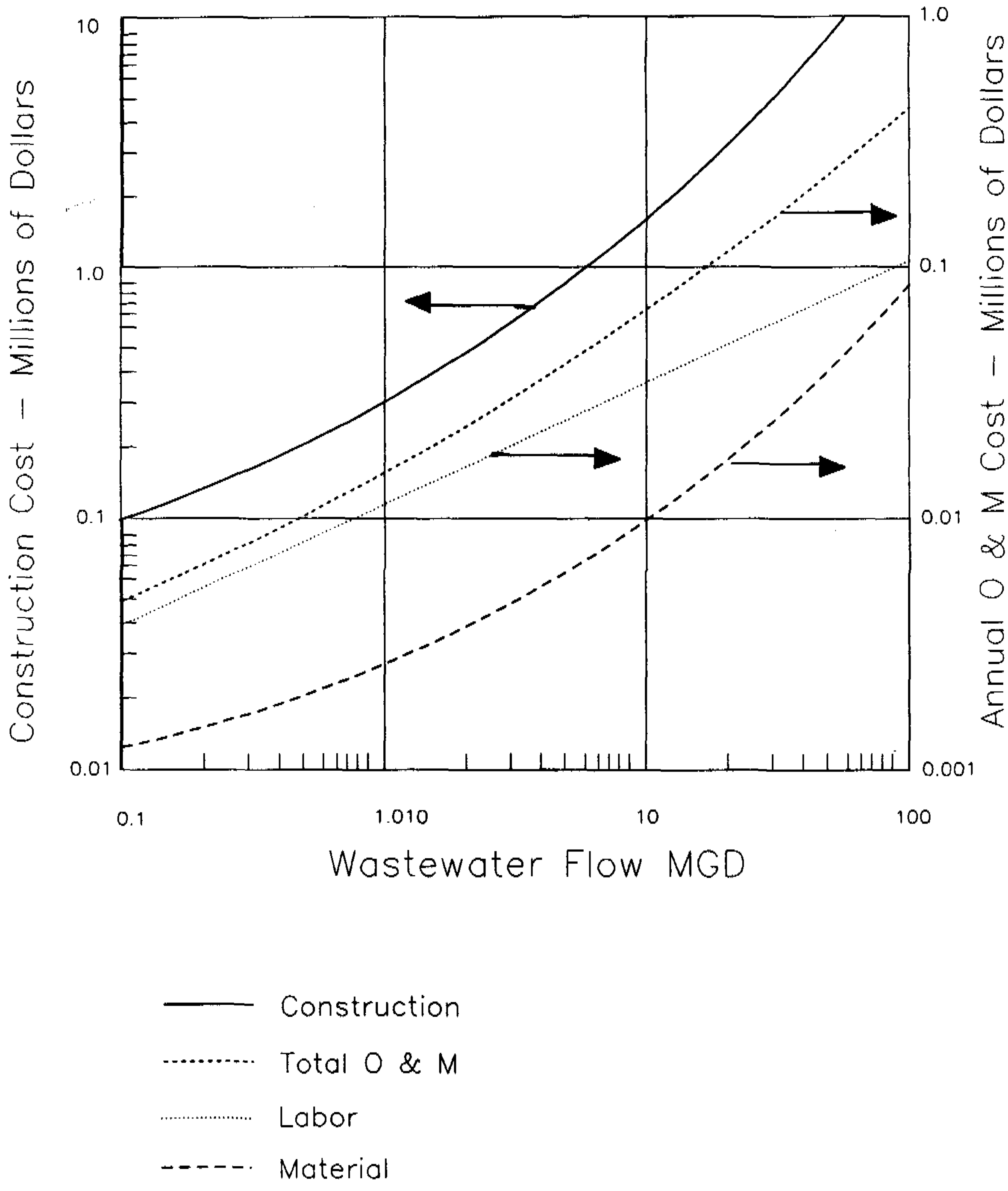
DESIGN PARAMETERS FOR ACTIVATED SLUDGE TREATMENT
(modified aeretion and completely mixed aeration)

Design Criteria - A partial listing of design criteria for the two high rate air activated sludge process options are summarized as follows:

	Modified Aeration	Compeltely Mixed Aeration (tentative)
Volumetric loading, lb BOD_5/d1000 ft^3	50 - 100	50 - 125
MLSS, mg/l	800 - 2000	3000 - 5000
Aeration detention time, hours (based on influent flow)	2 - 3	2 - 4
F/M, lb BOD_5/d/lb MLVSS	0.75 - 2	0.4 - 0.8
Std ft^3 air/lb BOD_5 removed	400 - 800	800 - 1200
Lb O_2/lb BOD_5 removed	0.4 - 0.7	0.9 - 1.2
Sludge retention time, days	0.75 - 2	2 - 5
Recycle ratio (R)	0.25 - 1.0	0.25 - 0.5
Volatile fraction of MLSS	0.7 - 0.85	0.7 - 0.8

Source: Reference 2

Figure 39.2: Construction, Operation & Maintenance Costs for Activated Sludge Treatment (Completely Mixed)



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39.5 Operation and Maintenance

In conventional activated sludge treatment the aeration basins are typically designed to operate in either complete mix or plug flow hydraulic configurations. Either surface or submerged aeration can be employed to transfer oxygen from air to wastewater. Compressors are used to supply air to submerged aeration systems through the network of diffusers. Diffusers used in activated sludge treatment process are porous ceramic plates, porous ceramic domes, ceramic or plastic tubes connected to the pipe header and lateral systems for fine or coarse air bubbles.

To operate an activated sludge treatment process efficiently, it is necessary to understand the importance of the micro-organisms in the system. It is especially important to operate the system on the basis of solids retention time.

39.6 Control

Problems in instability in activated sludge treatment will significantly reduce the process efficiency. Under uniform-flow conditions of wastewater flow and sludge recirculation the biological process would operate at a constant F/M ratio. The best control option is the management of solids retention time (SRT). Sludge wasting should be based on predetermined SRT values determined based on the process design and wastewater characteristics.

39.7 Special Factors

High-rate activated sludge and extended aeration processes are complete mixed systems. Complete mixing in high rate systems permits increased BOD loadings and shortened aeration periods. Thorough mixing in combination with long retention periods used in extended aeration plants provides the assimilative capacity necessary to accept intermittent loading without loss of efficiency. For example, extended aeration plants have been used successfully for schools in the United States where the load enters during a 10 to 12 hour period each day, for only 5 days per week. Activated sludge organisms can be conditioned (acclimatized) to accept reasonably high concentrations of toxic organic compounds and metals. Thus, industrial waste may be treated. These conditions however require very special operational attention by specially trained operators.

39.8 Recommendations

Proper and regular maintenance of activated sludge treatment process is required to achieve high percentage removal of BOD and suspended solids in the influent wastewater. This can be achieved with knowledgeable, trained and experienced plant/treatment operator(s). Therefore, a highly trained plant operator or engineer is required for long-term effectiveness. Many plants in the U.S. and elsewhere do not achieve the design efficiencies because of lack of trained operators, continuously on duty at the plant. The plants require 24 hour attention, and regular sampling and analysis.

40. STEEP SLOPE SEWERS

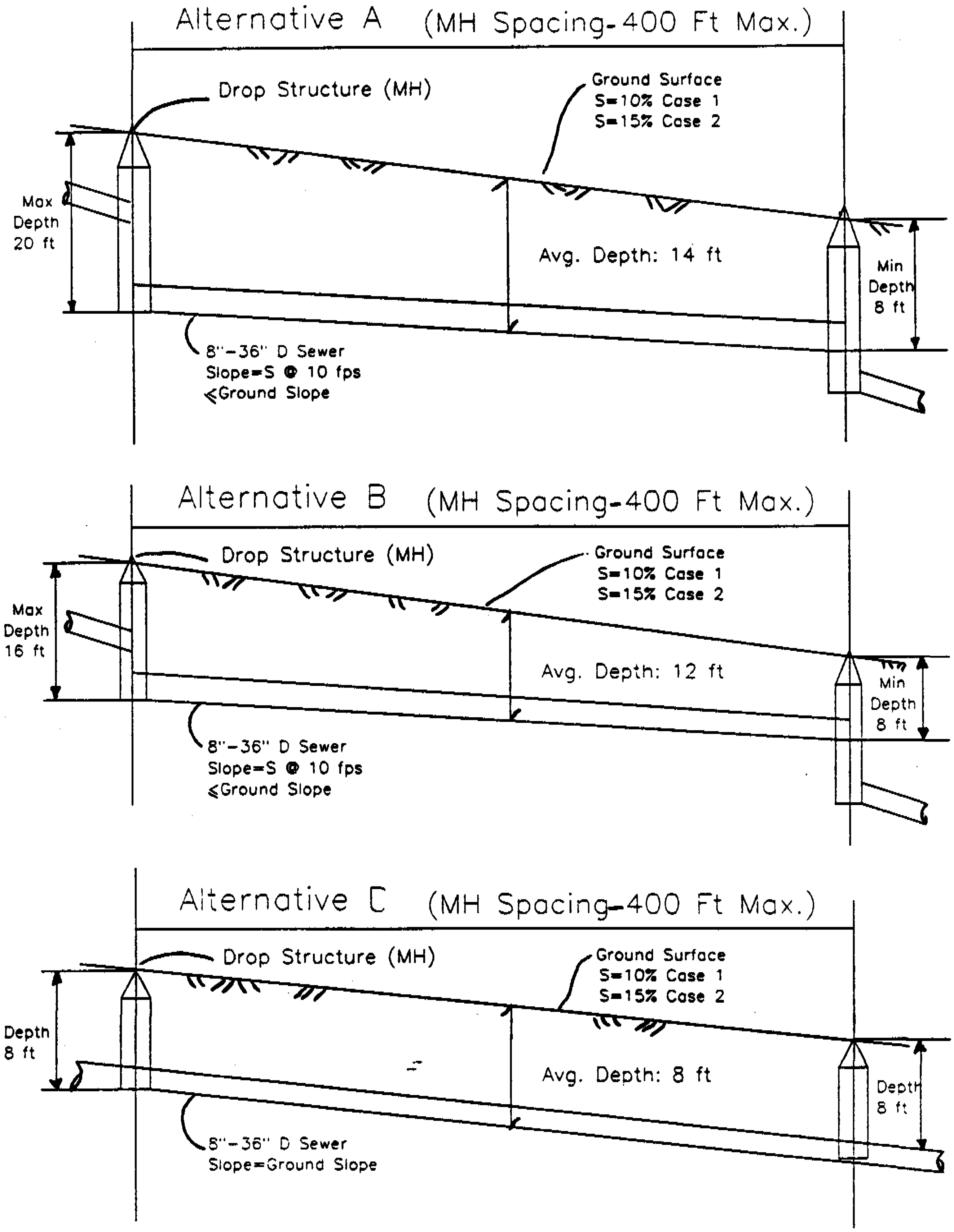
40.1 Description

Traditionally, sewers are constructed on slopes sufficient to allow a minimum 2 ft/sec velocity of water carried wastes. This is to prevent accumulation of settled material in the sewer, which might eventually lead to blockage. Also, traditional design practice requires that maximum velocities (i.e., maximum installed slopes) not be exceeded in order to keep erosion damage to the sewers to a minimum. This practice is costly, and especially costly in geographic areas where steep slopes are common. Steep slopes are common throughout Central America, and along the northern and western sectors of the South American continent. Most Latin American countries would benefit significantly from an altered sewer design practice.

Within the past twenty years the nature of construction materials has changed drastically. Plastic sewer products which have formerly been questionable for "high stress" applications have improved. The erosion characteristics have now been demonstrated to be comparable, and even better than cement-based sewer products (40). In addition, new cementaceous sewer products, reinforced concrete pipe, asbestos cement pipe and others may have improved erosion-resistant properties. Testing should be conducted to determine the nature of new products for steep slope applications.

Steep slope sewers should be used to reduce costs relative to gravity and pressure sewer systems where populations live in rough and hilly terrain. Installation of sewer lines may be done up and down slopes depending on the terrain slope, soil type and bedding type. Steep slope sewers may be constructed in a number of segments moving down the hill. Diagrams of some variations are given on Figure 40.1. Access to a steep slope sewer is made by

Figure 40.1: Alternatives A,B, and C



drop manholes or clean outs located conveniently at intervals or at changes in terrain slope.

40.2 Limitations

There may be increased erosion of sewer materials with the higher velocities likely in steep slope designs.

40.3 Costs

Considerable sewer installation cost savings can be realized from application of the steep slope approach. An example is included herein. Costs have been calculated for three alternative sewer design scenarios (refer to Figure 40.1):

Alternative A: traditional design factors with a maximum depth to sewer of 20 feet.

Alternative B: traditional design factors with a maximum depth of 16 feet.

Alternative C: a design approach where the depth to sewer remains constant (say at 8 feet). This approach is only possible where sewer construction material is erosion resistant, since if depth to sewer is constant, the sewer installation will basically follow the slope of the land surface.

The depth in Alternatives A and B is variable, as seen on Figure 40.1, while the depth is constant at 8 feet for Alternative C.

The calculations for comparative costs have been done for several different conditions as given in summary Table 40.1. Note that for Alternative C, the manhole depths and spacing (and thus the costs) are fixed for various pipe sizes, since it is not

TABLE 40.1

SUMMARY
SEWER COSTS PLUS MANHOLE COSTS

ALTERNATIVE A
AVERAGE SEWER DEPTH 14 FEET
MAXIMUM SEWER DEPTH 20 FEET
MINIMUM SEWER DEPTH 8 FEET

CONDITION 1 - ROCK CASE 1 : GROUND SLOPE = 10%

ITEM	COST PER LF 8	10	12	15	18	24	36
SEWER	60.28	62.07	63.30	95.80	105.79	146.11	173.49
MANHOLE	12.85	17.13	21.42	27.34	30.41	34.73	38.07
TOTAL	73.13	70.20	84.72	123.14	136.20	180.84	211.56

CONDITION 1 - ROCK CASE 2 : GROUND SLOPE = 15%

ITEM	COST PER LF 8	10	12	15	18	24	36
SEWER	60.28	62.07	63.30	95.80	105.79	146.11	173.49
MANHOLE	27.34	38.65	42.83	48.95	51.92	55.87	59.77
TOTAL	87.62	100.72	106.13	144.75	157.71	201.98	233.26

CONDITION 2 - SHORING CASE 1 : GROUND SLOPE = 10%

ITEM	COST PER LF 8	10	12	15	18	24	36
SEWER	35.55	37.46	38.80	47.37	57.59	74.39	102.86
MANHOLE	8.57	11.42	14.28	18.23	20.28	23.16	25.39
TOTAL	44.12	48.88	53.08	65.60	77.87	97.55	128.25

CONDITION 2 - SHORING CASE 2 : GROUND SLOPE = 15%

ITEM	COST PER LF 8	10	12	15	18	24	36
SEWER	35.55	37.46	38.80	47.37	57.59	74.39	102.86
MANHOLE	18.23	25.77	28.56	32.64	34.62	37.25	39.85
TOTAL	53.78	63.23	67.36	80.01	92.21	111.64	142.71

CONDITION 3 - SLOPED SIDES CASE 1 : GROUND SLOPE = 105%

ITEM	COST PER LF 8	10	12	15	18	24	36
SEWER	36.60	38.45	39.74	48.22	58.36	74.99	103.12
MANHOLE	10.38	13.84	17.30	22.09	24.57	28.06	30.76
TOTAL	46.98	52.29	57.04	70.31	82.93	103.05	133.88

TABLE 40.1 (CONT'D)

SUMMARY
SEWER COSTS PLUS MANHOLE COSTS

CONDITION 3 - SLOPED SIDES CASE 2 : GROUND SLOPE = 15%

ITEM	COST PER LF 8	10	12	15	18	24	36
SEWER	36.60	38.45	39.74	48.22	58.36	74.99	103.12
MANHOLE	22.09	31.23	34.61	39.55	41.95	45.14	48.29
TOTAL	58.69	69.68	74.35	87.77	100.31	120.13	151.41

CONDITION 4 - VERTICAL SIDES CASE 1 : GROUND SLOPE = 10%

ITEM	COST PER LF 8	10	12	15	18	24	36
SEWER	11.85	13.70	14.99	23.47	33.60	50.24	78.36
MANHOLE	6.37	8.49	10.62	13.55	15.08	17.22	18.87
TOTAL	18.22	22.19	25.61	37.02	48.68	67.46	97.23

CONDITION 4 - VERTICAL SIDES CASE 1 : GROUND SLOPE = 15%

ITEM	COST PER LF 8	10	12	15	18	24	36
SEWER	11.85	13.70	14.99	23.47	33.60	50.24	78.36
MANHOLE	13.55	19.16	21,23	24,27	25.74	27.70	29.63
TOTAL	25.40	32.86	36.22	47.74	59.34	77.94	107.99

TABLE 40.1 (CONT'D)

SUMMARY
SEWER COSTS & MANHOLE COSTS

ALTERNATIVE B
AVERAGE SEWER DEPTH 12 FEET
MAXIMUM SEWER DEPTH 16 FEET
MINIMUM SEWER DEPTH 8 FEET

CONDITION 1 - ROCK CASE 1 : GROUND SLOPE = 10%

	COST PER LF						
ITEM	8	10	12	15	18	24	36
SEWER	52.26	54.05	55.2	83.77	93.76	130.07	157.45
MANHOLE	10.50	21.00	26.24	33.59	37.16	42.41	46.66
TOTAL	62.76	75.05	81.52	117.36	130.92	172.48	204.11

CONDITION 1 - ROCK CASE 2 : GROUND SLOPE = 15%

	COST PER LF						
ITEM	8	10	12	15	18	24	36
SEWER	52.26	54.05	55.28	83.77	93.76	130.07	157.45
MANHOLE	33.59	47.18	52.49	59.99	63.62	68.84	72.40
TOTAL	85.85	101.23	107.77	143.76	157.38	198.91	229.85

CONDITION 2 - SHORING CASE 1 : GROUND SLOPE = 10%

	COST PER LF						
ITEM	8	10	12	15	18	24	36
SEWER	31.23	33.14	34.49	42.55	52.77	69.07	97.53
MANHOLE	7.07	14.15	17.68	22.63	25.04	28.58	31.43
TOTAL	38.30	47.29	52.17	65.18	77.81	97.65	128.96

CONDITION 2 - SHORING CASE 2 : GROUND SLOPE = 15%

	COST PER LF						
ITEM	8	10	12	15	18	24	36
SEWER	31.23	33.14	34.49	42.55	52.77	69.07	97.53
MANHOLE	22.63	31.79	35.36	40.41	42.86	46.38	48.78
TOTAL	53.86	64.93	69.85	82.96	95.63	115.45	146.31

CONDITION 3 - SHORING SIDES CASE 12 : GROUND SLOPE = 105%

	COST PER LF						
ITEM	8	10	12	15	18	24	36
SEWER	29.02	30.87	32.16	40.14	50.27	66.40	94.53
MANHOLE	7.71	15.42	19.28	24.67	27.29	31.15	34.27
TOTAL	36.73	46.29	51.44	64.81	77.56	97.55	128.80

TABLE 40.1 (CONT'D)

SUMMARY
SEWER COSTS & MANHOLE COSTS

CONDITION 3 - SLOPED SIDES CASE 2 : GROUND SLOPE 15%

ITEM	COST PER LF						
	8	10	12	15	18	24	36
SEWER	29.02	30.87	32.16	40.14	50.27	66.40	94.53
MANHOLE	24.67	34.65	38.65	44.06	46.73	50.56	53.17
TOTAL	53.69	65.52	70.71	84.20	97.00	116.96	147.70

CONDITION 4 - VERTICAL SIDES CASE 1 : GROUND SLOOPE = 10%

ITEM	COST PER LF						
	8	10	12	15	18	24	36
SEWER	10.84	12.69	13.98	21.95	32.09	48.22	76.34
MANHOLE	5.32	10.63	13.29	17.01	18.81	21.47	23.62
TOTAL	16.16	23.32	27.27	38.96	50.90	69.69	99.96

CONDITION 4 - VERTICAL SIDES CASE 1 : GROUND SLOOPE = 10%

ITEM	COST PER LF						
	8	10	12	15	18	24	36
SEWER	10.84	12.69	13.98	21.95	32.09	48.22	76.34
MANHOLE	17.01	23.89	26.58	30.37	32.21	34.85	36.66
TOTAL	27.85	36.58	40.56	52.32	64.30	83.07	113.00

TABLE 40.1 (CONT'D)

SUMMARY
SEWER COSTS & MANHOLE COSTS

ALTERNATIVE C
SEWER DEPTH 8 FEET
NOTE: COSTS ARE IDENTICAL FOR CASE 1 AND 2

CONDITION 1 - ROCK

	COST PER LF						
ITEM	8	10	12	15	18	24	36
SEWER	36.23	38.01	39.24	59.71	69.70	98.00	125.37
MANHOLE	5.80	5.80	5.80	5.80	5.80	5.80	5.80
TOTAL	42.03	43.81	45.04	65.51	75.50	103.80	131.17

CONDITION 2 - SHORING

	COST PER LF						
ITEM	8	10	12	15	18	24	36
SEWER	22.60	24.51	25.86	32.91	43.13	58.42	86.88
MANHOLE	4.08	4.08	4.08	4.08	4.08	4.08	4.08
TOTAL	26.68	28.59	29.94	36.99	47.21	62.50	90.96

CONDTION 3 - SLOPED SIDES

	COST PER LF						
ITEM	8	10	12	15	18	24	36
SEWER	16.90	18.75	20.04	27.00	37.14	52.26	80.38
MANHOLE	3.72	3.72	3.72	3.72	3.72	3.72	3.72
TOTAL	20.62	22.47	23.76	30.76	40.86	55.98	84.10

CONDITION 4 - VERTICAL SIDES

	COST PER LF						
ITEM	8	10	12	15	18	24	36
SEWER	8.81	10.67	11.95	18.92	29.06	44.17	72.30
MANHOLE	3.21	3.21	3.21	3.21	3.21	3.21	3.21
TOTAL	12.02	13.88	15.16	22.13	32.27	47.38	75.51

necessary to control maximum slope. For Alternates A and B the manhole costs increase for increasing pipe size. A and B costs are higher because the manhole spacing must be reduced (decreased length between manholes) in order to maintain proper traditional design slopes (allowing velocities between 2 feet/second, and about 10 feet/second).

The cost per lineal foot (COST PER LF, on Table 40.1 for the three alternatives may be compared. Manholes may not be eliminated completely (cleaning, direction changes, and other requirements). With alternative C, manholes may represent as little as 3% of the system costs. For Alternates A and B, manhole costs do not get below about 20% of system costs. The total costs should also be compared. This can be done in various ways. Note that for say Condition #1 (rock), a cost saving of about 40% is possible using alternative C compared to A. Differences are both larger and smaller when comparing other modes of construction.

Based on steep slope sewer case studies in different terrains, the construction cost is lower than gravity and pressure sewer system. The cost comparison is based on PVC pipe material. The operation and maintenance cost is probably higher than pressure sewer system. The actual cost figures are unavailable during report preparation.

There is another important consideration. When depths are fixed, say at the suggested depth of 8 feet, it may be possible to use vertical side construction generally throughout the project. It is certainly not possible to use vertical side techniques at depths of 14 to 20 feet, unless in solid rock. Using this consideration it is possible to compare DIFFERENT conditions on Table 40.1. For example, if the cost for Alternate C (vertical sides), is compared to Alternate A (with shoring), the cost for the shoring technique (probably required for large depths), is about 4 1/2 times higher.

Cost is related significantly to depth of the installation. It is probably not possible however, to construct systems at depths less than about 6 to 8 feet, because of house connection considerations, including friction losses.

40.4 Availability

The materials of construction and construction approaches are the same as for conventional practice; the design approach is different.

40.5 Operation and Maintenance

Steep slope sewers have been studied by the Institute for Hydromechanic and Hydraulic Structures of the Technical University of Darmstadt of West Germany Uni-Bell PVÇ Pipe Association (40).

Solids deposition is possible where slope changes occur, especially where steep slopes change back to flatter slopes. Cleanouts may be added at these points. More likely, it will be possible to locate manholes at these locations during design. Variations in velocity may cause higher erosion rates than flat slope systems. Early maintenance programs will identify such areas.

40.6 Control

The steep slope sewers design should be based on the following factors: terrain slope, bedding type, soil type, piping material type, and diameter of pipe, operation and maintenance programs available for sewers by the local utilities.

40.7 Special Factors

Definite advantages are low construction cost, energy use and need of minimal operation skills. The technology is best suited to rough and hilly terrains where extensive construction cost is involved.

40.8 Recommendations

Recent studies indicate that for construction of steep slope sewers in rough terrain, PVC pipe is highly regarded because of its light weight and ease of handling and installation. Therefore, applications should be considered South and Central America and the Caribbean.

41. SEQUENCING BATCH REACTORS (SBR)

41.1 Description

A Sequencing Batch Reactor (SBR) is a batch operation very similar to the fill-and-draw activated sludge treatment system. A typical SBR may be composed of one or more tanks. In biological waste treatment, each tank in the system has five basic operating modes or periods based on its primary function: fill, react, settle, draw and idle in a time sequence. Fill (the receiving of raw wastewater) and draw (the discharge of treated effluent) must occur in each complete cycle for a given tank. React (the time to complete desired reaction), settle (the time to separate the micro-organisms from treated effluent), and idle (the time after discharging the tank and before refilling) can be eliminated depending on requirements of treatment. The time for a complete cycle is the total time between beginning of fill to end of idle in a single tank system. In multiple tank system the time between beginning of fill for the first reactor and the end of the idle for the last reactor. In multiple-tank system, the reactors fill in sequence. See Figure 41.1 (41). The operating sequence is shown in Figure 41.2 (42).

The single-tank system is applicable for noncontinuous flow situations especially small rural towns where smaller wastewater flow occurs. Minimal operator input is required to this system. The operation of multiple-tank system can be either simple, with a minimum operator input or complex which depends on flow and load variation and degree of treatment is required.

The typical design parameters of SBR systems are described in sequence as follows (41, 42, 43, 44, 45, 46, 47):

1. Decide if primary treatment is needed. Primary treatment is unnecessary in most SBR systems, especially if the design sludge age or sludge retention

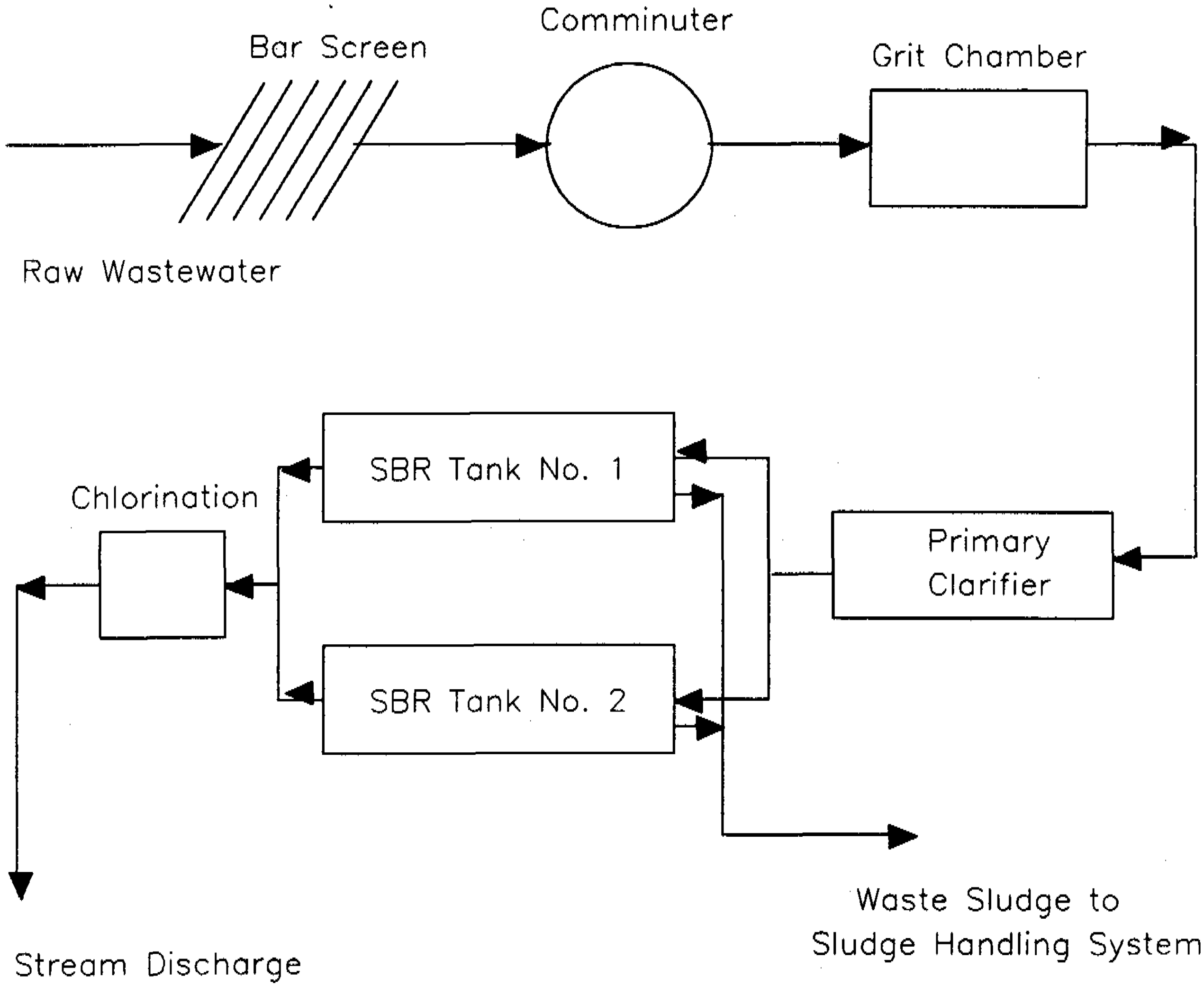
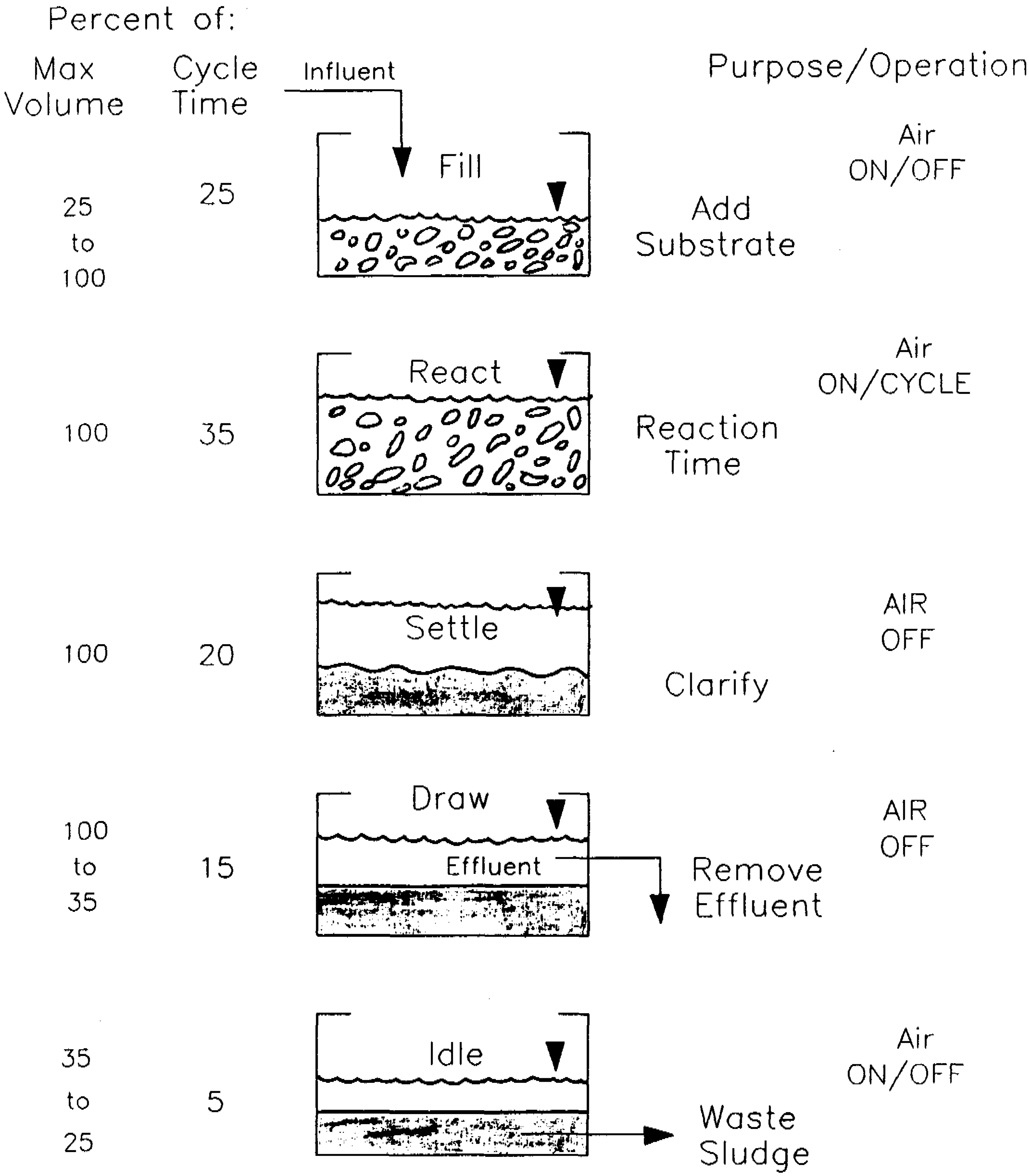


Figure 41.1: Typical Sequence Batch Reactor Wastewater Treatment Process.

(Source: Ref. 41)

Figure 41.2: Typical SBR Operation for One Cycle.



(Source: Ref. 42)

time (SRT) is high (more than 20 days). A high SRT system will also accomplish some sludge digestion aerobically in the reactor. The treatment selected must, of course, comply with applicable Federal and local discharge regulations and codes.

2. Select the desired food/micro-organism (F/M) ratio. The selection of the design F/M ratio should be based on considerations such as nitrification requirements and desired SRT. From a given influent BOD, F can be calculated in pounds of BOD/day, and application of the selected F/M ratio yields the design M or sludge mass.

   F = BOD mg/l x 8.33 lb/gal x flow (10 (6) gal/day)
   M = F divided by F/M ratio

3. Select a value of Mixed Liquor Suspended Solids (MLSS) concentration in the reactor at the end of DRAW. This is slightly different from designing a conventional continuous flow system. The MLSS concentration in an SBR design corresponds to a particular period in the SBR operating cycle, since the concentration changes throughout the cycle. In an SBR, the MLSS concentration is lowest at the end of FILL and highest at the end of DRAW. With most SBR systems, the MLSS concentration at the end of DRAW should be high than the corresponding value used in the design of a conventional continuous flow system, because the MSS concentration in the SBR system at the end of DRAW represents a completely settled mixed liquor, similar to that in a conventional clarifier underflow. The design mixed liquor volume can then be calculated from the selected MLSS concentration.

   Volume = M x (10 (8) gal/day)/
   (8.33 x MLSS concentration)

4. Select the number os SBR tanks. The number selected will depend on the mixed liquor volume determined in step 3, as well as on considerations of area, unit availability, projected maintenance, and operational flexibility. There are no basic rules of judgment in this regard, except that i most cases it is desirable to provide at least two tanks.

5. Select a cycle length, comprised of FILL, REACT, SETTLE, DRAW and IDLE, for each "batch" treatment. The total time for a cycle will be the sum of the times allowed for the cycle phases..

   $T = tf + tr + ts + td + ti$

   The time for FILL, tf, can be calculated from the peak daily flow divided by the number of tanks. The combined time for SETTLE, ts, and the time for DRAW, td, can be estimated to be less than 3 hours. The time for REACT, tr, should be determined from kinetic studies, but for domestic wastewater the range of time for REACT will generally be between 1/2 and 2 hours. The final time factor for IDLE, ti, is selected to provide the operating characteristics needed so that the active part of the cycle will achieve required performance levels (see Section 3, Performance).

6. Calculate the volume of liquid per tank per decant.
   Volume per decant (Vd) = Average Flow/cycles
   Volume per tank per decant = Vd/no. of tanks.

7. Calculate the tank size. The total volume required per tank is the sum of the volume of mixed liquor per tank at the end of DRAW and the volume of liquid decanted per tank per cycle.

The final dimensions of the tanks can be developed by selecting a reasonable tank depth. In most cases a depth of 15 feet or less is practical from the standpoint of oxygen transfer efficiency. Also, allowance must be made for appropriate freeboard, usually 3 to 4 feet.

Volume of tank = Volume mixed liquor + Volume decant
Area of tank = Volume of tank divided by tank depth

8. Size the aeration equipment. This is done in the same manner as in a conventional continuous flow system, except that sine the aeration equipment runs for only a portion of the operating cycle in an SBR system (REACT, or REACT and a part of FILL), the calculated daily oxygen requirement must be met in this shorter time frame. The size of the aeration equipment is therefore increased over that of a conventional continuous flow system of the same capacity.

9. Size the decanter and associated piping. The decant rate is calculated from the maximum volume of liquid decanted per tank per cycle. This volume is then divided by the desired decant or DRAW time. The DRAW period is typically chosen to be approximately 45 minutes.

   Factors to be considered that can place constraints on the design process are the ability to maintain treatment quality in a single tank system, the optimum or maximum sizes for an individual reactor unit in a multitank system, and desired sludge storage volume.

The design steps outlined above illustrate a simplified approach. In a real situation, many iterative calculations may be necessary to accommodate several conditions such as different MLSS concentrations, different number of operating cycles to achieve flexibility during actual plant operation, diurnal flow variations, and different decant heights to correspond to different conditions of sludge settleability, respectively.

41.2 Limitations

The SBR is a sophisticated treatment system, as system gets larger, of the timing units and level sensors used to control the process sequences, and the difficulties and limitation involved in controlling the treatment process system performance such as settle, draw and idle periods.

41.3 Costs

Table 41.1 (39) has estimated costs for constructing SBR systems to handle flow rates of 379, 1893, 3785 and 18,925 m3/d (or 1, 5, 10 and 50 MGD, respectively). Table 41.2 (39) defines the operation and maintenance costs. A two tank system is used for the 379 m3/d plant, and three tank system for the other three daily flow rates. The design criteria for cost purposes are summarized as follows:

Flow (m3/d)	Flow (MGD)	Sets of Tanks	Tanks for Set	Total Volume (M3)
379	1	2	1	252
1,893	5	3	1	947
3,785	10	3	1	1,893
18,925	50	3	4	9,465

TABLE 41.1

COST ESTIMATES FOR SBR FOR FOUR AVERAGE DAILY FLOW RATES*

Process Unit	Flow Rates (m^3/d; MGD in parentheses) 379 (1)	1893 (2)	3785 (10)	18,925 (50)
Inlet Control System	$ 2,300	$ 3,400	$ 4,500	$ 23,000
Contact Chamber Baffle Walls	2,300	4,500	5,600	27,000
Aerators	28,000	56,000	68,000	290,000
Excavation, Concrete and/Handrail	79,000	170,000	280,000	960,000
Microprocessors	11,000	11,000	11,000	11,000
Level Control/Monitoring	2,300	4,500	4,500	18,000
Dacant System	10,000	18,000	20,000	102,000
Subtotal (1)	$ 136,000	$ 268,000	$ 400,000	$ 1,400,000
Noncomponent Costs*	34,000	67,000	100,000	354,000
Subtotal(2)	$ 170,000	$ 335,000	$ 497,000	$ 1,800,000
Engineering, Construction on Supervision and Contingencies**	51,000	100,000	150,000	530,000
Total Installed Capital Costs	$ 220,000	$ 435,000	$ 645,000	$ 2,300,000
Annual Operation and Maintenance	15,000	27,000	45,000	167,000
Present Worth Costs***	$ 572,000	$ 714,000	$ 1,110,000	$ 167,000
Costs/(m^3/d)	$ 983	$ 373	$ 294	$ 215

* At 25 percent of subtotal (1), includes piping, electrical, instrumentation and site preparation.

** At 30 percent of subtota (2).

*** Present worth computed at 7 3/8 percent interest rate and 20 year life (PWF = 10.29213). Add present worth O & M costs to Total Installed Capital Costs.

Source: Reference 39.

TABLE 41.2

OPERATION AND MAINTENANCE COST ESTIMATES FOR THE SBR
FOR FOUR AVERAGE DAILY FLOW RATES**

Cost (dollar/yr)	Flow Rates (m^3/d; MGD in parentheses) 379 (1)	1983 (5)	3785 (10)	18,925 (50)
Operation Labor	$ 9,000	11,500	$18,000	$38,000
Maintenance Labor	15,000	2,200	2,700	58,000
Power*	2,500	11,000	22,000	111,000
Material	2,200	3,000	4,300	15,000
TOTAL O & M (rounded)	$ 15,000	$ 28,000	$ 47,000	$170,000

* Includes mixing, aeration and decanting at a power rate of $0.06/kWh.

** Costs based on Jarnuary 1983.

Source: Reference 39

Capital and operation and maintenance costs are lower than the costs for conventional activated sludge. These comparisons are summarized in Figure 41.3

41.4 Availability

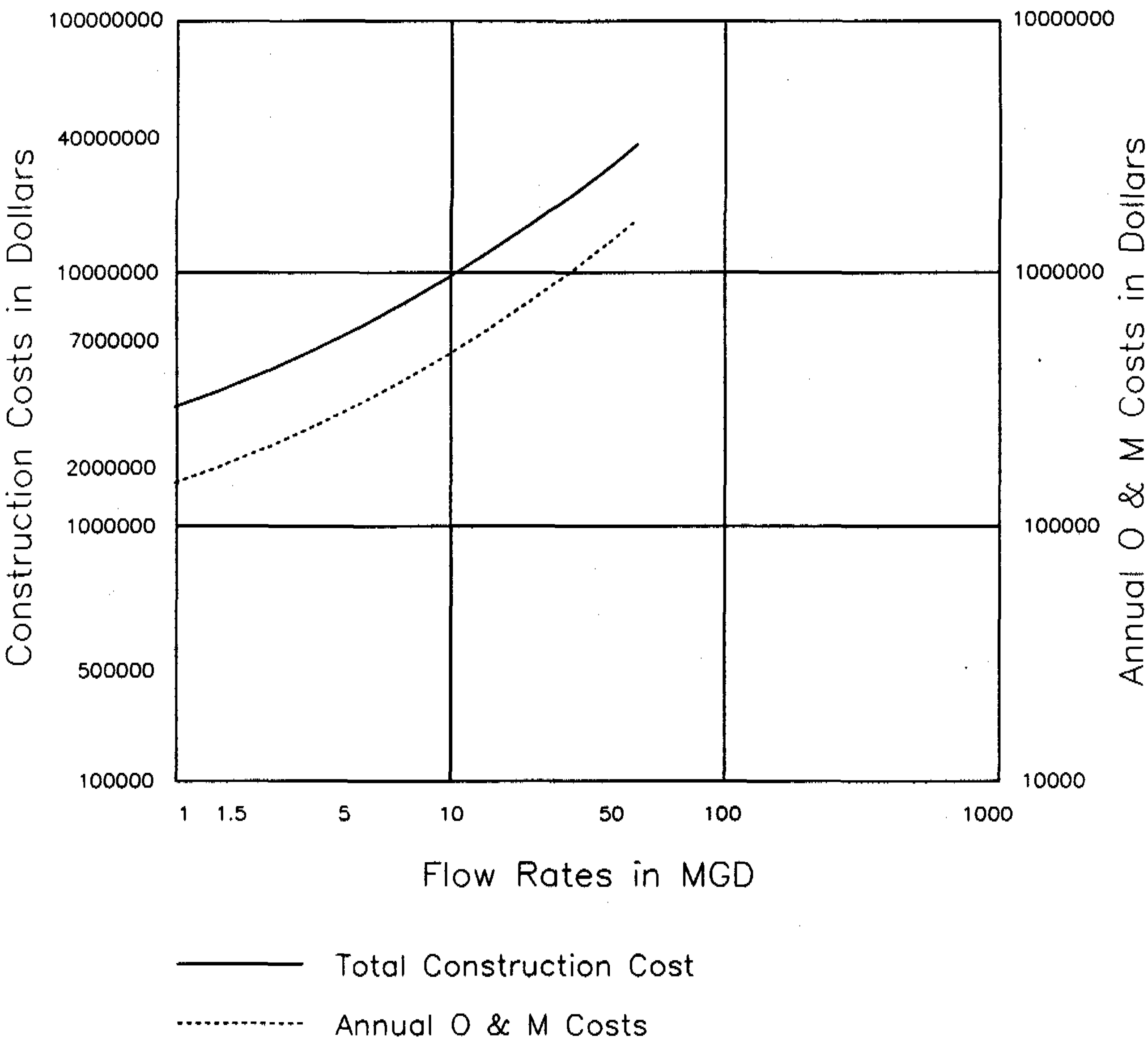
The SBR wastewater treatment process is at least a century old technology. It has to be designed and constructed by experienced engineers based on requirements. SBR wastewater treatment plants are currently operating at several sites in Australia, Canada, and the United States. The evaluation summary of these plants are given in Table 41.3. The designs of these plants differ in several aspects including inlet design, aeration/mixing system design but they all operate on sequencing batch principles.

41.5 Operation and Maintenance

Sequencing Batch Reaction is the treatment of wastewater on a batch basin and is no more than an activated sludge system which operates in time rather than in space, i.e., all steps of the process take place one after the other, in the same tank instead of moving to a second tank for the continuation of the treatment. Typical SBR operation (Figure 41.2) involves filling a tank with raw wastewater or primary effluent, aerating the wastewater to convert the organics into microbial mass, proving a period for settling, discharging the treated effluent, and a period identified as IDLE that represents the time after discharging the tank and before refilling. This configuration allows incoming flow to be switched to one tank while the other is going through the aeration, clarification , and discharge functions.

A key element in the SBR process is that a tank is never completely emptied, and a portion of settled solids is left in the tank for the next cycle. The remaining portion of this

Figure 41.3: Construction and Annual Operation & Maintenance Costs for the SBR.



(File: Martin14)

lants Evaluation Summary

Parameter	Canada: Rivercrest, Manitoba (a)	Canada: Glenlea, Manitoba (a)	United States: Choctaw, Oklahoma	United States: Grundy Center, Iowa	United States: Eldora, Iowa	United States: Culver, Indiana (b)	Australia: Tamworth, New South Wales	Australia: Yamba, New South Wales	Remarks
Design Firm	Topnik & Assoc. Ltd.	Topnik & Assoc. Ltd.	Res Engg. & Assoc.	Clapsaddle Garber Assoc.	Jensen, Carry & Shott, Inc.	University of Notre Dame	Laurie, Montgomeria & Petit Pty. Ltd.	Austgen-Biojet	a. Rivercrest and Glenlea data obtained from reference (20)
Date of first visit	5/16/84	5/16/84	5/30/84	6/11/84	6/12/84	6/14/84	7/10/84	7/11/84	b. Culver data obtained from reference (15).
Design avg. flow, gpd	24,000	2,000	500,000	832,000	220,000	—	535,000	253,000	c. Actual operating data.
Design loading BOD, mg/l SS, mg/l NH_3, mg/l	236(c) 200(c) 37(c)	251(c) 152(c) 55(c)	260,366(c) 260,350(c) 19(c)	200 — 15	250,120(c) — 25	170(c,d) 150(c,d) 20(c,d)	260 — 35-40	260(d) — —	d. Raw Sewage e. Jet motive pumps on all the time, but air on and off for 40 and 10 mins, respectively repeated three times during the 150 minutes fill & react periods.
Current avg. flow, gpd	60,000	1,165	200,000 283,000 (equivalent)	800,000	220,000 106,000 (equivalent)	353,000	535,000(est)	—	
Desired eff. qual. BOD, mg/l SS, mg/l NH_3, mg/l	TOC-40 30 —	30 30 —	20 20 15	30 30 8 (summer), 11 (winter)	30 30 8(summer), 10(winter)	10 10	30 30	30 30	
Actual eff. qual BOD, mg/l SS, mg/l NH_3, mg/l	11 15 10	5 6 2	8 18 -	Not being met because of decanter problems. See discussion.	Data was not available. Effluent appeared to be satisfactory.	10 5 1.0	5 to 10 5 to 10 2.2	8 to 10 10 to 16 1.0	
Mode of operation at design flow Fill time R time S time D time I time	 90 min 45 min 20–60 min —	 22 hrs 1 hr 1 hr —	 18 hrs 3 hrs 3 hrs —	40 min (w/o air/pumps) 120 min (w/air/pumps) 60 min 40 min 60 min	 150 min(e) 80 min 50 min 45 min	180 min 42 min 42 min 42 min 60 min Fill 30% mixed 70% aerated	continuous 120–150 min 45 min 45 min —	continuous 150 min 180 min 45 min —	
Important design parameters DT, hours F/M, kg BOD/kg ML SS SRT, days	7.6 0.18(c) 43(c)	49 0.032(c) 18.80(c)	48 0.037, 0.028(c) Sludge wasted twice in 10 months	20.4 0.078 0.067(c) 25.30(c)	43 0.05 Sludge not wasted in last 2 months	16.5(c) 0.08:0.16 15.45(c)	46 0.04 —	36 0.05 —	
Power usage kwh/kg BOD applied	0.8	22.9	2.9	0.8 to 1.3	2.2	2.1	1.9	1.5	
Unit Processes Trash Rack Mech. Screens Comminutor Grit Removal Equalization Primary Treat. SBR Disinfection Sludge Treat.	Yes — — — Yes — Yes — Holding tank & land application	— — — — Lift station wet well — Yes — Agriculture farm	Yes (bypass) — Yes — Emergency holding pond — Yes Yes Holding pond & land appl.	Yes (bypass) Yes Yes, aerated Sideline equilization — Yes Yes Aerated sludge holding & sludge beds	— Yes — Yes, aerated — — Yes — Anaerobic digestors & sludge beds	— Yes or Yes — — Yes Yes Yes Aerobic digesters & sludge beds	— Yes or Yes — — — Yes Polishing lagoon Sludge lagoon	Yes — — Yes — Yes Yes Polishing lagoon Aerobic lagoon	
Reasons for providing this technology	Capital cost savings & simple operation	Capital cost savings & simple operation	8.4 percent savings in life cycle costs	19% capital cost savings in secondary treatment process or 8 percent savings in overall plant cost	Capital cost savings & simple operation (100% city funding)	Full scale study funded by EPA	Capital cost savings	Capital cost savings	

Source: Reference 39

residue (sludge) is transferred to the sludge handling facility. The fraction of wasted sludge will depend upon the desired sludge age.

The retention of sludge within the tank establishes a population of micro-organisms uniquely suited to treating the waste. During the process, the micro-organisms are subject to periods of high and low oxygen and high and food availability. This condition develops a population or organisms which is very efficient at treating the particular wastewater.

The maintenance of SBR treatment process system is very easy because of new technology and equipment. Improvements in aeration devices, electronic control systems include automatic switches and valves and mechanical times has been allowed smooth maintenance of plant.

41.6 Control

The SBR treatment process is designed for BOD, suspended solids, nitrogen, and phosphorous removal. As mentioned earlier in a single-tank system, the desired treatment levels can be achieved through the simple operation technologies. The complex SBR system will involve a multiple-tank system with adjustable time, level, dissolved oxygen, and turbidity control, regulating valves, and compressors, mixtures, and pumps controls the removal efficiency of the treatment.

41.7 Special Factors

Factors to be considered that can place constraints on the design process are the ability to maintain treatment quality in a single tank system, the optimum or maximum sizes for an individual reactor unit in a multiple tank system and desired sludge storage volume.

41.8 Recommendations

The capital cost consideration is playing an increasingly important role. The SBR wastewater treatment technology is: worthy of consideration because of high treatment performance, likely to provide more trouble free operation, may be more economical from an operation and maintenance point of view. Therefore, consideration and implementation of this process is recommended in South and Central America and the Caribbean.

42. DRAFT TUBE SUBMERGED TURBINE AERATION (DTSTA)

42.1 Description

Draft tube submerged turbine aeration (DTSTA) consists of a down pumping, airfoil-type axial flow impeller (Figure 42.1); and a compressed air supply (blowers, valves, and piping). Coarse air bubbles are sheared into smaller bubbles by energy induced by high pumping rate, which the manufacturers claim provides superior wire-to-water transfer efficiencies due to high oxygen dissolution from the air phase. DTSTA are employed in conventional complete-mix configurations and in the "barrier oxidation ditch" process.

In conventional complete-mix applications DTSTA are normally supported from a deck inside the tank. Deep tanks (25 to 30 foot (7.6 to 9.1 m) water depth) are used and Nameplate-horsepower (NPHP) ranges from 0.15 to 0.20 horsepower (111 to 150 W) per 1000 gallon tank volume. Pumping rates are such that the entire tank volume is turned over about 12 to 15 times per hour.

The two principle features of the barrier oxidation ditch attributable to DTSTA are the use of an U-Tube conduit and a barrier wall (Figure 42.2). Pumping rates are set to cycle basin contents 6 to 8 times per hour.

The DTSTA advantages cited by manufacturers' of the equipment include:

- Highly efficient 02 transfer
  - Standard Oxygen Transfer Efficiency = 50-60%
  - Actual Oxygen Transfer Efficiency = 37=43%
- Highly efficient mass transfer per wire horsepower
- Independent control of aeration and mixing

Figure 42.1: Draft Tube Submerged Turbine Aerator

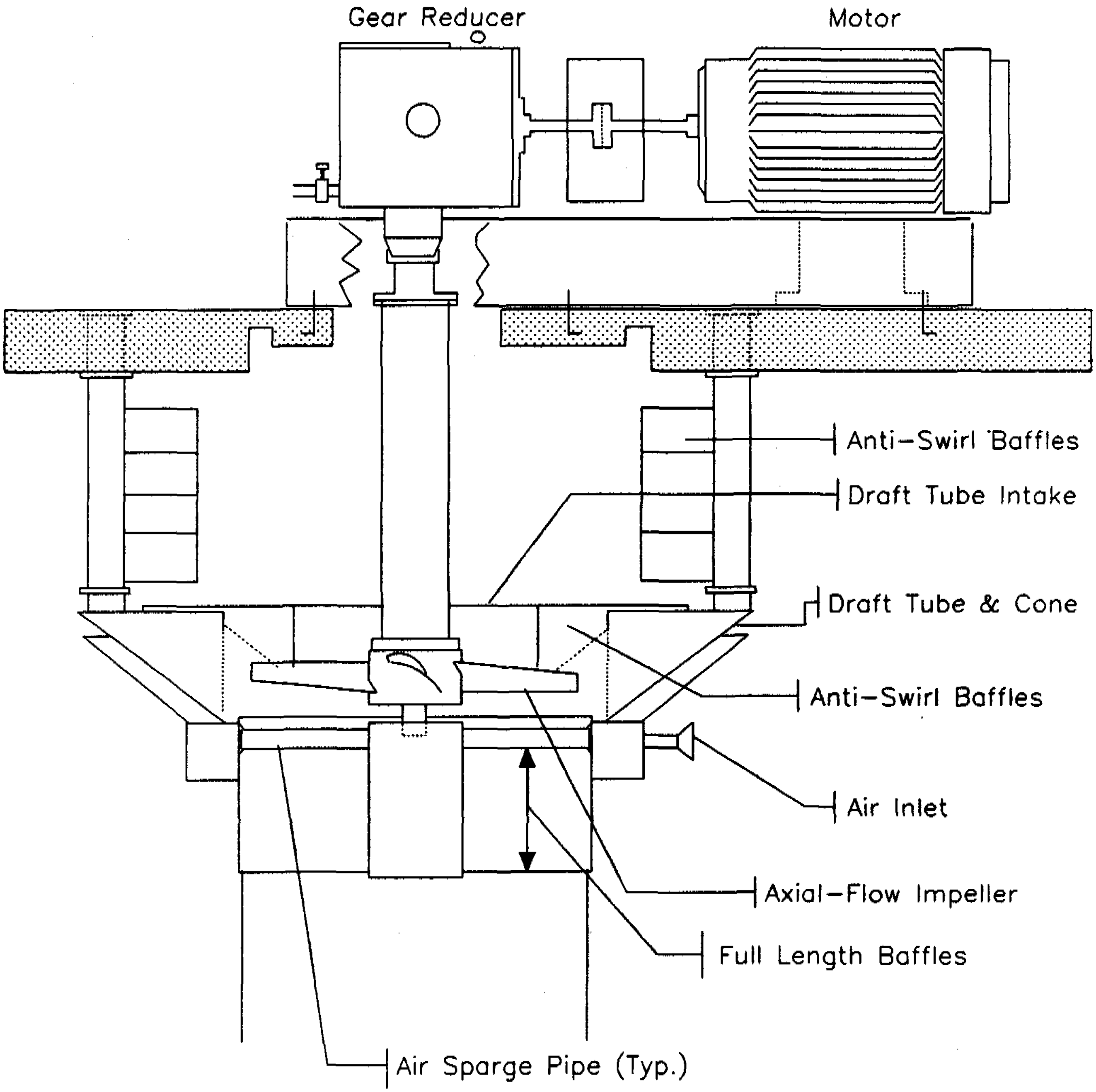


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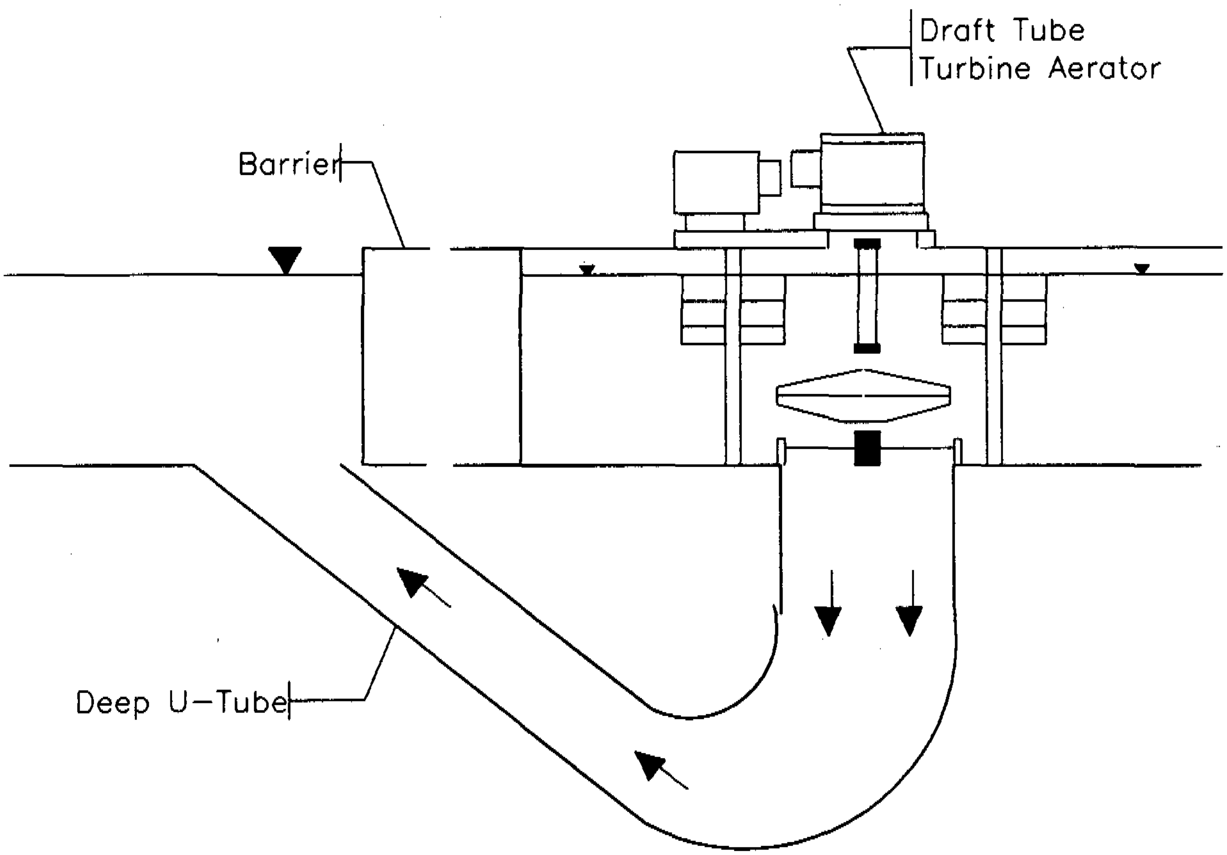
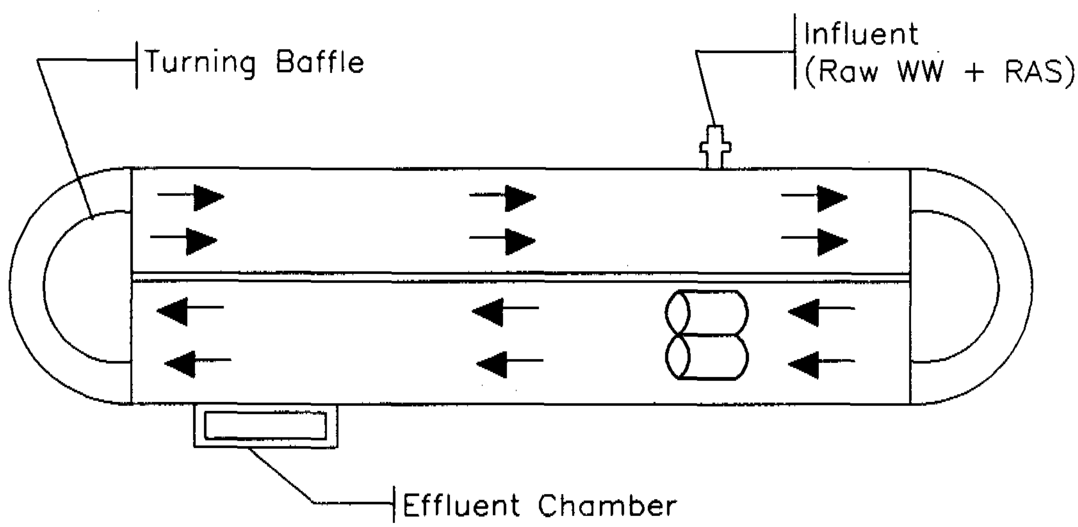
Figure 42.2: Barrier Oxidation Ditch





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- Minimal basin heat loss
- Stable operation over a wide range of liquid levels.

The USEPA conducted a technological assessment of facilities using DTSTA for oxygenation of activated sludge to evaluate process performance, mechanical reliability, system operation and design features. Six facilities were evaluated including those located at Cranston RI, Fairfield IA, Santa Fe NM, Atmore AL, Foley AL and Presque Isle ME. The Cranston RI plant employed DTSTA in the complete-mix mode, all others were barrier oxidation ditch applications.

42.2 Findings

Average SAE (Standard Aeration Efficiency) values for DTSTA equipment tested in the factory (3.81 lb O_2/hr/whp) were over double the average SAE's for field tested equipment (1.87 lb O_2/hr/whp). It appears that the energy loss of the U-Tube configuration is a very important factor in the design of DTSTA systems.

All facilities visited exhibited a decrease in performance efficiency over time under process conditions. Average reductions for the six plants evaluated are as follows:

Turbine power increase - 35-40%
Pumping rate decrease - 12-18%
Air handling capacity decrease - 23-33%
SAE decrease - 38-45%

Also, the carbon-steel impellers at the Cranston RI facility showed significant blade wear attributable to cavitation and erosion/corrosion. The DTSTA units at this facility are comparatively high speed/small diameter units which could have accounted for increased impeller wear.

The decrease in DTSTA performance over time is attributable to the accumulation of minute debris in the leading edge of the impeller which adversely changes its hydrodynamic characteristics. A new non-fouling impeller design has been introduced by manufacturers which eliminates the time related deterioration of DTSTA units. The new impeller has a swept back blade and is made of 304 stainless steel. Also certain of the new designs use a teflon-life material on the leading edge to reduce friction and thereby optimize debris shedding.

Manufacturerers of DTSTA equipment continue to address performance problems with emphasis being placed on side-by-side clean and dirty (wastewater) testing. Recent process 02 transfer tests reveal that the alpha factor for DTSTA equipment may be in the range of from 0.40 to 0.50 which is considerably less than values (0.70 to 0.90) cited in the specifications for the six facilities evaluated, and those normally found in literature.

42.3 Benefits

DTSTA is well suited to aeration of deep tanks and deep basins save land area. The air sparger is placed at mid-depth which allows deep tank aeration at conventional blower pressures (10-15 feet water). DTSTA eliminates ice and mist formation and conserves basin heat energy. When employed in the barrier oxidation ditch configuration DTSTA offer the below listed benefits:

- Point source aeration of all mixed liquor
- Adjustable channel velocities (0.5-2.0 fps, 0.15 to 0.9 m/sec) independent of 02 transfer rate
- Water level adjustment (1-6 ft, 0.3 to 1.8 m) for flow equalization

42.4 Applications

DTSTA equipment is used in conventional complete mix activated sludge configurations and applied to the barrier oxidation ditch process. The equipment is applicability to aerobic digestion, pure oxygen covered systems and for post aeration of treated wastewater.

43. INTERMITTENT SAND FILTERS

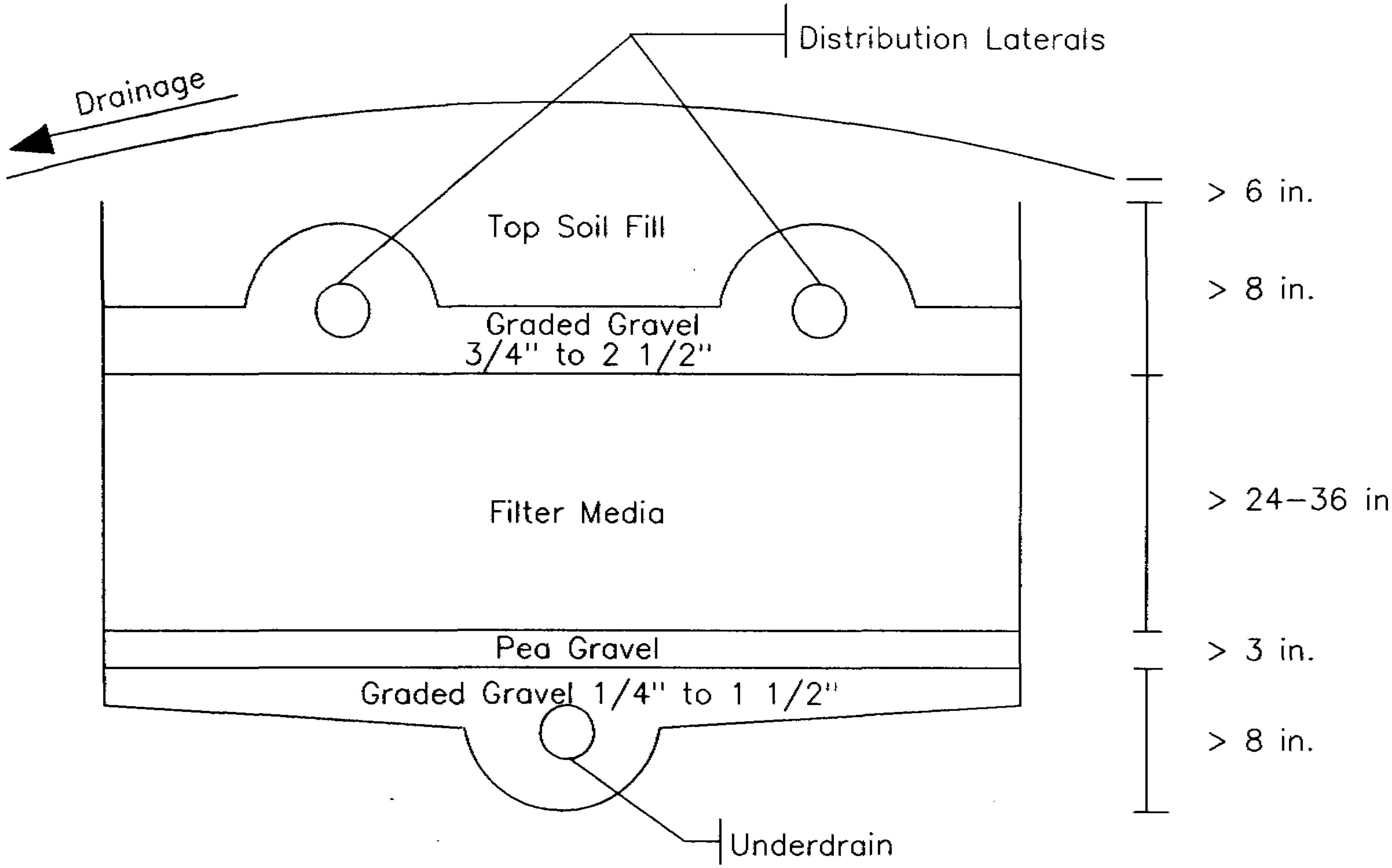
43.1 Description

Intermittent sand filters are beds of medium to coarse sands, from 2 to 4 (0.6 to 1.2 m) deep and underlain with gravel containing underdrains. Effluent is intermittently applied to the surface and purification of the effluent occurs as it infiltrates and percolates through the sand bed. Underdrains collect the filtrate and convey it to additional treatment processes and/or discharge. The full-scale use of intermittent sand filters as a secondary wastewater treatment process is not a new technology. They were frequently used by sewered communities in the USA around the turn of the century. Intermittent sand filter design concepts include buried filters, open single-pass filters and open recirculating sand filters (19,66).

Buried sand filters are constructed below grade and covered with backfill material. Buried filter designs are most commonly used for very small flows such as those from single homes and small commercial establishments. These filters are designed to perform for very long periods of time (up to 20 years) without sand filters (Figure 43.1) are similar to buried sand filters except that the surface of the filter is left exposed and higher hydraulic and organic loadings are generally applied. These filters are used for individual homes as well as larger flows from small communities or industries (up to 0.2 MGD) 0.01 m^3/sec). Recirculating sand filters are open filters which utilize somewhat coarser media and employ filtrate recirculation. A portion of the filtrate is diverted for further treatment or disposal during each dose. A recirculation rate of 3:1 to 5:1 is typical.

Intermittent sand filters can produce very high quality effluents. Concentrations of effluent BOD_5 and TSS are typically less than 10 mg/L with ammonia nitrogen less than 5 mg-N/L. Only

Figure 43.1: Typical Buried Sand Filter



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limited removal of phosphorous and fecal coliform bacteria are achieved, however. Design considerations important to achieving this level of treatment include pretreatment, sand characteristics, hydraulic and organic loading rates, temperature and filter dosing techniques.

43.2 Sand Characteristics

Sizes: Sand with an effective size of 0.4 to 1.5 mm and a uniformity coefficient not greater than 4.0 is satisfactory.

Depth: Media depths used in intermittent sand filters were initially 4 to 10 feet. However, studies revealed that most of the purification of wastewater occurred within the top 9 to 12 inches (23 to 30 cm.) of the bed. It is critical to maintain sufficient depth of sand so that the zone of capillarity does not infringe on the zone required for treatment. For these reasons most media depths used today range from 24 to 42 inches (62-107 cm.). The use of shallower filter beds helps to keep the cost of installation low. Deeper beds tend to produce a more constant effluent quality.

43.3 Pretreatment

The operation and performance of ISF's are directly related to the degree of pretreatment of the applied wastewater. There appears a direct relationship between degree of pretreatment and the acceptance rates of wastewater hydraulic longevity and effluent quality.

Rates

The allowable loading of intermittent sand filters varies with the nature of the sand, the strength of the sewage, and the method of pretreatment. Average values are presented below.

Maximum allowable loading of intermittent sand filters

Applied Sewage	Gpd/Acre	Persons/ *	Pounds of BOD Daily
Untreated	20,000- 80,000	250-1,000	Up to 75
Settled	50,000-125,000	500-1,500	Up to 165
Biologically Treated	Up to 50,000+	Up to 5,000	---

*Sewage flow = 80 to 100 gpcd.
+Up to 125,000 gpd and 1,500 persons per acre for small plants with unskilled supervision.
4 ha/acre.

Temperature

Temperature directly affects the rate of microbial growth, chemical reactions, absorption mechanisms, and other factors that contribute to the stabilization of wastewater within an intermittent sand filter. Somewhat better operation and performance therefore can be expected from filters in warmer locales.

Dosage

A sufficient amount of sewage is run onto the bed to cover it to a depth of 2 to 4 in. (5 to 20 cnm). The higher amount loads large beds more uniformly. Dosage is regulated so that flooding is completed in 10 to 20 minutes. The necessary rate of discharge is about 0.2 cfs/1,000 sq. ft. under the average available head. Each dose carries from 50,000 to 100,000 gal. of sewage onto an acre of bed. Two or more doses are applied daily, depending upon the rate at which the sewage is absorbed. Sufficient time must be allowed for recovery of the bed by reaeration. Dosing is controlled by hand operation of gates or by automatic dosing tanks. Dosing tanks store the full dose of a bed. Siphons, or other regulating devices are sized large enough to discharge, under minimum head, at about twice the maximum expected rate of inflow. Intermittent operation is thereby assured.

Construction of intermittent sand filters involved (1) clearing and grubbing the area and stripping the top soil; (2) leveling the area and subdividing it into beds of suitable size by throwing up the top soil and other waste soil into partition embankments and access embankments; (3) undertraining the beds; (4) laying distribution mains and constructing distribution manholes and inlets in the access embankments; and (5) building sewage carriers or troughs on the beds. Shape and arrangement of beds depend upon local topographic conditions. Their size and number should be kept in line with the storage needed for the intermittent operation of the beds. Also, it must be possible to take at least one bed out of service for cleaning or repairs without overloading the remaining area. Beds more than an acre in size are uncommon.

Sewage is unusually conveyed to the beds in vitrified-tile or concrete pipe, sometimes in cast-iron force main. The conduits are laid in the access embankments which are made wide enough (8 to 10 ft.) for necessary construction and maintenance equipment. Ramps lead onto each bed. The height of the access embankment is determined by the necessary hydraulic gradient of the influent sewer and depth of cover. A depth of 2 ft. (0.6 m) will protect pipe against breakage and against freezing in all but very cold climates. The partition embankments are often low (12 to 18 in.) (30 to 59 cnm) but wide enough for a foot path (2 ft.) (0.6m). Embankment dimensions are normally governed by the amount of spoil to be disposed of.

Sewage is discharged onto individual beds through branch pipes, generally connected to the main distributor at manholes and controlled by shear or sluice gates inside the manhole. One or more outlets are so placed that lateral travel of the applied sewage is not more than 20 ft. (6.1m). To prevent erosion of the sand surface, the sewage discharges onto a concrete or stone apron covering the nearby surface of the bed or into a trough or carrier made of wood or concrete running almost across the bed and equipped with side outlets, deflectors, and splash slabs.

Perforated Underdrains: Pipes of vitrified tile or concrete or pipe laid with open joints (about 3/8 in. apart) are placed at least 2 ft. (0.6 m) below the sand surface. Their minimum diameter is 4 in. (10 cm), and their spacing preferably not more than 10 ft. The greater the depth and the coarser the sand, the wider can be the spacing. In order to insure full aeration of the bed after dosing, underdrains are sloped to the outlet in order to discharge the sewage at the maximum rate of percolation. The figures shown below give the general magnitudes of percolation.

Maximum rates of percolation of water through intermittent sand filters (MGD x 0.044 = m^3/sec)

Effective size, mm	0.2	0.3	0.4	0.5	0.6
Maximum rate of percolation, mgd/acre					
at 50 F	20-50	50-100	100-200	150-300	200-400
at 70 F	30-60	70-150	120-150	200-400	300-600

As a rule, the lower values are encountered.

Under drains are carefully laid to line and grade in trenches opened in the sand bed. The pipe is surrounded by successive layers of coarse stone or gravel an coarse sand.

43.4 Advantages and Disadvantages

Advantages for developing countries are: reasonably low construction cost using manual labor, simplicity of design and operation, unskilled maintenance labor, no chemicals required and sand can usually be found locally, low power requirement, and a very high quality of effluent is produced. Disadvantages are: pretreatment (at least sedimentation) is probably required for most applications, odor problems from open filters, units require substantial land area, colder temperatures can dramatically affect system cost and performance, system probably not feasible in absence of natural sand deposits., i.e., construction of sand filters in the absence of natural, sandy areas is unusual.

43.5 Costs

Costs (both Capital and O&M) of intermittent sand filters exclusive of land cost, are presented in Figure 43.2 (19, 66).

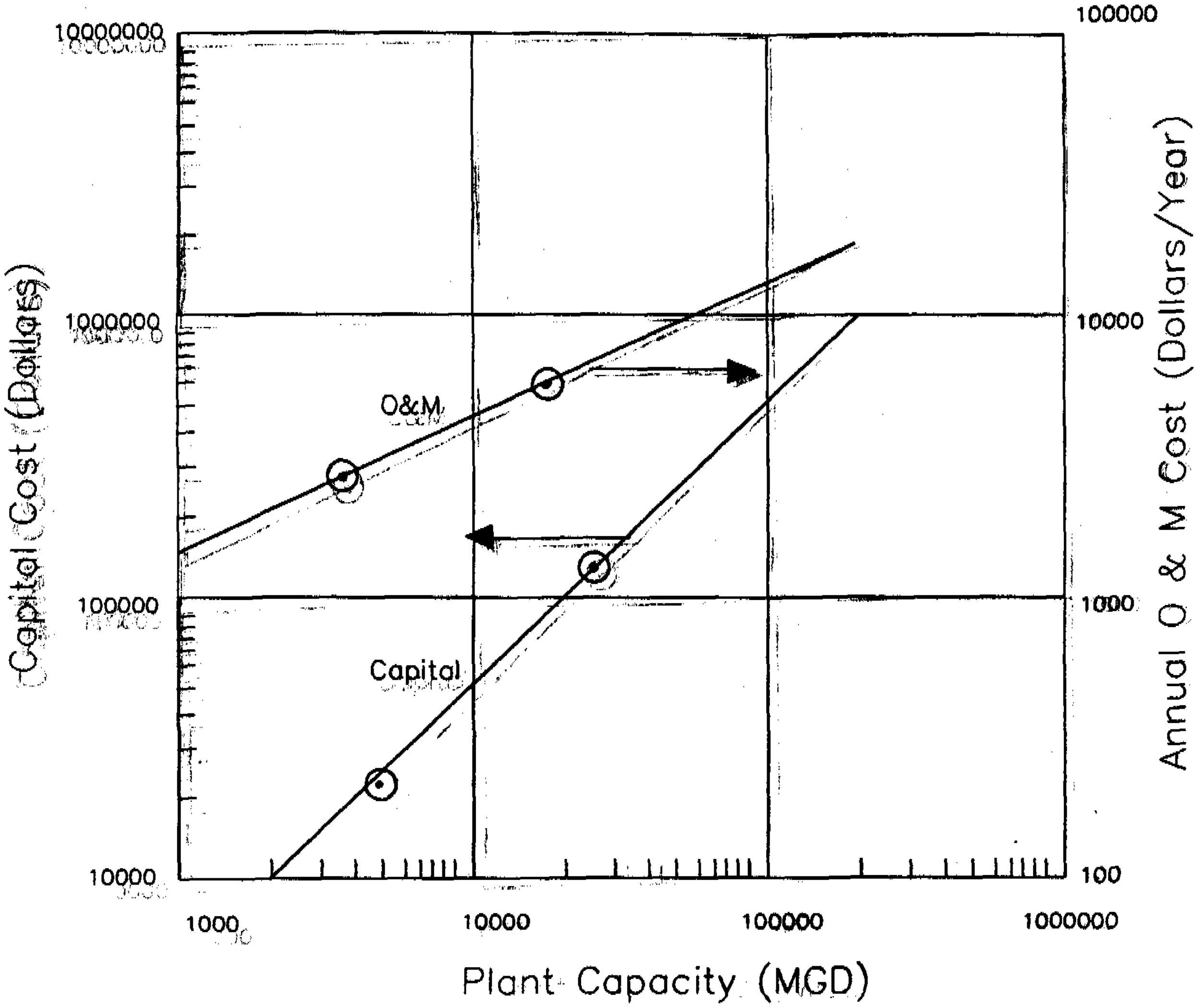
43.6 Availability

The technology, equipment and hardware typically utilized in intermittent sand filters should be available locally in most areas of the world. The critical component is the sand media, which often is available locally.

43.7 Operation and Maintenance

Intermittent sand filters do not unload. Untreated sewage leaves a mat of solids upon the surface of the bed. On drying, this mat cracks and curls and is stripped from the bed. The solids left in settled sewage penetrate further into the sand. This necessitates occasional removal of the surface sand and resanding the bed. Sludge mat and waste sand are generally used to fill low-lying areas in the vicinity of the plant. Raking, harrowing and plowing will open up a clogged bed but may

Figure 43.2: Intermittent Sand Filter Costs *



* Costs Shown Are Exclusive (Do Not Include) Land Cost

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ultimately intensify clogging by forcing solids deeper into the sand. Light harrowing is useful in releveling beds and loosening sand that has been packed down by heavy rains. These procedures apply to open filters only. Buried filters are designed to perform without maintenance for up to 20 years.

Pooling should not be allowed to develop on the beds as this tends to produce septic action, obnoxious odors and an effluent of poor quality. Pooling indicates that cleaning is necessary. The surface of the beds should be kept level to afford uniform distribution of the sewage and weeds, grass, etc., should not be allowed to grow on the beds. However, slight odors are unavoidable in plants filtering raw sewage.

In cold climates, winter operation required furrowing the surface and dosing the bed deeply on a cold night so that a sheet of ice will form and span the furrows between the ridges. The bed is kept from freezing, and the sewage underruns the ice sheet. Beds should be thoroughly cleaned before and after they have been furrowed.

43.8 Control

Intermittent sand filters require relatively little operational control. Once wastewater is applied to the filter, it takes from a few days to several weeks before the sand has matured. BOD and SS concentrations in the effluent will normally drop rapidly after maturation. Depending upon media size, rate of application, and ambient temperature, nitrification may take from 2 weeks up to 6 months to develop. Cold weather (winter) start-up should be avoided since the biological growth on the filter media may not develop properly.

A small measure of process control can be achieved through variations in Loading Rate, Dosing Cycle, Resting Period and Frequency of Cleaning.

43.9 Special Factors

Intermittent sand filters should be preceded by some form of pretreatment. Septic tanks, Imhoff tanks, Primary Sedimentation/Digestion, Trickling Filters and Activated Sludge can be used depending on the level of effluent quality desired. Filter effluent should be disinfected using chlorination or ultraviolet irradiation.

Intermittent sand filters are not applicable in the treatment of those industrial wastes which contain constituents toxic to biological processes and/or resistant to biodegration.

Care must be exercised when siting intermittent sand filtration systems in natural sand deposits to insure that percolation into the groundwater table and subsequent pollution of groundwater aquifers does not occur. In areas where percolation into an aquifer could pose a threat to groundwater quality, the filter must be provided with an impermeable base or lining thereby increasing construction costs.

43.10 Recommendations

Intermittent sand filtration is a well proven and highly stable process and a relatively low cost method of wastewater treatment which produces a high quality effluent. The process is highly reliable and requires minimal operator attention. Energy requirements are low, less than 0.3 HP-hr per 100 gallons (0.55) KWhr per m 3) and O&M can be performed by unskilled labor. Removal efficiencies for BOD and SS can exceed 95% and concentrations of ammonia nitrogen in the filter effluent are normally less than 5 mg-N/L with appropriate pretreatment. The process is most suited for treatment of domestic/commercial wastes for flows generally less than 0.2 MGD (0.01 m^3/sec).

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SECTION II

CASE STUDIES OF OTHER APPROPRIATE TECHNOLOGIES

II.A Introduction

The technologies in this section are presented because they are felt to have relevance for application in Latin America and the Caribbean. Note that in some cases additional information is presented in Section I. The technologies should be considered for application, in that the results thus far are promising. Each has been applied at least five times in the US and/or other parts of the world. The technologies are likely to be applicable for many parts of the study area.

Most of the technologies in this section apply to wastewater, but since a number of them relate to water reuse, the distinction becomes clouded, as it should be. All of the technologies are applicable to treatment of contaminated water (water supply sources or wastewater). Many sludge treatment technologies are applicable to both water and wastewater treatment sludges. Figure II-1 shows schematically how water reuse may be applied (ref: II-9).

Many countries in Latin America are water short. Many areas of other countries are water short. Engineers should consider that application of water treatment technologies is a continuum, starting with highly contaminated water to less contaminated. Systems should be designed for meeting water quality criteria for various water uses, and not on the basis of presumed water supply vs. wastewater distinctions. Water used for carriage of wastes is undoubtedly water destined for drinking at another time and place. There is necessarily, some overlap with Section I. This Section presents specific examples taken from studies which are underway (refs: II-1, II-2).

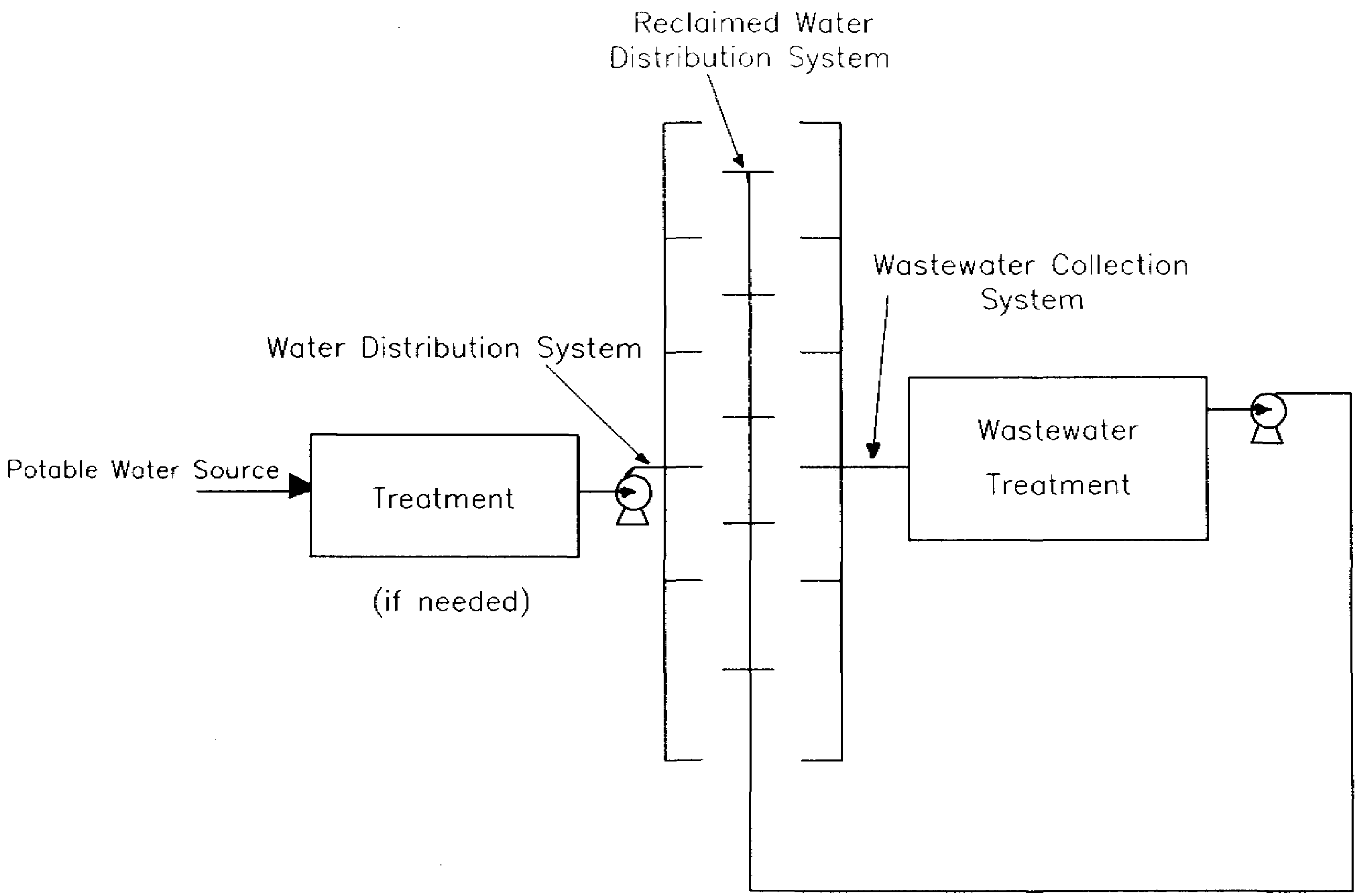

Figure II–1: Dual Distribution System Schematic.

II.B Innovative Technologies

An innovative technology is defined as a water treatment process or technique which has not been fully proven for the proposed application and which offers a significant advancement over the state of the art. In order to qualify as innovative, a technology should meet two conditions. First, the technology or its application must include an inherent risk which is outweighed by a corresponding benefit, thereby making the risk acceptable. If a technology or its application is fully proven, there would be no "risk" involved and it could not qualify as innovative. However, if a specific application of a proven technology is not proven, the specific application may qualify as innovative.

The second condition is that the technology should meet one or more of a series of criteria which measure its advancement over the state of the art. Six of these are suggested: 1) cost reduction, say in the order of 15% on a life cycle basis, 2) reduction in the use of energy, again in the order of 15%, 3) improved removal or destruction of a toxic substance, 4) improved operational reliability, 5) improved environmental benefits, 6) improved potential for joint treatment of industrial and municipal wastes, or for water reuse.

The following data may lend perspective to the application of innovative technologies. The distribution of the full range of innovative technologies which have been applied within the last 5 to 10 years is (all technologies are not included):

aeration	11.4%
clarifiers	10.9%
disinfection	9.4%
energy conservation & recovery	5.0%
filtration	5.3%
lagoons	7.8%
land application of effluent	4.4%
nutrient removal	7.7%
oxidation ditches	7.0%
sludge technologies	10.7%

Table II-B.1 presents a summary of some innovative technology applications in the U.S., broken down by applications in the northern and southern portions. Some, notably counter-current and draft tube aeration, oxidation ditches, and ultraviolet disinfection have probably been used often enough to classify them as alternative technologies.

TABLE II-B.1

SUMMARY OF INNOVATIVE TECHNOLOGIES FUNDED FIVE OR MORE TIMES

TECHNOLOGY	SOUTHERN U.S., HAWAII AND TERRITORIES	NORTHERN U.S., ALASKA
Anoxic/Oxic Systems	1	4
Counter-Current Aeration	21	-
Draft Tube Aeration	6	15
Fine Bubble Diffusers	2	8
Flocculating Clarifiers	-	5
Hydrograph Controlled Release Lagoons	18	1
Integral Clarifiers	1	4
Land Treatment	11	6
Microscreens	3	3
Oxidation Ditches	21	15
Phostrip Activated Sludge	1	4
Sequencing Batch Reactors (SBR)	7	9
Single Cell Lagoons with Sand Filter	-	10
Small Diameter Gravity Sewers	1	9
Solar Heating	48	
Swirl Concentrators	1	7
Trickling Filter/Solids Contact	4	6
Ultraviolet Disinfection	5	37
Vacuum Assisted Sludge Drying Beds	5	6

II.C Alternative Technologies

An alternative technology is a fully proven method of water, wastewater or sludge treatment that has not been used frequently. It is a technology that 1) provides for the reclaiming and/or reuse of water, 2) productively recycles treated water constituents, 3) eliminates the discharge of pollutants with water or treatment sludges/residues, 4) recovers energy. Alternative technology may be considered to emphasize land treatment of contaminated water and sludges, sludge/residue handling and disposal techniques that reuse or reclaim pollutants, on-site methods of disposal at the household, industrial park, or small community level, and alternative methods of waste conveyance that are especially applicable to small communities.

Composting of sludge and land treatment of wastewater and sludge are perhaps the best known alternative technologies. Some other technologies, although proven, are less known because of infrequent use. Effluent treatment alternative technologies include aquifer recharge, aquaculture, revegetation of disturbed lands, horticulture, direct reuse (non-potable), and total containment ponds. Energy recovery alternative technologies include self sustaining incineration and anaerobic digestion with greater than 90 percent methane recovery and use. For small community systems, alternative technologies include individual or cluster on-site treatment, septage treatment, small diameter collection and conveyance systems such as pressure sewers, and some centralized treatment systems.

Alternative technologies are distributed as follows:

land treatment	29.1%
onsite treatment (small systems)	11.7%
collection systems	19.5%
energy recovery form sludge	7.1%

sludge treatment	24.8%
other	7.8%

II-C.1 Table II-C.1 lists the alternative technologies which have been implemented in the U.S. within the past several years. These data illustrate the heavy recent use of land treatment for both water and sludge, and energy recovery.

TABLE II - C.1

SUMMARY OF ALTERNATIVE TECHNOLOGY PROJECTS

TECHNOLOGY	SOUTHERN U.S., HAWAII AND TERRITORIES	NORTHERN U.S., ALASKA
LAND TREATMENT		
Aquaculture/Wetlands/Marsh	6	14
Overland Flow	29	20
Rapid Infiltration	26	43
Slow Rate	172	116
Preapplication Treatment or Storage	69	61
Other Land Treatment	2	20
COLLECTION SYSTEMS		
Pressure Sewers/Effluent Pump	25	57
Pressure Sewers/Grinder Pump	31	107
Small Diameter Gravity Sewers	34	117
Vacuum Sewers	4	14
ENERGY RECOVERY/SLUDGE		
90% Methane Recovery/Anaerobic Digestion	42	85
Self-Sustaining Incineration	5	8
SLUDGE TREATMENT		
Land Spreading of POTW Sludge (Publicly Owned Treatment Works)	74	286
Preapplication Treatment	14	25
Composting	12	41
Other Sludge Treatment or Disposal	11	26
OTHER		
Aquifer Recharge	1	1
Direct Wastewater Reuse	15	5
Total Containment Ponds	37	96

Latin America and the Caribbean must make use of water and energy recovery techniques. Even areas of high rainfall in the Caribbean for example can make use of water reuse technologies because of the seasonal nature of high rainfall volumes. High intensity rainfall common to some areas does not help shortage problems significantly. Even capture of high intensity rain water is probably not cost effective because of high evaporation rates.

Any water/wastewater management program can make effective use of energy recovery, and materials recovery (even simple recovery of the soil conditioning benefits of sludge), regardless of the economic and water availability status of the country.

II.D Other Appropriate Technologies

In the following section, technologies are briefly described which have applicability for Latin America and the Caribbean. In each case an attempt has been made to include as much information as possible to allow an evaluation. It may be seen that less information is provided than in Section I. The information provided has been in general, based on actual applications and is provided in the categories of:

Description - short written description including a diagram where applicable.

Application - notes on the applicability of the technology and guidance on applicability elsewhere, where possible.

Benefits - general performance information. Usually, the full results are not available as yet for a particular application. Also, benefits probably outweighed potential disadvantages, or the technology would not have been selected for the project.

Status - If available, notes are provided for the status of the technology for other locations than the one being tested. This gives perspective on the general use patterns.

Findings - If a project is in early stages data and information about progress is presented.

II-1 Bardenpho Process

II-1.1 Description

The Bardenpho process was originally developed as a four stage system for BOD and nitrogen removal from wastewater where partial removal of phosphorus also occurs. In order to maximize phosphorus removal, an anaerobic stage is added to the front of the four-stage process. Nitrogen and phosphorous removal are achieved by carefully controlling the concentration of oxygen in each of the five stages. Nitrogen removal is by biological denitrification, while phosphorus removal is by microbial uptake into the waste activated sludge. The process is shown schematically in Figure II-1.1.

II.1.2 Application

The Bardenpho process is applicable to wastewater systems where phosphorus and/or a nitrogen discharge is of concern. The basic four-stage system can be used when the discharge is nitrogen, but not phosphorus, limited. By adding the fifth stage, the system can be used where the discharge is phosphorus limited.

II.1.3 Benefit

Chemicals do not have to be added to remove nitrogen and phosphorus, but are probably required if low nutrient concentrations in the effluent are necessary. Capital costs and maintenance costs are lower for non-chemical systems since chemical handling facilities are not required. Operating costs may be reduced when reducing the need for additional digestion equipment; and thereby potentially lowering capital, operating, and maintenance costs of sludge treatment and disposal. Operation of the process is claimed to be similar to conventional activated sludge system operation, but is probably more

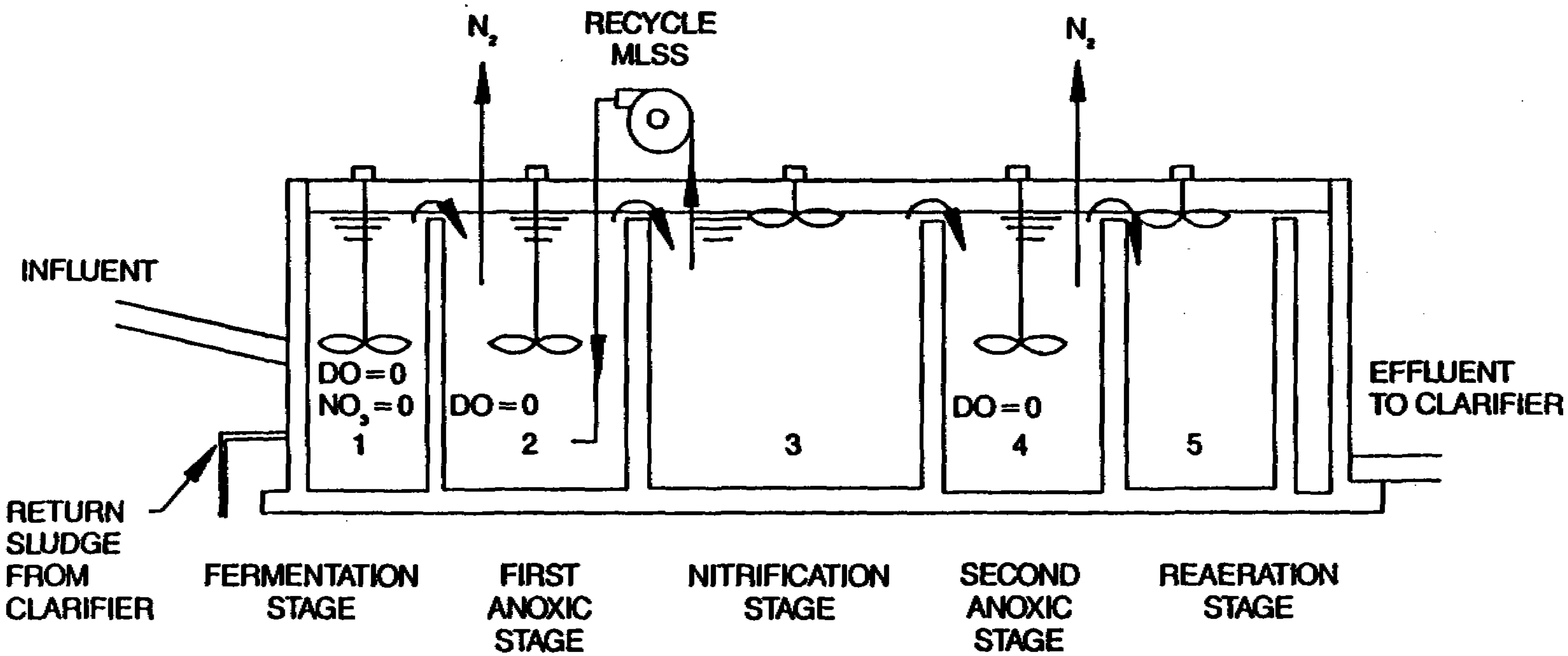

Figure II–1.1: Bardenpho Process Flow Diagram.

complicated. The long solids retention time provides process stability.

II.1.4 Status

There are nine wastewater treatment facilities in the U.S. using the Bardenpho· process. Worldwide, there are another forty systems in operation. At present, there are six facilities under construction and another eight being designed. The operating systems consistently report good removal of nitrogen and phosphorus. However, alum must be added to enhance phosphorus removal in cases where effluent standards require consistent phosphorus levels at or below 1.0 mg/L.

II-2 Biological Aerated Filters (BAF)[1], Oneonta, AL

II-2.1 Description

Prior to the addition of the Biological Aerated Filters (BAF), the Oneonta, AL, treatment system was a single lagoon. The BAF units were added to achieve effluent limits that are BOD, ammonia-nitrogen, and suspended solids limited. The treatment system is a 2.2 mgd (0.1 m^3/sec) system consisting of two pond cells, an aerated channel, eight BAF units, and chlorination prior to discharge. It serves a population of approximately 4,500. The BAF units are high rate, attached growth, aerobic treatment units which use a patented catalyst bed to remove soluble and suspended organic material. See Figure II-2.1.

II-2.2 Application

The BAF process could be used in many systems where improved BOD and suspended solids removal is required, especially where low effluent limits are required. BAF units may also be applicable where nitrification is required. If land is limited, BAF units can be especially attractive. Table II-2.1 presents some design parameter ranges.

II-2.3 Benefit

Capital costs are reduced because secondary clarifiers and effluent filters are not required, costly aeration basins are not required, and labor for installation is reduced. BAF units require less land than conventional systems, providing another potential saving. O&M cost savings are claimed because energy requirements are potentially less than for conventional systems; there are no chemical requirements; personnel requirements are

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[1]The BAF system was developed and is patented by OTV of Paris, France. EMICO in Salt Lake City, Utah, has exclusive marketing rights for the patented BAF system in the United States.

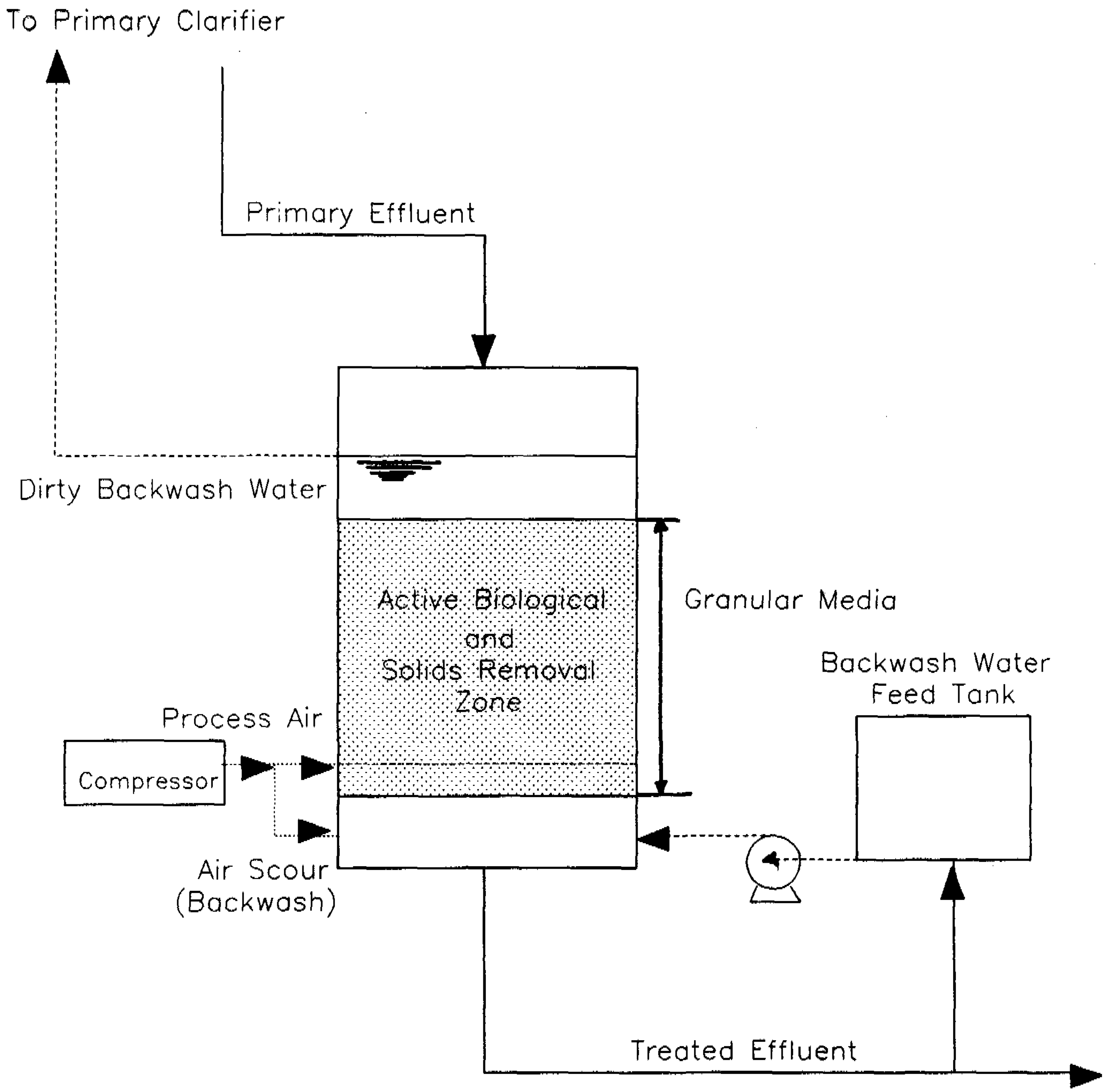


Figure II–2.1: Biological Aerated Filter.

TABLE II - 2.1

TYPICAL BAF DESIGN RANGES

Effluent Quality	
BOD_5	10-20 mg/1
Suspended Solids	10-20 mg/1
Dissolved Oxygen	0.5-2.0 mg/1
Removal Efficiency	
BOD_5	80-96 percent
Suspended Solids	85-96 percent
Bed Depth	5-6 feet (1.5 - 1.8 m)
Organic Loading Rate	140-350 lb. BOD_5/day/ 1000 cu. ft. of media volume (2.2-5.6kg/da.1)
Detention Time	30-80 minutes
Sludge Production	0.4-0.7 lb./lb. BOD_5 removed
Air flow	2-5 scfm/ft^2 (600-1500 l/min./-m^2)

reduced by automating process cycling. BAF units can reduce BOD to below 10 mg/L and effluent ammonia to 1 mg/L, when properly designed according to influent BOD loading. Effluent suspended solids are generally less than 10 mg/L. BAF systems are simple to operate. BAF· has certain features which should reduce costs: land requirements one-fifth to one-tenth of conventional systems, no secondary clarifier or filter required, single source of sludge, simplicity of operation, (ref: II-3).

II-2.4 Status

The system has been achieving effluent concentrations which are better than the required limits since start-up. Studies are being conducted to optimize performance and reduce power costs. Systems have been successfully operated in France since 1978. In the United States, there are four BAF· systems in operation, one under construction, and one in design.

II-3 Teacup Grit Removal System, Calera, AL

II-3.1 Description

The wastewater treatment system at Calera, AL consists of twin Teacup solids classifiers with stacked static screens, counter-current aeration, clarification, and chlorination. The system has a design flow of 750,000 gpd (33 l/s). The effluent is BOD, ammonia-nitrogen, and suspended solids limited. The Teacup solids classifier (see Figure II-3.1) removes grit by a combination of centrifugal and gravity forces. Flow enters tangentially near the top, creating a free vortex, and resultant centrifugal forces. Grit particles settle toward the bottom, where the free vortex boundary layer sweeps them to a central well. Acceleration within the boundary layer separates the particles by density. The denser grit particles are separated and removed, while the less dense organics tend to remain with the wastewater.

II-3.2 Application

The Teacup solids classifier is applicable to a variety of wastewaters, including municipal treatment systems, food processing wastewater reclamation, and industrial cooling waste reclamation. The system can be used in any system where grit accumulation and/or damage is a problem.

II-3.3 Benefit

If grit is not effectively removed from wastewater before it enters a treatment system, it adds sludge volume, additional sludge solids, and abrasives which cause excessive wear on mechanical equipment. All of these increase operation and maintenance costs. Removal of grit decreases costs and maintenance time. The Teacup solids classifier removes 95 percent of the grit under peak flow conditions. The grit removed

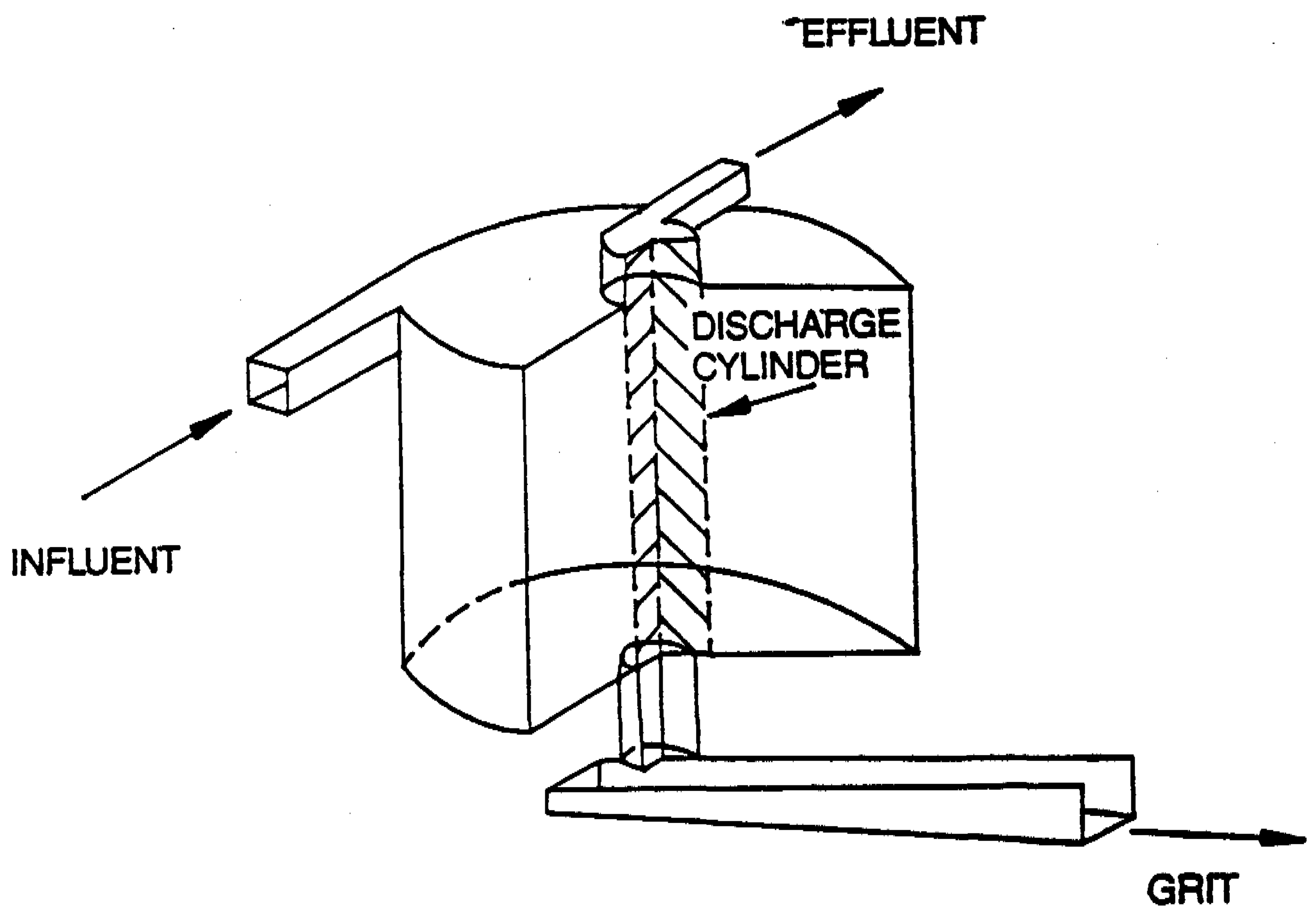


Figure II–3.1: Teacup Grit Removal/Solids Classifier.

is less than 15 percent organics, which results in less odor than typical biological sludge. Since grit is still about 15 percent organics, careful disposal is necessary, and treatment may be required beforehand. The Teacup solids classifier is a hydraulic system, which saves energy and reduces maintenance. The Teacup has no moving parts, which reduces maintenance costs. The aerated discharge maintains dissolved oxygen levels.

II-3.4 Status

The Teacup solids classifier at Calera is performing as designed. The system is removing more than 95 percent of the grit in the influent. There are no odor problems.

II-4 Pressure Sewer Technology

II-4.1 Description

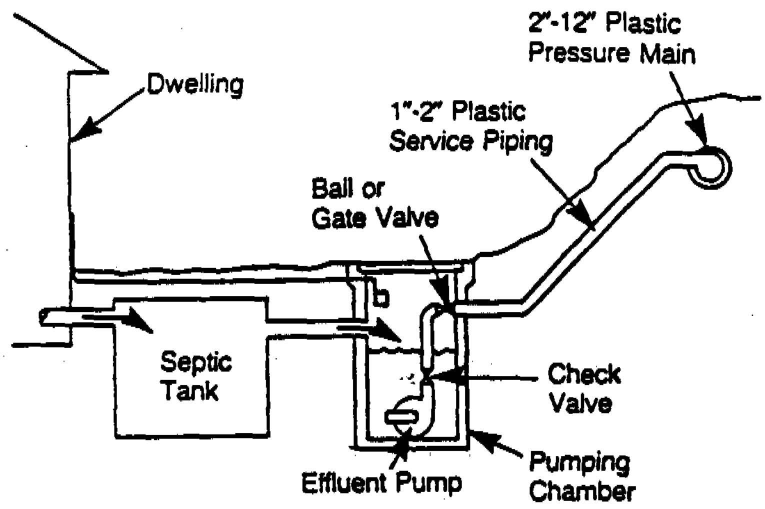
There are two basic variations of on-site pressure sewer systems: septic tank effluent pump (STEP) units and grinder pump (GP) units. Both are shown on Figure II-4.1. STEP systems consist of a septic tank, a wet well with an effluent pump, and accessories such as valves and level control system. GP systems have a pumping chamber storage tank and a grinder pump with accessories similar to a STEP system. Both system variations pump wastewater into small diameter, sealed sewer lines. As illustrated, the STEP systems produce a wastewater with lower organic loading than conventional sewers due to pretreatment in the septic tank; whereas, GP systems produce a wastewater with higher than normal organic loading due to little or no dilution from infiltration/inflow (I/I).

II-4.2 Application

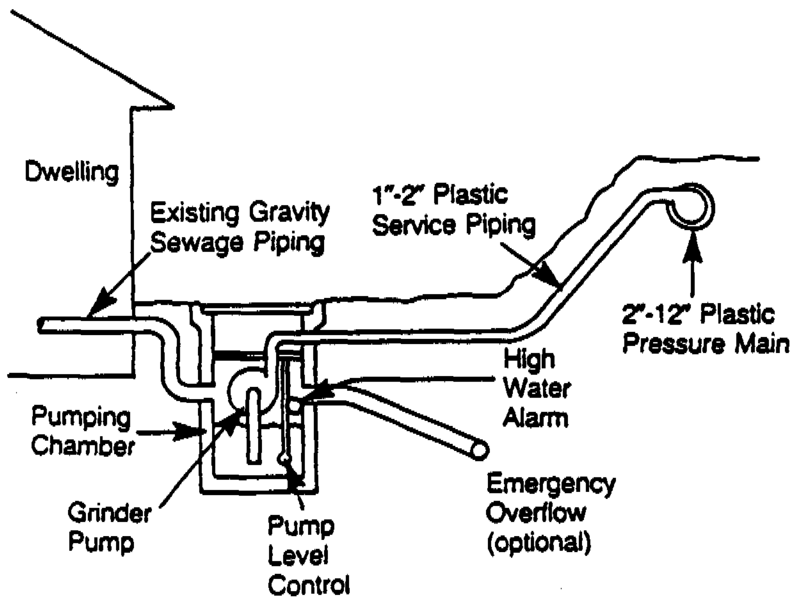
Pressure sewers allow small and/or widely dispersed communities to add collection/generation areas as sporadic growth occurs. This type of system is well suited where the treatment plant is uphill of the collection system, but can also be used effectively in areas of slight or widely varying topography. Pressure sewers are very advantageous in areas with shallow bedrock or high ground water tables.

II-4.3 Benefit

Initial costs are lower than gravity sewers due to easier installation using smaller diameter pipe; shallower, narrower trenches; and non-critical variable grade which can be adjusted for specific site conditions. System expansion can be accomplished one house at a time without the need to install large collector lines based on future expansion projections. The



Pressure Sewer System
Using Septic Tank (STEP)



Pressure Sewer System
Using Grinder Pump (GP)

Figure II–4.1: Pressure Sewer Technology.

sealed pipe system reduces I/I which may result in smaller treatment system sizing, thereby saving capital costs.

II-4.4 Status

Pressure sewer systems are applicable in numerous communities where conditions are not conducive to gravity systems, and/or growth of the area warrants this type of system. In order to select this technology, capital costs must be low enough to offset higher operating costs. Construction with corrosion resistant valves, water level sensors, and switches should increase long-term reliability and ultimately decrease O&M costs.

II-5 Grinder Pump Wastewater Collection System, Greene County, VA

II-5.1 Description

The Greene Mountain Lake Subdivision is located in a rough terrain area downhill from an existing wastewater collection system. To connect the two systems, a small low pressure system with individual grinder pumps at each residence was designed. Each residential station has a sixty-gallon storage tank which is pumped at a predetermined tank capacity by a two-horsepower packaged grinder pump. The collection system includes 1.5-inch to 4.0-inch low pressure mains connected to a central pump station which discharges to the existing gravity collection system. The system is designed to serve approximately 120 residences. See Figure II-5.1.

II-5.2 Application

Any area of wastewater generation that is topographically isolated from collection/treatment facilities can benefit from this technology; provided the cost of pumping required to overcome the topography is cost-effective. Additional applications might include state parks, recreational/second home developments, or business parks. Low gradient areas (e.g., beach communities) might also benefit by using this system.

II-5.3 Benefit

The grinder pump/low pressure wastewater collection system may reduce the number and/or size of major pump stations required by an equivalent gravity collection system. The collection lines can also be located in existing road rights-of-way at shallow depths avoiding stream channels. Small shallow lines following the mostly uphill topography provided a cost savings in this project. The closed nature of the system also reduces inflow and

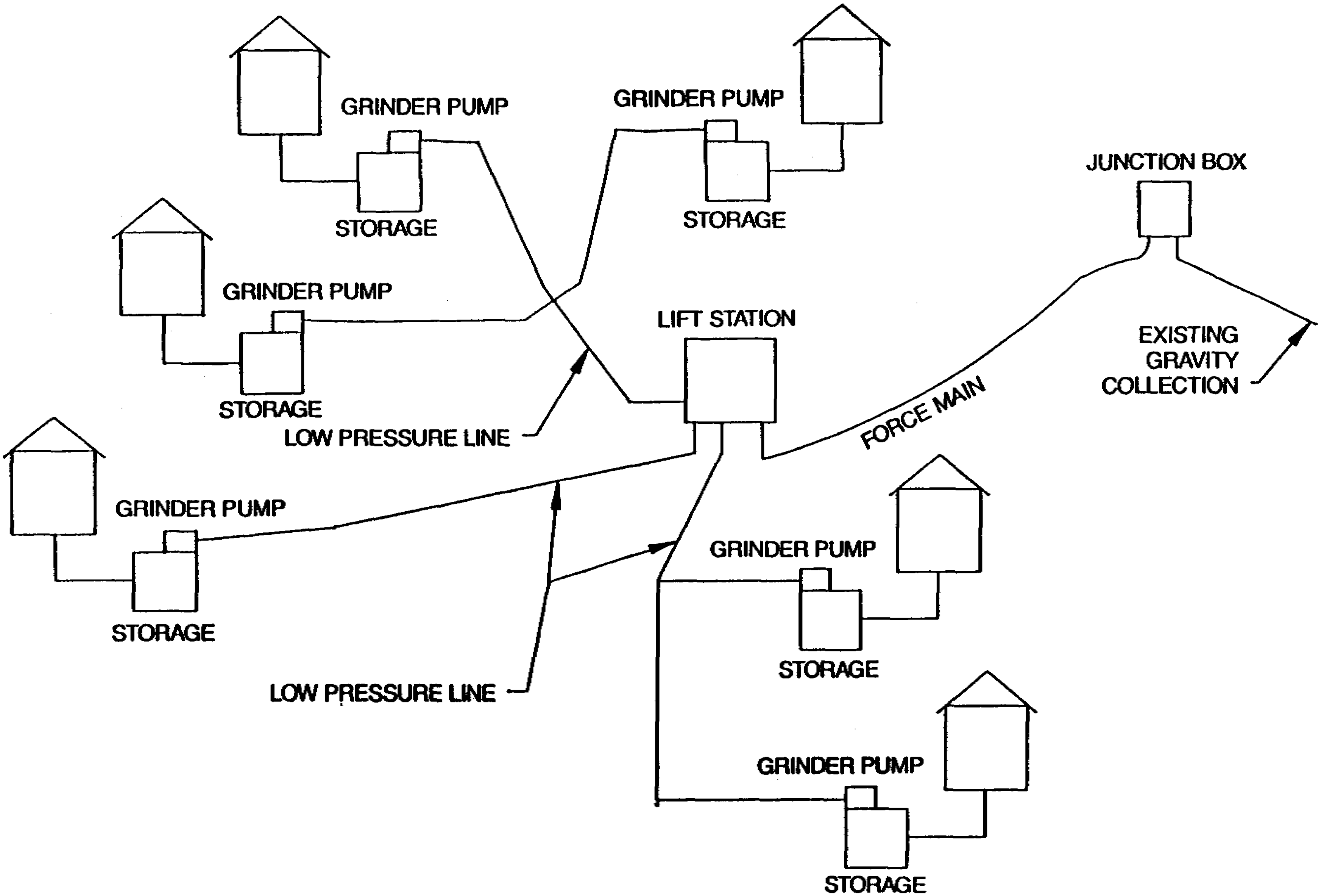


Figure II–5.1: Grinder Pump Flow Schematic;
Greene Co., Va.

infiltration, providing an additional cost savings to the system operation.

II-5.4 Status

The Greene County wastewater collection and treatment facilities has an expected completion date of September 1988.

II-6 Small Diameter Effluent Sewers, Mt. Andrew, Alabama

II-6.1 Description

The Mt. Andrew, Alabama small diameter effluent sewer system was installed in 1975 and serves a subdivision community of 31 houses. The system consists of modified septic tanks, small-diameter transport lines, and a lagoon for final treatment. The system uses 2-inch and 3-inch PVC gravity lines and a 3-inch pressure/gravity line grade, and the effluent from these houses is pumped to the collection line. Collection lines were installed along the existing grades, independent of the elevation and without manholes or cleanouts. The collection line grades go uphill at several points. See Figure II-6.1.

II-6.2 Application

Small diameter effluent sewers are best suited to reasonably small user groups which will not be experiencing large amounts of growth.

II-6.3 Benefit

The benefits to Mt. Andrew are: 1) lower installation costs due to the use of small diameter pipe and pipe installation following existing contours, which eliminated costly deep cuts and list stations; 2) a reduced number of manholes, cleanouts, and associated infiltration/inflow; and 3) negligible maintenance costs due to smaller pipe sizes and an essentially closed system. Advantages and disadvantages are given in Table II-6.1 (ref: II-8).

II-6.4 Status

The small diameter effluent sewer system at Mt. Andrew has been operating satisfactorily since 1975. The transport lines

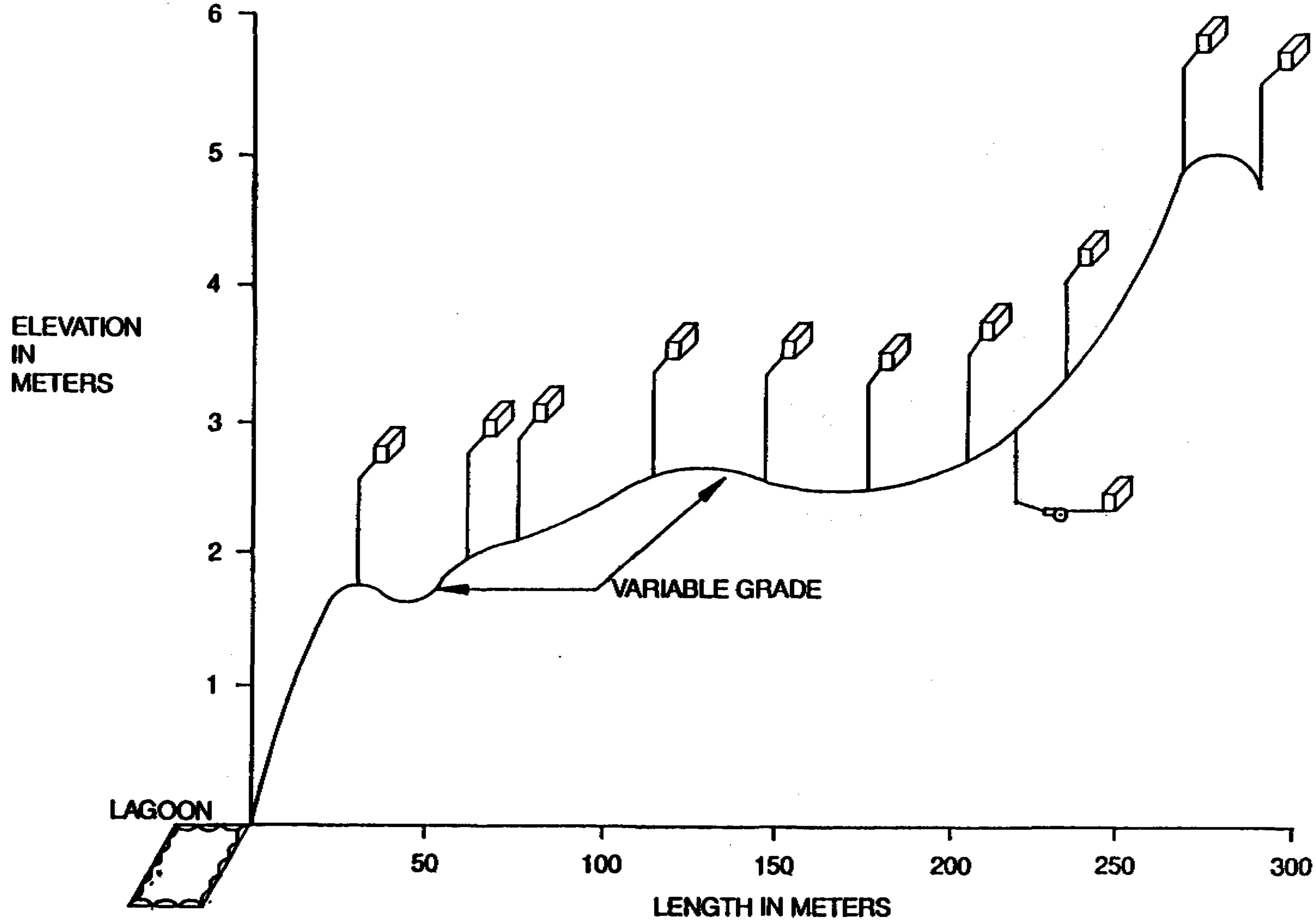


Figure II–6.1: Small Diameter Effluent Sewers;
Mt. Andrew, AL.

TABLE II-6.1

SMALL DIAMETER GRAVITY SEWERS

Advantages	Disadvantages
Low construction cost	Each Service connection requires septic tank
Relatively low operation/ maintenance requirements	Septic tank cleaning and septate disposal required
Reduced grit, grease, and solids present in waste flow	Additional pump stations may be required in hilly areas
Sewers can be installed at lesser gradients	Potential odor/corrosion problems
Cleanouts used instead of manholes	May require mainline flushing
Reduced infiltration/ exfiltration	

have proven to be very reliable with only minimal maintenance requirements. The modified septic tanks have functioned as designed, although rapid solids buildup in the primary section of the tanks occurred due to their initial undersizing which caused more frequent pumping than anticipated.

II-7 Communal Treatment System, Mayo Peninsula, Maryland

II-7.1 Description

The decentralized wastewater treatment project developed for the 8-mile Mayo Peninsula, Maryland, includes three treatment approaches. One approach is on-site septic systems in areas with suitable soils. The second approach is cluster soil absorption systems where septic tank effluent is collected from several homes and conveyed to an area with soils suitable for a communal infiltration field. The final approach (shown on Figure II-7.1) is a 0.9 mgd (0.04 m^3/sec) communal treatment system for the majority of the peninsula. The communal system starts with collection and discharge of septic tank effluent into seven acres of recirculating sand filters. Following this sequentially are a 7-acre constructed bulrush/cattail wetland, with intermediate ultraviolet disinfection, an 8-acre constructed peat wetland with final ultraviolet disinfection, and final discharge into constructed, offshore, submerged wetland.

II-7.2 Application

Decentralized systems are feasible for rural areas with widespread clusters of population. Systems similar to the Mayo Peninsula project enhance the current lifestyle, while not contributing to unplanned growth. Areas striving to maintain a simplified infrastructure could benefit from a decentralized waste treatment plan.

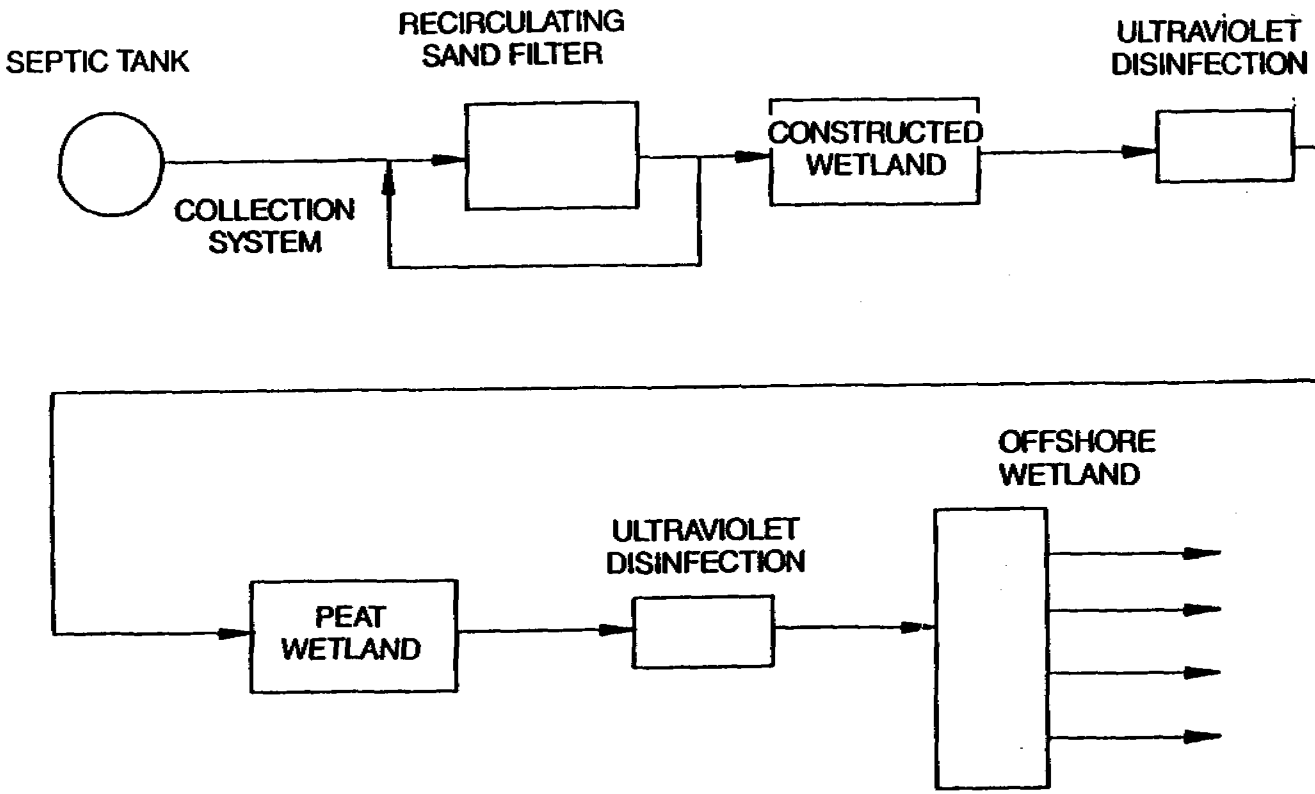


Figure II–7.1: Communal System Flow Diagram; Mayo Peninsula, MD.

II-7.3 Benefits

Following a history of failed septic tank systems with the accompanying flooding and adversely affected well water quality, local residents encouraged development of a system which would treat the residential wastes, but would not contribute to rapid development of the area. The decentralized system will achieve the community goals while reducing initial costs by $12 million when compared to conventional systems.

II-7.4 Status

The system is completely installed and operational as of 1988.

II-8 Constructed Wetlands Systems Technology

II-8.1 Description

A constructed wetlands (CW) system is essentially a lateral, subsurface flow trickling filter. Primary or secondary treated wastewater flows into a long, shallow trough filled with a stone base and topped with a layer of pea gravel (shown as a dotted cross section) supporting rooted aquatic plants. Figure II-8.1 shows a system treating septic tank discharge. The biological treatment of the wastewater is restricted to the aerobic root zone below the pea gravel surface. Open surface and root/rhizone-produced aeration provide the necessary oxygen. Degradation of organic material by bacteria in the root zone produces substances (e.g., metabolites) which are assimilated by plants. In turn, microorganisms utilize plant metabolites and dead plant material as a food source.

III-8.2 Application

CW systems have a wide range of applications for small to medium size residential, commercial, and industrial waste streams. Following primary treatment to prevent gravel clogging, the CW system can serve as a secondary or tertiary level of treatment. The most promising application may be the replacement of septic tank drain fields. CW systems are also being used to treat river water contaminated with organic pollutants, acid mine drainage, and agricultural runoff.

II-8.3 Benefit

The CW concept has the potential to lower capital and O&M costs compared to conventional mechanical treatment alternatives. The process is flexible and can be designed to meet specific treatment needs, including the removal of toxics and nutrients. Reeds used in CW systems should have a wide range of tolerance

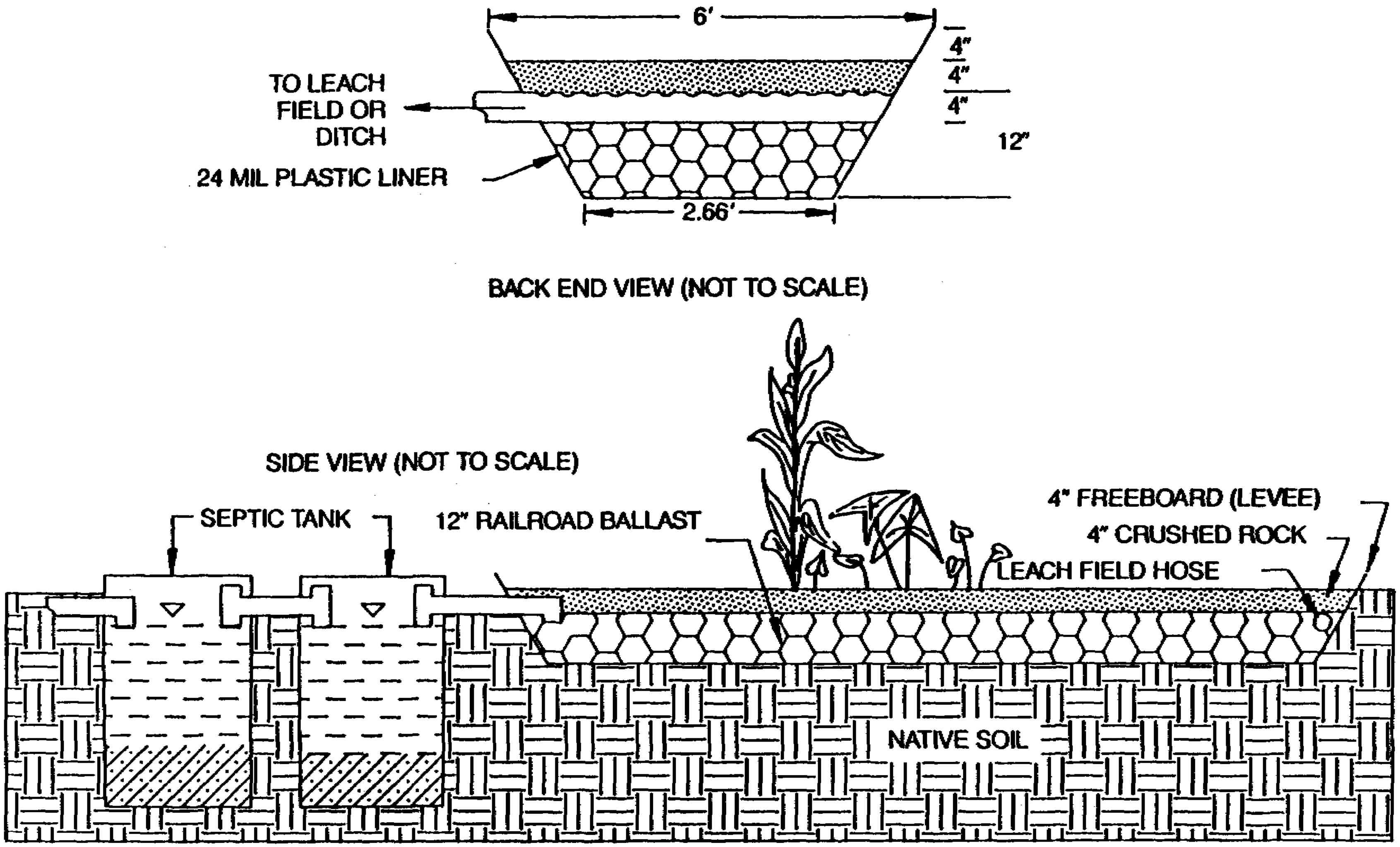


Figure II–8.1: Constructed Wetlands.

for temperature, salinity, and toxicity. This will expand its applicability. Compared to floating marsh treatment system, the CW system requires less land area. The CW system has a nice appearance and the biomass produced may also have an economic value.

II-8.4 Status

The National Space Technology Laboratories Station in Mississippi is operating three CW systems. Several systems are currently being designed or constructed in Alabama, Louisiana, and Mississippi. The Public Health Service is designing a system at their hospital facility in Corvallis, Mississippi.

II-9 Pulsed Bed Filtration (PBF), Clear Lake, WI

II-9.1 Description

The primary purpose of this field test was to evaluate the ability of PBF to reduce organic loading to secondary biological treatment systems and, thereby, increase the operational performance. The filter selected was the Hydro Clear Pulsed Bed Filter developed and marketed by Zimpro, Inc. It uses a shallow bed of fine sand with an air diffuser just above the bed's surface to keep solids in suspension. See Figure II-9.1. Periodically, an air pulse is generated through the backwash/underdrain system that re-suspends trapped solids and/or distributes them throughout the bed. After a set number of pulses, the filter is backwashed through the underdrain system. A semi-automatic grease cleaning system restores the sand to its original condition. The PBF was tested in the primary filtration mode utilizing primary clarifier and/or roughing filter effluent.

II-9.2 Application

In addition to primary effluent filtration, PBF has proven effective in the filtration of raw water supplies, process waters, wastewater roughing streams, cooling tower water, and boiler feed water.

II-9.3 Benefit

The benefits of primary filtration include the removal of large quantities of solids, increased capacity of existing secondary biological treatment facilities, and reduction of biological treatment sludge.

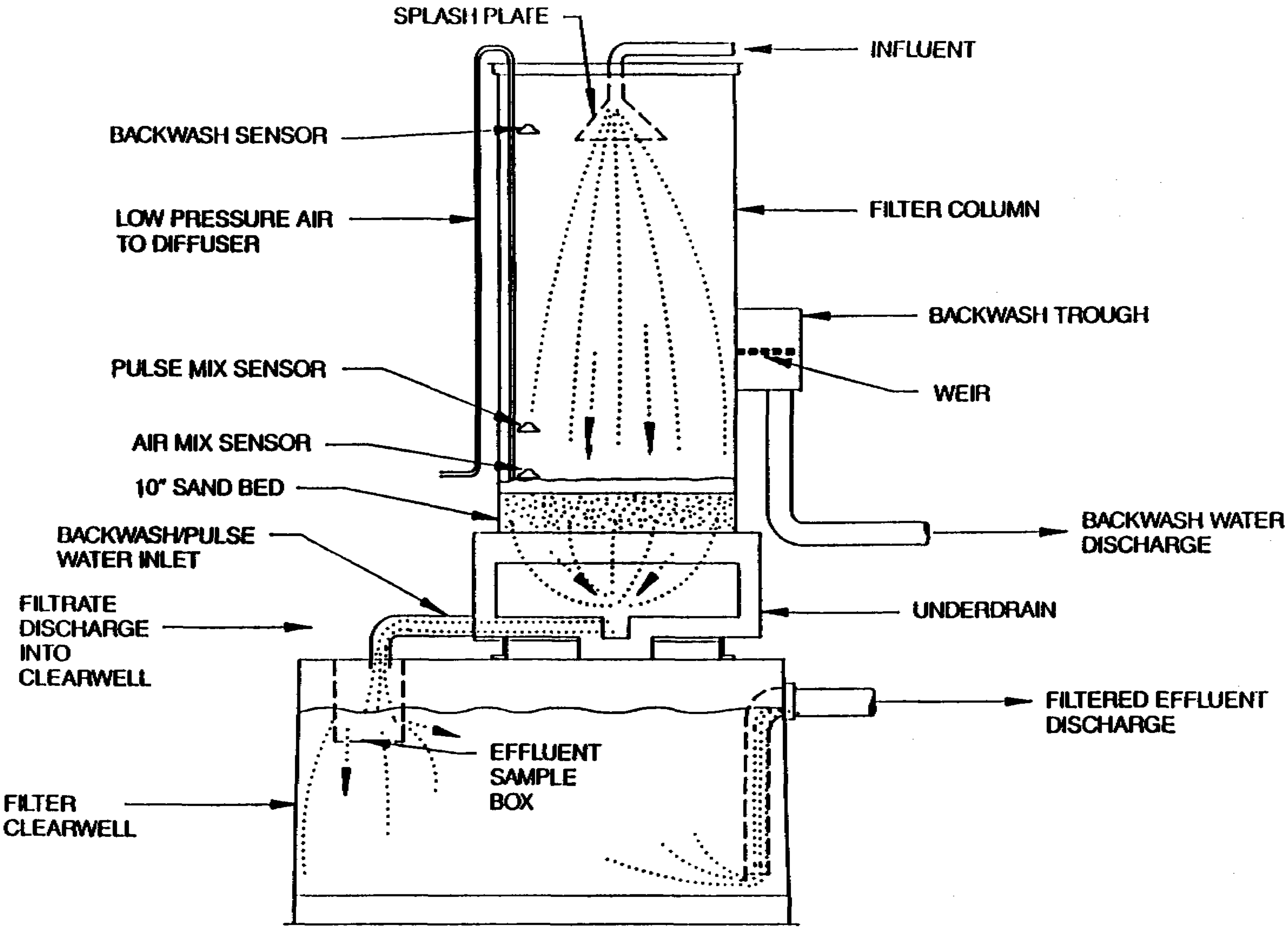


Figure II–9.1: ZIMPRO Pulse Bed Filtration; Clear Lake, WI.

II-9.4 Findings

Throughout the two month field test, the PBF reduced suspended solids by an average of approximately 52 percent, with a corresponding average reduction of approximately 24 percent in total BOD at the trickling filter effluent. The best results were achieved during the third of five test periods when the discharge to the PBF was changed from the combined primary/roughing filter effluent to roughing filter effluent only. The additional biological activity in the roughing filter produced a higher proportion of larger particle sizes which were more amenable to filtration.

II-10 Anoxic/Oxic (A/O) Biological Nutrient Removal, Fayetteville, AR

II-10.1 Description

A pilot-scale test study was operated at Fayetteville, AR to determine if the A/O process could achieve desired operational performance under design conditions. The pilot test was a one gpm pilot plant sized to allow the same retention time as the full-scale plant, thereby simulating the full-scale process. In the A/O process, microorganisms solubilize phosphorus in the absence of oxygen in the anaerobic cells. In the oxic cells, soluble phosphorus uptake occurs; organic matter is converted to cell matter, carbon dioxide, and water; and ammonia is oxidized to nitrite and nitrate. The process is shown in Figure II-10.1.

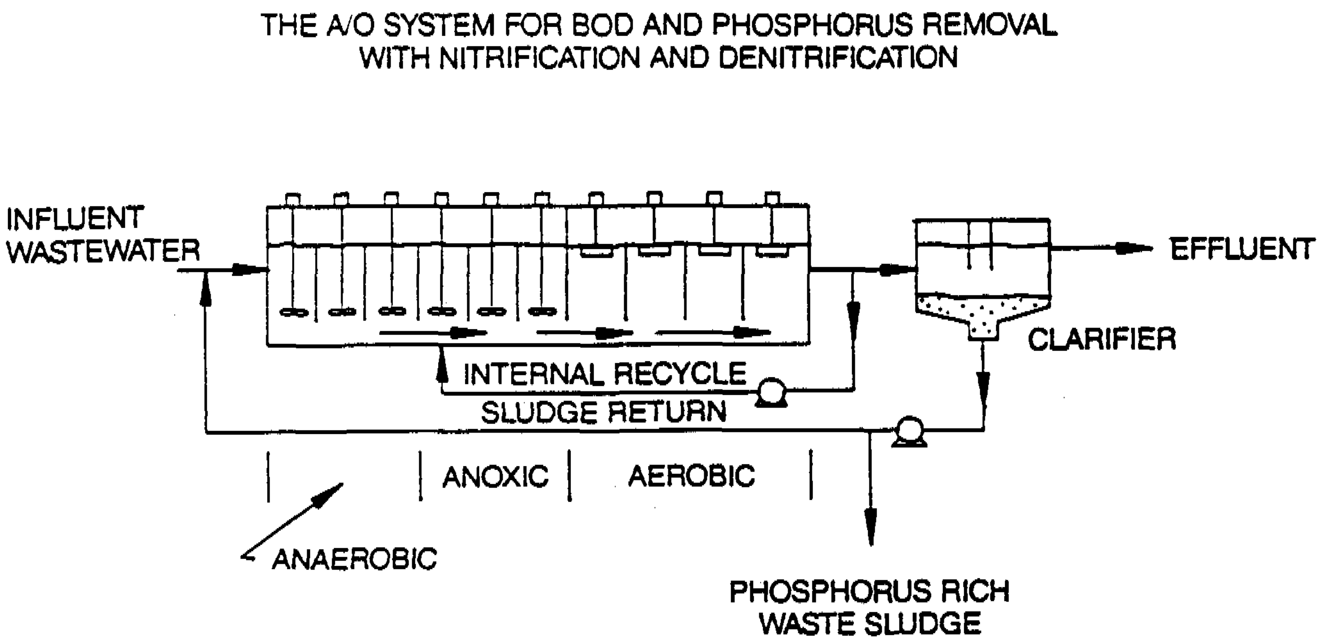
II-10.2 Applicaton

The A/O process is applicable to wastewater systems that have a phosphorus and/or nitrogen discharge limit.

II-10.3 Benefit

The A/O process can save costs because oversized clarifiers are not required for phosphorus removal, separate nitrification and denitrification basins are not required for ammonia removal, and chemical storage/handling facilities are not required. Since the only chemicals required are relatively small amounts of alum, operating and maintenance costs are reduced. Stringent effluent limits for BOD, suspended solids, ammonia, and phosphorus reduction can be met with relatively simple operating controls. The A/O process substantially reduces sludge volumes when compared to conventional systems.

II-10.4 Findings



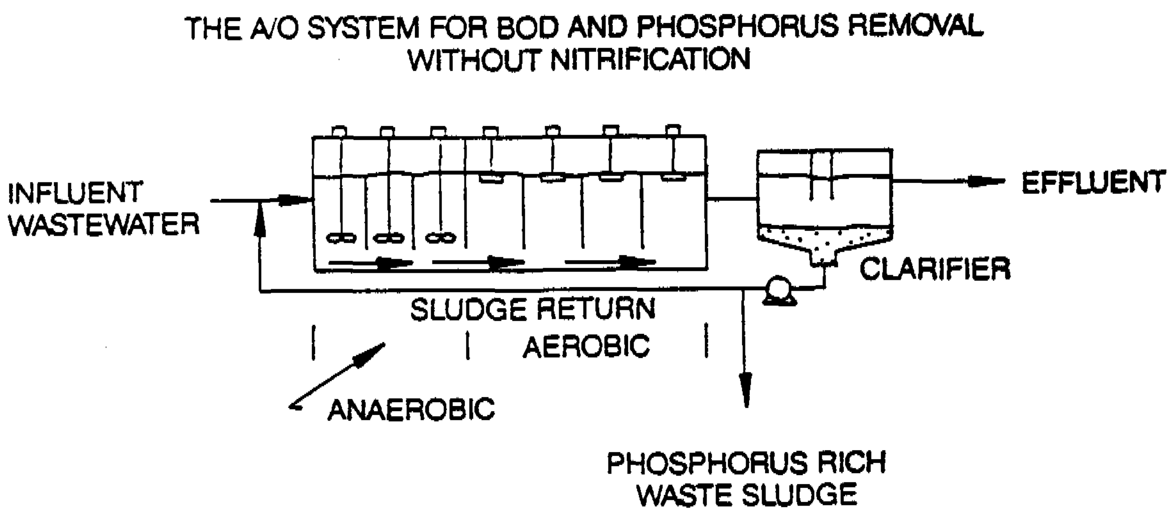


Figure II-10.1: Anaerobic Biological Nutrient Removal; Fayetteville, AR.

The pilot plant generally achieved excellent BOD, suspended solids, ammonia, and phosphorus removal. Effluent concentrations of BOD, ammonia, and suspended solids were consistently at or below permit limits. Alum had to be added to the oxic basin effluent during low flows to reach the 1 mg/l phosphorus effluent limit. Without alum addition, effluent phosphorus ranged from 0.5 to 3.1 mg/L. The field test demonstrated that the full-scale facility will be capable of meeting effluent limits.

II-11 Sequencing Batch Reactors (SBRs)

II-11.1 Process Description

In the SBR process, all of the treatment steps occur in one tank as depicted in Figure II-11.1. The tank is first filled with raw primary wastewater and then aerated to convert the organics into microbial mass, thereby treating the wastewater. After treatment, the aerators are turned off, allowing the solids to settle. During this idle period, clarifier effluent is withdrawn and solids are wasted. The SBR process is then ready to begin again. The dynamics of this process follow batch rather than continuous flow kinetics. The process is consequently easier to operate and degradation of organics is probably more complete following each cycle than is possible in a flow-through system.

II-11.2 Application

SBRs are well suited for small communities which require wastewater treatment systems that are economical to build, simple to operate and maintain, and reliable in meeting secondary effluent quality limitations, or better.

II-11.3 Benefit

SBR systems require less land area and operator attention than conventional activated sludge treatment systems. Biological treatment and clarification are conducted in one basin, thereby eliminating secondary clarifiers and the associated piping and mechanical systems.

II-11.4 Status

Full-scale SBR systems are operational in Culver, IN and Poolesville, MD. The Poolesville system received a national

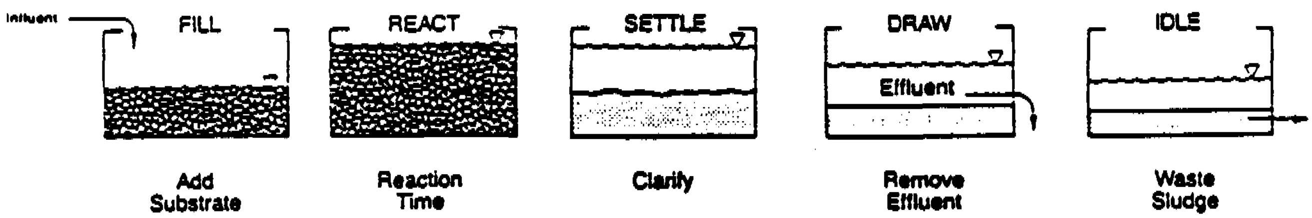


Figure II–11.1: Typical Sequencing Batch Reactor Sequence (One Cycle).

award for design excellence. Recent data suggest that SBRs can produce excellent biochemical oxygen demand and suspended solids removal with minimal energy input. SBRs can also be operated in a mode which will remove substantial nitrogen and phosphorus.

II-12 Intrachannel Clarification (ICC)

II-12.1 Process Description

The ICC concept combines a secondary clarifier with an oxidation ditch. The unique feature of ICC is that wastewater enters the clarifier, effluent is withdrawn from the clarifier, and sludge is returned to the ditch without pumping. Figure II-12.1 shows one type of intrachannel clarifier within an oxidation ditch.

II-12.2 Application

ICC is applicable for use by communities of all sizes seeking to reduce the costs associated with a conventional oxidation ditch process. In designing, it is important to provide adequate mixing and aeration capacity, scum removal systems where flow barriers occur, adequate sludge handling capacity, and adequate structural support for the mixing and aeration systems. In addition, one manufacturer recommends not using an intrachannel clarifier if the peak-to-average flow ratio exceeds 2.5. Table II-12.1 (II-5).

II-12.3 Benefit

The advantages of intrachannel clarifiers can include reduced construction and O&M costs and a reduction in land area requirements. Common wall construction reduces concrete requirements. Hydraulic head differences and gravity are used to force wastewater into the clarifier and return sludge back into the ditch. Pumping requirements are thereby reduced. The need for control over sludge return is eliminated, and sludge age is easily controlled by wasting mixed liquor from the ditch or from the intrachannel clarifier. The operational problems reported may thus be more representative of start-up problems rather than long-term design deficiencies. At several systems, problems have

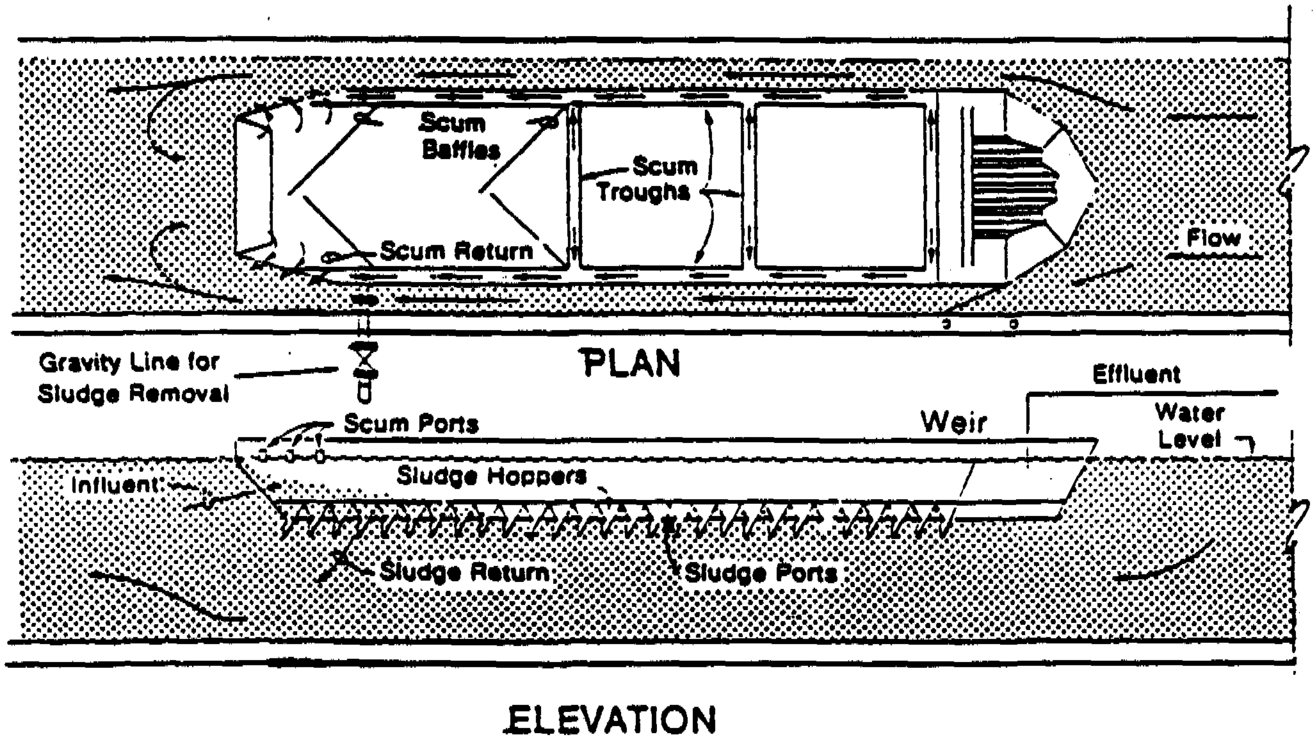


Figure II–12.1: Typical Boat Clarifier*.

*The Boat Clarifier is the registered trademark of United Industries, Inc.

TABLE II-12.1

TYPICAL PILOT PLANT OPERATING CONDITIONS

Sludge Area	10 Days
Overflow Rate	600-800 gallon per day per sq.ft.
Mixed Liquor Concentration	3,000-3,500 mg/l
Aeration	10 hour
Effluent Quality	<20 mg/l BOD_5 <20 mg/l SS

been encountered with obtaining adequate flow velocity in the oxidation ditch. Proper operation of the clarifier is dependent upon adequate wastewater flow velocity around the ditch. Several facilities have reported that inadequate velocity has caused solids settling in the ditch, resulting in sludge bulking and excess scum accumulation. Changes in mixer design or mixing systems have since corrected velocity problems at some facilities. Insufficient aeration has also occurred in several systems. In general, aeration systems which have performed well in conventional oxidation ditch systems provide adequate aeration in intrachannel clarifier systems. (II-4)

II-12.4 Status

Approximately 80 ICC systems are currently in design, construction, or operation in the United States; and seven manufacturers currently market ICC systems. Twelve operational systems are in existence including Morgan City, LA; Sedalia, MO; Owensboro, KY; and Thompson, NY. The current performance data for these systems shows that effluent biochemical oxygen demand and suspended solids concentrations of 20 mg/L can be achieved where adequate mixing is provided.

II-13 Hydrograph controlled Release (HCR) Lagoons

II-13.1 Process Description

There are three principal components of a HCR lagoon: a storage lagoon which receives effluent from the conventional lagoon system, a stream flow monitoring system, and an effluent discharge structure. The effluent discharge structure releases the treated wastewater from the storage lagoon in proportion to the stream flow as measured by the monitoring system. The size of the storage lagoon is determined by the stream flow characteristics. A schematic diagram is presented in Figure II-13.1. The system is basically pollution control by dilution.

II-13.2 Application

The HCR concept is applicable to systems where the receiving stream's assimilative capacity does not permit continuous discharge from a conventional lagoon system. In such cases, the HCR lagoon is used in combination with the conventional lagoon system. HCR lagoons will not be a cost effective alternative to other treatment systems in all cases. Design considerations which must be evaluated include: site availability, receiving stream effluent requirements, receiving stream flow pattern. Due to the relatively large area required for construction of an HCR lagoon, lack of a suitable site near the treatment plant may not permit cost effective construction of the HCR lagoon. Receiving streams which have stringent year round effluent requirements or low flow patterns in comparison to the WWTP flow may not permit the variable discharge characteristic of HCR lagoons to be used effectively. In general, capital costs for HRC lagoons are dependent upon the following factors: storage volume required, pond liner requirements, and land costs. The total storage volume required is related to both the treatment plant flow and the receiving stream flow pattern. If the receiving stream has a relatively high flow in comparison to the plant flow, a storage

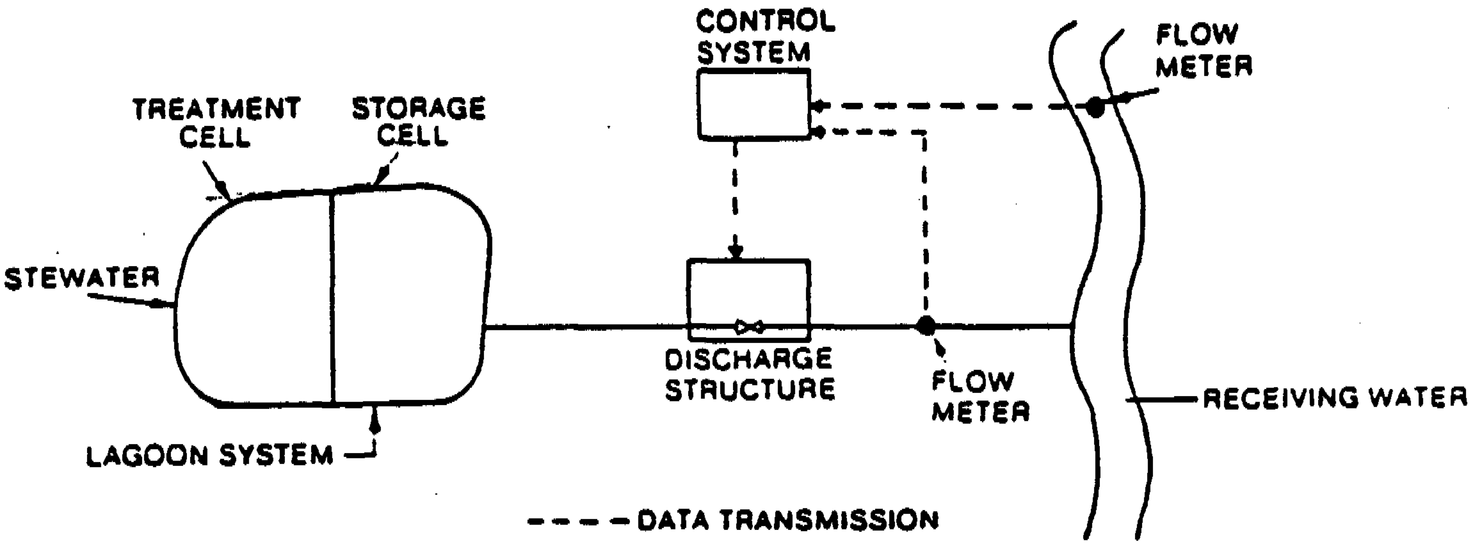


Figure II–13.1: Hydrograph Controlled Release Lagoon Schematic.

volume equal to 30 days of the average plant flow may be adequate. Conversely, a relatively low stream to plant flow ratio may require a storage capacity in excess of 120 days.

The pond liner requirements are site specific, and depend upon the nature of the existing soils, proximity of ground water, and State requirements regarding permissible pond leakage. Typical pond liner materials include clay, plastic liners, and soil additives such as bentonite. In some cases, asphalt or concrete liners may also be used. Clay seals or soil additives are generally less costly than the other liners if clay is readily available or the soils are suitable for use with an additive. Costs range from $1,000,000 to $1,500,000 for 25 and 50 acre (10-20 ha) lagoon size systems, respectively (II-6).

II-13.3 Benefit

An HCR lagoon system can be used to make the maximum use of a stream's assimilative capacity since effluent discharge is hoped to be proportioned to stream flow, thereby allowing the use of low-cost, easy-to-operate lagoon systems where higher levels of treatment might otherwise be required. Planning and manual override may be required to manage forced discharges of the system when it is full. Extended periods of low stream flow will abort the effectiveness of the HCR system.

One of the key design aspects of an HCR lagoon system is the proper sizing of the storage lagoon relative to the flow and water quality characteristics of the receiving stream. Design of the storage cell for an HCR lagoon system is based upon performing a water balance for the storage cell with the first step being the determination of the amount of wastewater which may be discharged as a function of the stream flow. Information included in the water balance includes both water quality and flow characteristics of the receiving stream. A time span between discharge events is then determined and a storage volume calculated (II-7). Four different discharge systems are currently in use: motor operated valves, motor driven sluice gates, floating weirs, and, a series of variable sized pumps.

II-13.4 Status

Over 18 HCR systems are currently in design, construction, or operation, primarily in the Southeastern United States. There have been no major operational problems related to the HCR components. Examples of operational systems are Linden, AL; Heidelberg and Canton, MS; and West Monroe, LA.

II-14 Vacuum Assisted Sludge Dewatering Beds (VASDB)

II-14.1 Process Description

In a VASDB system, the sludge is first chemically conditioned and then distributed onto porous media plates. After an initial gravity drying phase, a vacuum is created beneath the beds, thereby drawing off additional water. After the sludge begins to crack, the sludge is allowed to air dry before being removed. A cross-section of a typical VASDB is shown in Figure II-14.1.

II-14.2 Application

VASDB systems can dewater municipal and industrial sludges unless they are highly viscous or contain high concentrations of grease or fine solids. The process may be effective on more granular chemical sludges, such as from water treatment plants. No examples of the latter were found. See Table II-14.1 for design criteria (II-10, II-11).

II-14.3 Benefit

VASDBs may reduce the area required for drying beds by as much as 90 percent compared with conventional drying beds. Cycle times for dewatering are also less, thereby reducing the effects of weather on sludge drying and increasing treatment capacity in available spaces.

Costs were developed to compare covered and uncovered VASDB systems to uncovered, roofed, and totally enclosed sand drying beds for a wastewater treatment plant generating 2000 lb/day (910 kg/day) of aerobically digested sludge solids. It is important to note that a VASDB system is normally designed to yield only a liftable dewatered sludge cake in contrast to the sand drying bed which yields a much drier sludge cake.

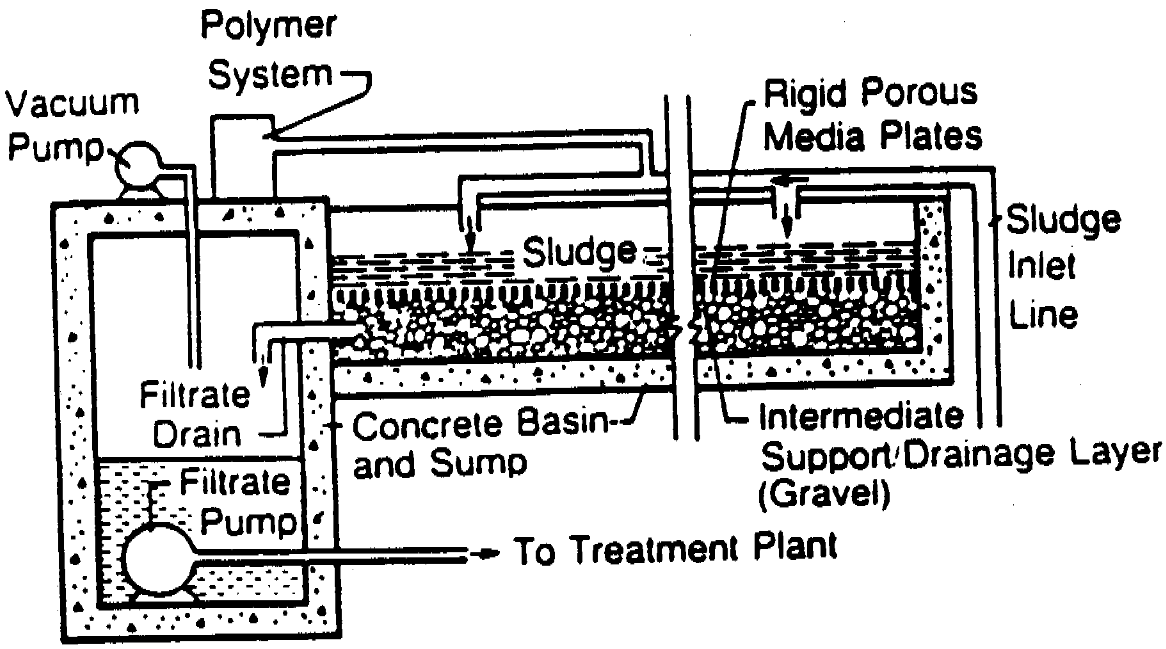


Figure II–14.1: Vacuum Assisted Sludge Dewatering Bed Cross Section.

TABLE II - 14.1

DESIGN CONSIDERATIONS FOR VASDB

BED GEOMETRY AND SIZE	Standard sizes are 20' X 20' (6.1 x 6.1 m) or 20' X 40' (6.1 x 12.2 m); media plates are generally 2' X 2' (0.61 x 0.61 m) or 2' X 4' (0.61 x 1.2 m). The number of VASDBs per facility range from one to four depending on operation flexibility and schedule desired.
BED LOADING FACTORS	Manufacturers suggest 1-2 lb dry solids/ft^2 (4.9 - 9.8 kg/m^2) per cycle for digester sludge; less for waste activated sludge, and higher for lmhoff tank sludge or lime stabilized sludge. Facilities have have been able to increase loading by decanting clear supernatant from sludge bed, then adding more sludge.
FEED SLUDGE TANK	Since optimum polymer dosage is a critical parameter to sludge dewatering, it is suggested that a feed sludge tank be included in designs where polymer mixing and addition are provided.
POLYMER MAKE UP & FEED SYSTEM	A polymer feed pump which can be adjusted during operation is necessary since sludge characteristics may change during bed loading.
POLYMER-SLUDGE MIXING	Mixing is provided in some systems by air injection, residence time, or a series of 90 degree or 180 degree elbows.
SLUDGE PUMPING & DISTRIBUTION	A uniform sludge loading on the plates must be maintained, otherwise premature loss of vacuum may result. It was suggested that bed flooding at the start or use of dual discharge headers may improve uniformity.

Figure II-14.2 presents sand drying bed estimated total system costs as a function of sludge solids loading rates. Also entered on the Figure are the estimated total system costs for equivalent capacity uncovered and covered VASDB systems. The intersections in Figure II-14.2 indicate the following (II-10, II-11):

- An uncovered VASDB system would be more cost effective than an uncovered sand drying bed at loading rates of less than 16-17 $lb/ft^2/yr$ (78.4 - 83.3 $kg/m^2.yr$).

- A covered VASDB system would be more cost effective than a covered sand drying bed at loading rates of less than 38-39 lb/ft^2yr 4 (186 - 191 $kg/m^2.yr$).

- A covered VASDB system would be more cost effective than a totally enclosed sand drying bed at loading rates of less than 62-63 $lb/ft^2/yr$ (304 - 309 $kg/m^2.yr$).

II-14.4 Status

Treatment systems utilizing VASDBs include Portage, IN; Sunrise City, FL; Lumberton, NC; and Grand Junction, CO. Data from operational systems indicate that solids concentrations of 8 to 23 percent can be produced with cycle times ranging from 8 to 48 hours.

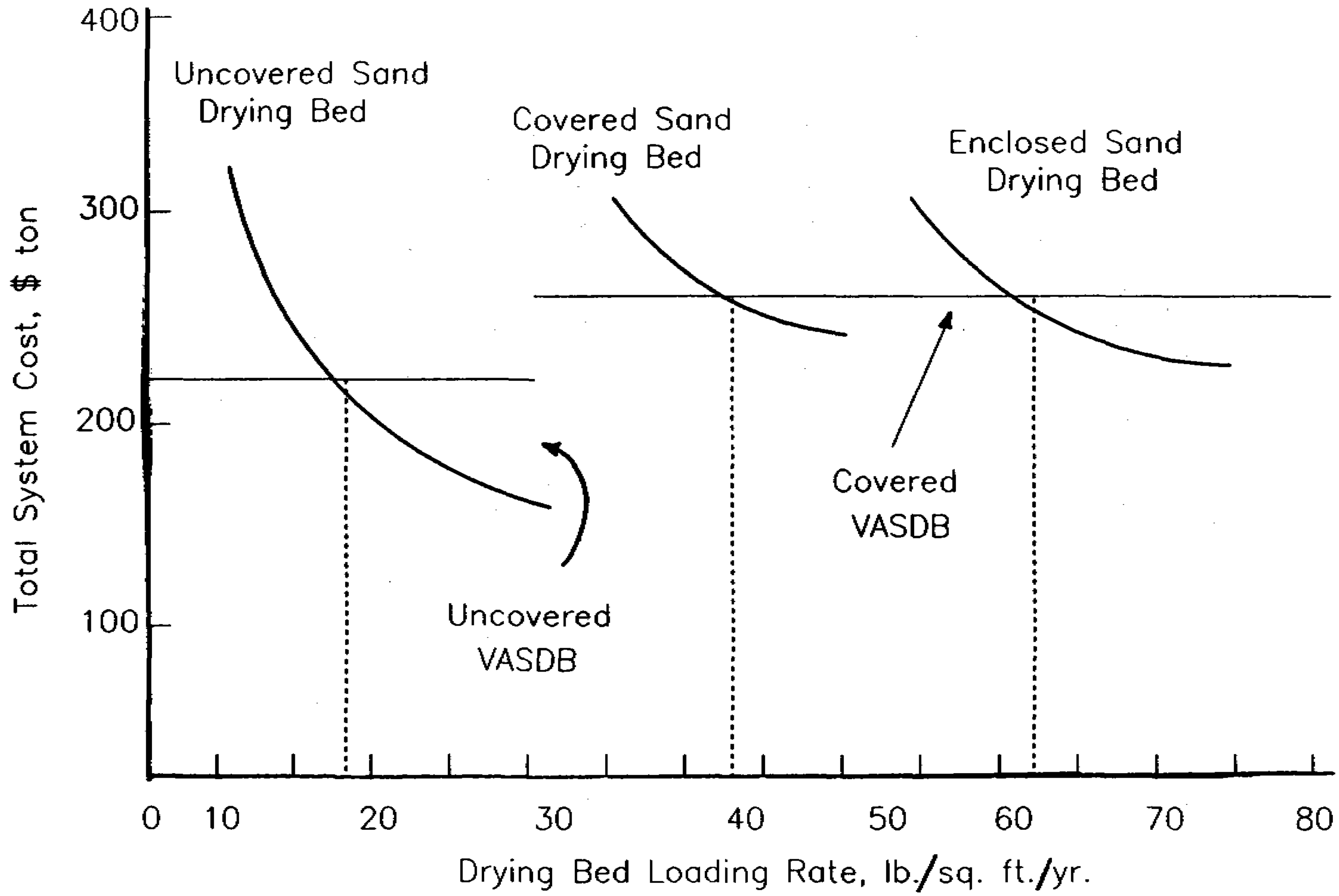


Figure II–14.2: Estimated Sand Drying Bed and VASDB System Total Costs as a Function of Solids Loading Rate for Systems Processing 365 Tons of Sludge Solids/Year.

II-15 Counter-Current Aeration (CCA) Systems

II-15.1 Process Description

In CCA, the aeration system moves with respect to the solids, unlike conventional systems where the aeration system is stationary. In one of the six configurations of a CCA system, shown in Figure II-15.1, the aeration system rotates around a circular tank about once per minute. The rotation creates a longer bubble flow path which may result in a greater oxygen transfer.

II-15.2 Application

CCA systems can be cost-competitive for plant sizes over 0.15 MGD.

II-15.3 Benefit

CCA may reduce the land area and energy requirements for extended aeration systems. Oxygen transfer efficiency may also be higher with CCA systems than with other aeration systems..

Capital costs are similar to activates sludge system aeration. Operation and maintenance costs will be similar to other extended aeration processes with the exception of power. Table II-15.1 shows the dramatic energy savings that can be achieved by using the countercurrent aeration process (which also uses the fine bubble diffusers) (II.12).

LIMITATIONS:

- o Generally not cost-competitive for plant sizes under 150,000 gpd.

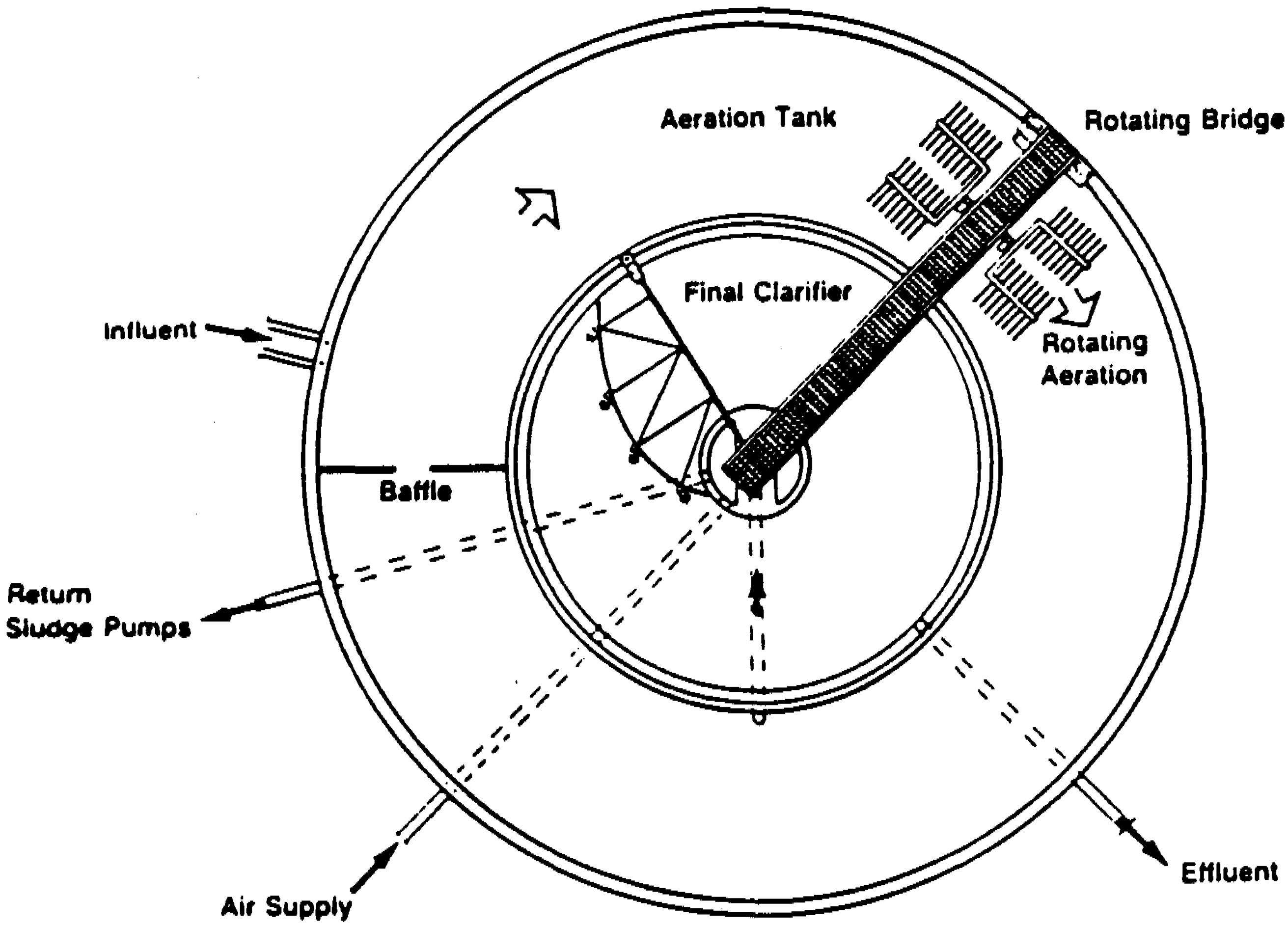


Figure II–15.1: Counter–Current Aeration System.

TABLE II-15.1

POWER USE COMPARISON

	Oxygen Transfer* lb O_2/hp/hr	Power Use hp/MGD
Extended Aeration with Coarse Bubble Diffusers	3.0	83
Oxidation Ditch	3.4	73
Mechanical Aerators	3.6	69
Extended Aeration with Fine Bubble Diffusers	5.0	49
Countercurrent Aeration	6.0	41

* with clean water

OBSERVED ADVANTAGES

- Significant power savings over other activated sludge processes.

- Requires less land than some other extended aeration processes (e.g., oxidation ditch).

PROCESS CONSIDERATIONS

- Concentric clarifier/aeration tanks possible up to 1.25 mgd.

- Careful tank construction and rotating equipment placement required.

- Standard design may require some additions or modifications for maximum operation flexibility and safety.

II-15.4 Status

CCA systems are currently in design, construction, or operation at over 20 locations in the United States. Over 500 systems are operational worldwide. Operational systems in the United States include Grand Island, NY; Loudon, TN; Rome and Clayton County, GA; and Tuskegee, AL. Operational data from these and other operating facilities demonstrate the energy savings in operating these systems.

II-16 Vacuum Collection System, Cedar Rocks, West Virginia

II-16.1 Description

A vacuum collection system consists of a special vacuum valve which allows a mixture of air and wastewater to enter the vacuum system from each residence. The vacuum valve opens automatically when wastewater accumulates in the storage reservoir below the valve, and remains open for a preset interval to allow the wastewater and air to enter the vacuum system. The air/wastewater mixture is drawn towards the collection station by pressure differentials between the vacuum valves and a vacuum pump station which maintains the vacuum throughout the system. Figure II-16.1 shows a schematic diagram of a vacuum sewer system.

II-16.2 Application

The system consists of three main trunks which are controlled separately from the vacuum station to allow isolation of problems or installation of a new service without disruption of the other branches. Two hundred vacuum valves were installed in the Cedar Rocks system, with one valve serving two homes in some cases. The collection station operates an average of 4 1/2 hours per day. A vacuum is applied to the collection system by a vacuum pump through a fiberglass collection tank. An 800 gallon vacuum reserve is also used for moisture collection. A collection tank receives the wastewater from the three mains. Sewage collected from the Cedar Rocks area is then discharged to the Wheeling, West Virginia, wastewater collection system.

The advantages and disadvantages of vacuum sewers are:

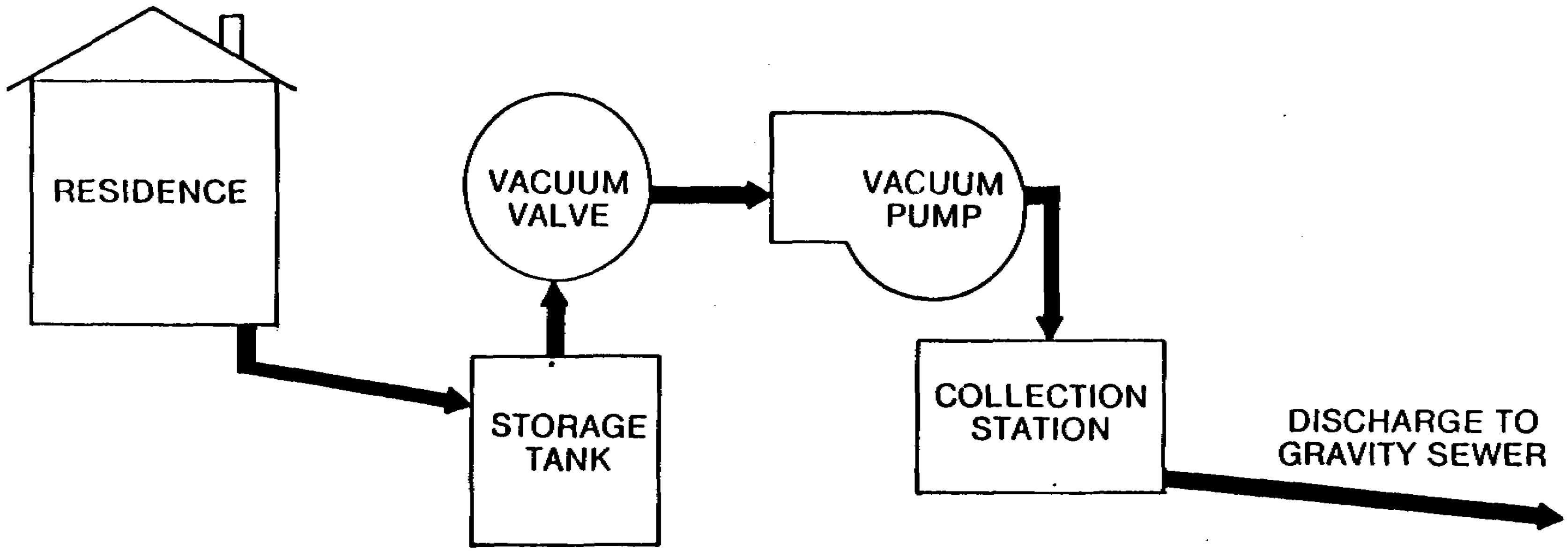


Figure II–16.1: Vacuum Sewer System Schematic Diagram.

Advantages	Disadvantages
Low construction cost	Necessary to maintain vacuum
Reduced infiltration/ exfiltration	High operation/maintenance requirements
Shallow sewer depths	Malfunctions difficult to locate
Cleanouts used instead of manholes	Not adaptable to hilly terrains
Reduced water use when vacuum toilets utilized	

II-16.3 Status

A gravity collection system was proposed for Cedar Rocks, West Virginia, in the original wastewater facilities plan for the area. The Cedar Rocks vacuum collection system began serving 250 users in December 1984. Although some problems were encountered during the construction phase, they were readily solved, and the system has been operating satisfactorily since start-up.

II-17 Wetlands/Marsh System, Cannon Beach, Oregon

II-17.1 Description

The three lagoons and chlorination facilities were modified to include the addition of an aeration basin and a new chlorine contact chamber. A portion of the adjoining forested wetlands is used to polish the secondary effluent before discharge. The wetlands/marsh system was designed to serve approximately 7,000 people. The system operates from June 1 to October 31, with all of the treatment plant effluent going into the marsh. The wetland/marsh system is not used during the other months because increased flows during the winter rainy season provide sufficient dilution in Ecola Creek. The marsh system covers 16 acres and consists of two 8-acre cells used in series. The average depth is two feet. Winter flooding structures allow periodic flushing of the marsh. The site plan is shown in Figure II-17.1.

II-17.2 Application

The Cannon Beach, Oregon, stabilization pond treatment system could not meet the stringent effluent discharge requirements of 10 milligrams per liter (mg/L) suspended solids (SS) and biochemical oxygen demand (BOD). Higher flows in the summer, resulting from a tripling of the summer population, caused the noncompliance. To solve the problem, the city selected an artificial marsh and aquaculture system to expand the existing wastewater treatment system. However, because the selected site was a wooded wetland, the plan was altered to employ a natural wetlands/marsh in the treatment system. The primary objective of the project was to meet the discharge requirements. Secondary objectives were to minimize disturbance to existing wetland habitat and allow continuing usage of the site by wildlife.

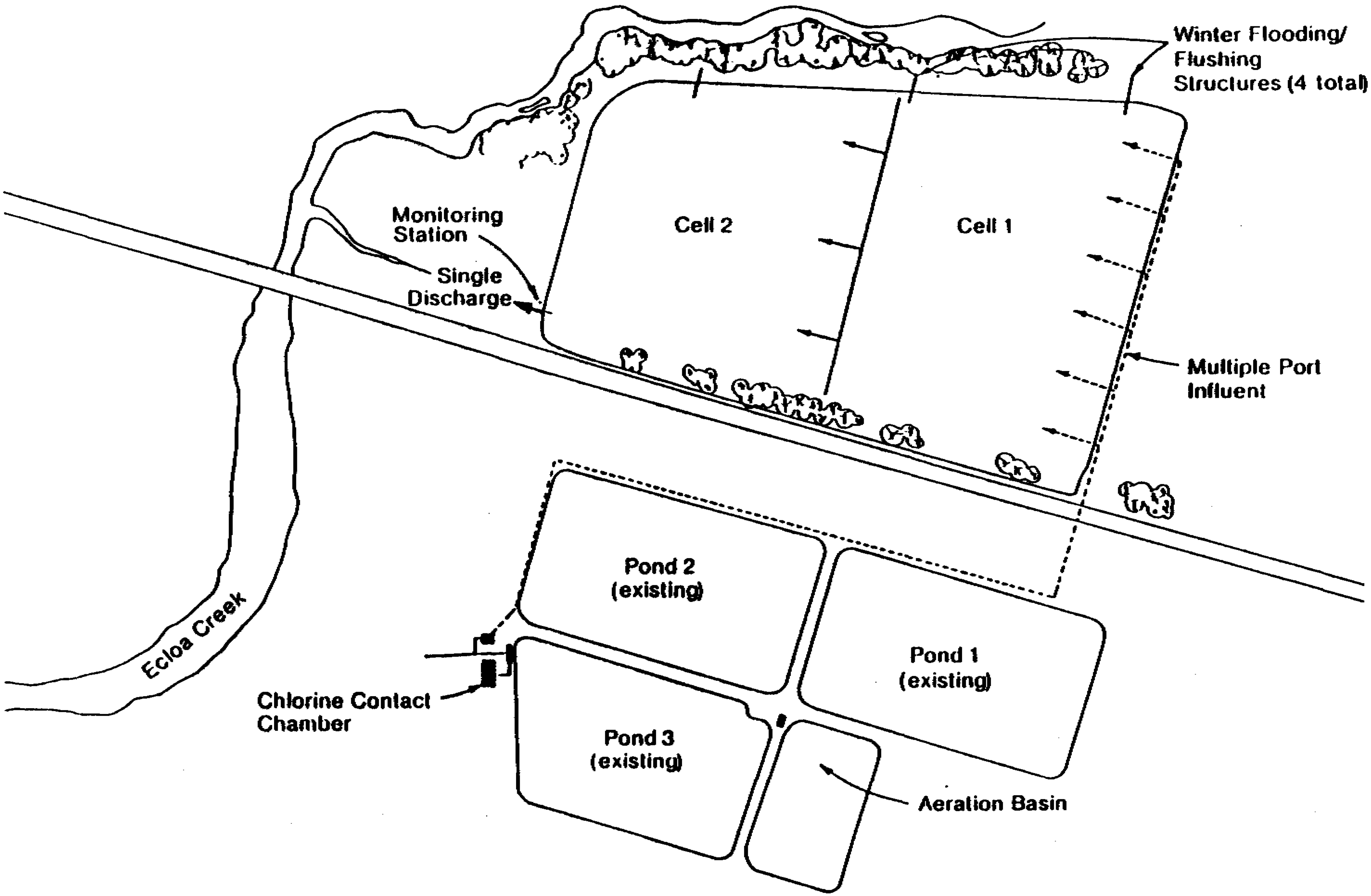


Figure II–17.1: Cannon Beach Wetlands/Marsh Treatment System.

II-17.3 Findings

Operating data available for 1985 proved that effluent discharge limits can consistently be met. Average BOD in the influent to the marsh was 12.5 mg/L, while the average BOD in the effluent from the marsh was 4.1 mg/L. This represents an average BOD removal efficiency of approximately 70 percent. The average suspended solids concentration in the influent to the marsh was 41 mg/L, while the average in the influent from the marsh was 9 mg/l. This represents a suspended solids removal of approximately 80 percent.

II-18 Spray Irrigation and Wastewater Recycling System, Clayton County, Georgia

II-18.1 Description

In 1974, the county began a planning process that evolved into a unique system for recycling the county's wastewater into its water supply system. Figure II-18.1 presents the flow diagram for the system. The major component of the system is a 19.5 million gallons per day (MGD) (0.86 m^3/sec) spray irrigation system. The irrigation system is located in the headwaters of Pates Creek, which is the backbone of the county's water supply system. Effluent from the Flint River and the R.L. Jackson activated sludge treatment facilities are pumped to a 12-day storage pond at the spray irrigation site. Three 15,000 gallons per minute (950 l/s) pumps then distribute the wastewater through 18,300 sprinklers onto the 2,400-acre (971 ha) site. The irrigation site, which is planted in pine trees, is divided into seven cells. Each cell is irrigated one day per week for 12 hours at a hydraulic loading rate of 2.5in./wk (6.35 cm/wk). The site is located approximately 7.5 miles (4.65 km) upstream of the Clayton County water reservoir. The wastewater applied to the site percolates into the ground water and reappears as streamflow in Pates Creek. At design flows, the wastewater will represent approximately 84 percent of the water flowing into the water supply reservoir during low flow conditions, and approximately 33 percent during normal flow conditions. The second segment of the recycling system is the discharge of 4.0 MGD (0.175 m^3/sec) of advanced treated effluent into Big Cotton Indian Creek. Clayton County operates an auxiliary water intake on Big Cotton Indian Creek that pumps water back into the reservoir. At design flows during low flow conditions, wastewater could represent approximately 62 percent of the flow in Big Cotton Indian Creek at the auxiliary intake.

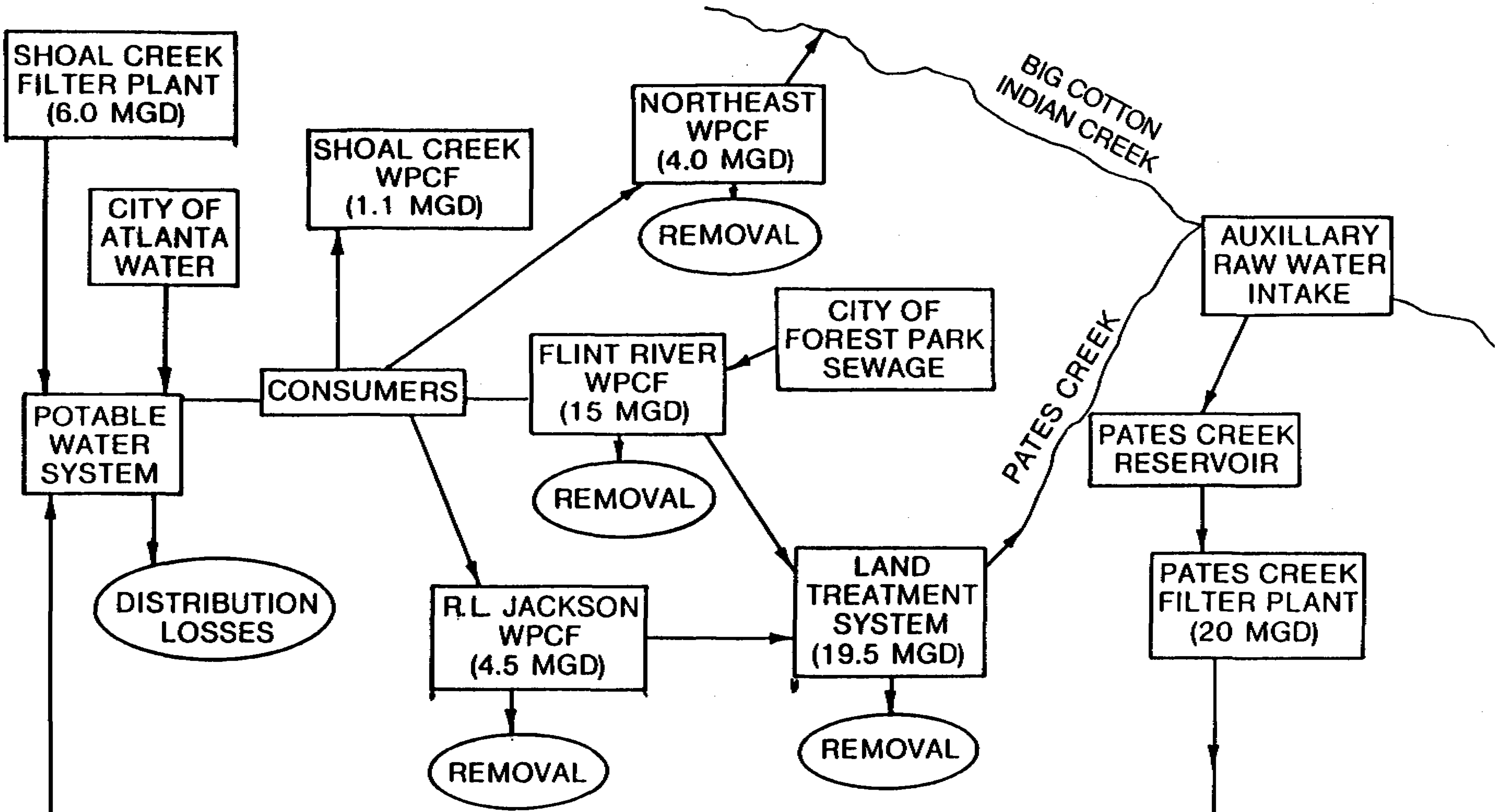


Figure II–18.1: Clayton County, Georgia, Wastewater Recycling System Flow Schematic.

II-18.2 Application

Clayton County, Georgia, is a metro Atlanta county. The topography and geology of the county create unique water supply and wastewater treatment problems. Two ridges divide the county into three drainage basins. Because of this, all streams within the borders of the county are headwaters and are too small to serve as a water supply. Consequently, Clayton County's water supply is located in an adjacent county. In addition, each stream has a limited capacity to assimilate wastewater.

II-18.3 Findings

An extensive monitoring program has provided substantial data on the system. With the exception of chlorides, no change from background levels of all constituents monitored has been detected during five years of operation of the system. Chlorides in the groundwater at the site have increased from 6 milligrams per liter (mg/L) to 15 mg/L, which is far below the threshold limit of 250 mg/L for drinking water.

II-19 Overland Flow (OLF) System, Kenbridge, Virginia

II-19.1 Description

In the OLF process, wastewater is applied at the top of uniformly graded terraces. Renovation of the wastewater occurs as it flows in a thin film over the vegetated soil surface. Typically, 40 to 80 percent of the applied wastewater runs off and is collected in ditches at the bottom of the slope. A schematic diagram of the OLF process is presented in Figure II-19.1. The existing wastewater treatment facilities were incorporated into the design as preapplication treatment. A 15-million gallon (57,000 m^3) pond was added for storage during inclement weather. Effluent from the preapplication treatment system flows to the storage pond and is then pumped to the overland flow terraces. The final design required 22 acres (8.9 ha) of overland flow terraces, with an application rate of 3.5 inches per week (8.9 cm/wk). Fourteen independently controlled overland flow terraces were designed. The wastewater is applied to the terraces by an 8-inch diameter slotted pipe. Figure II-19.2 shows the layout of the overland flow system. The cover crop is a mixture of water tolerant grasses. From January 1986 to June 1986, the system produced an average effluent BOD of approximately 8.5 mg/L and an average SS of approximately 6.1 mg/L. Grass is cut and removed from the terraces, thereby removing solids and nutrients from the system discharge.

II-19.2 Application

Kenbridge, Virginia, upgraded its existing trickling filter wastewater treatment system in an economic and effective manner. The effluent from the existing treatment facility was discharged into Seay Creek, which is a tributary to the water supply reservoir for several communities. The trickling filter system was not capable of meeting the discharge limitations of 28 milligrams per liter (mg/L) biochemical oxygen demand (BOD) and

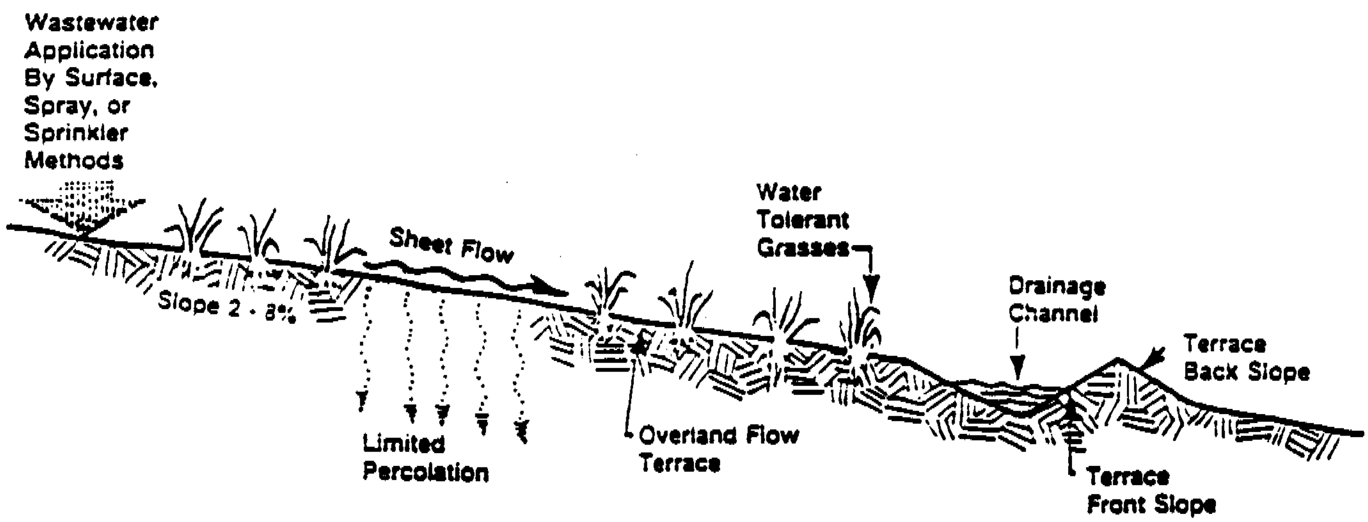


Figure II–19.1: Schematic Diagram of Overland Flow Process.

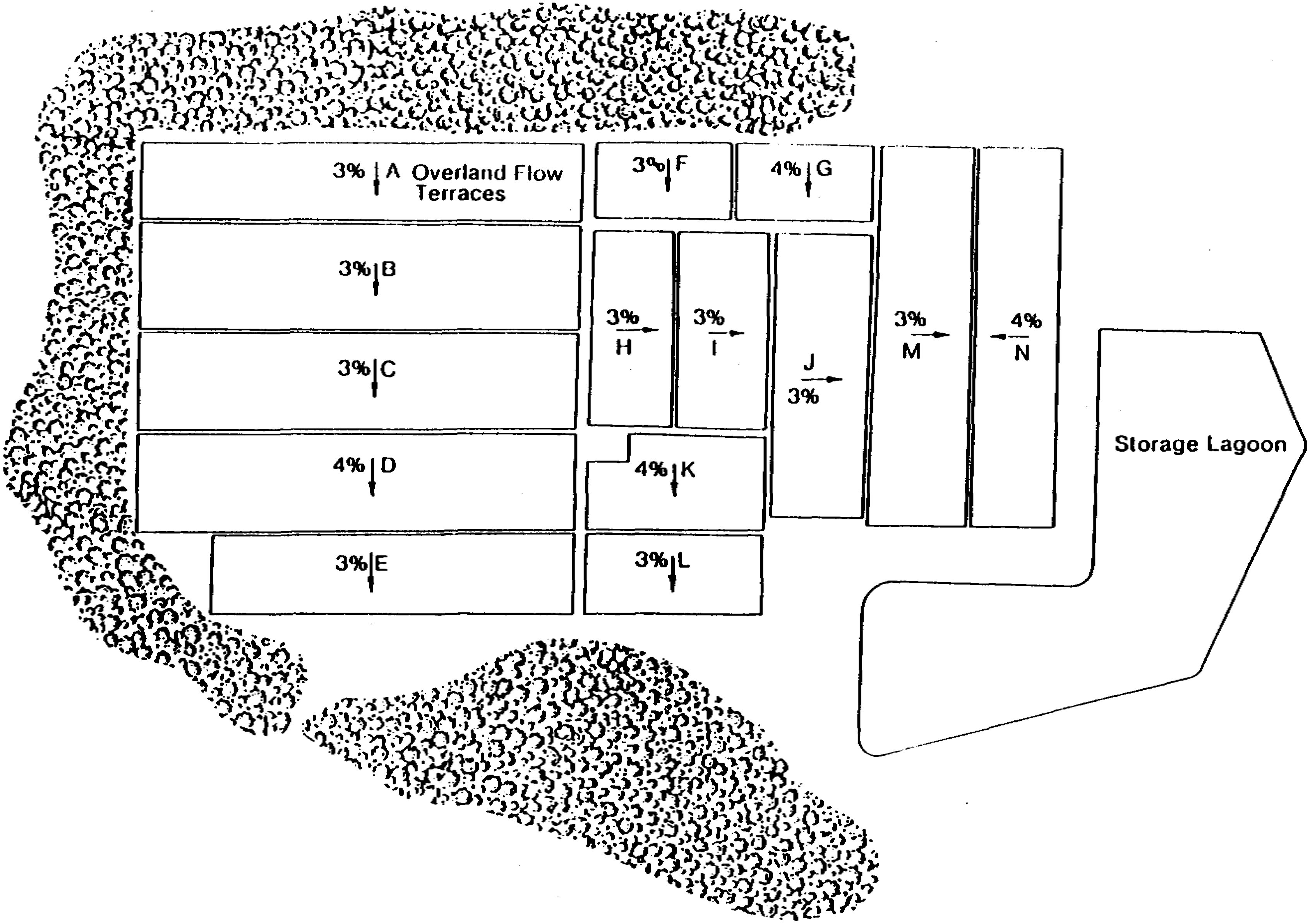


Figure II–19.2: Kenbridge, Virginia, Overland Flow System.

30 mg/L suspended solids (SS) at the design flow of 0.3 million gallons per day (0.0132 m^3/sec). OLF can produce advanced treatment quality effluent by treating screened, primary, or secondary wastewater. Operation and maintenance costs are low, and land and storage volume requirements are less than those for slow rate land treatment. OLF can be used in areas with low permeability soils where land area is somewhat limited and is not prohibitively expensive. A site evaluation of nearby property revealed that an available 100-acre (40.5 ha) tract was well suited for land treatment by overland flow. This form of land treatment can be used in areas with low permeability soils where land area is somewhat limited but not prohibitively expensive. The site was located adjacent to the existing treatment plant in a rural area with little potential for future development. The shallow subsoils at this site had a permeability of less than 1.3 in./hr. (3.3 cm/hr).

II-19.3 Findings

An economic analysis of the overland flow concept compared to an aerated lagoon system showed that the overland flow system would be more cost-effective. The total construction cost for the facility was approximately $1.1 million.

II-19.4 Status

Numerous OLF systems are in operation, including systems in Cleveland, MS; Davis, CA; Kenbridge, VA; and Raiford, FL. Effluent biochemical oxygen demand and suspend solids concentrations of less than 10 mg/L can be achieved. Significant reductions in nitrogen and phosphorus can also be achieved.

II-20 Sludge Composting System, (IVC), East Richland County, South Carolina

II-20.1 Description

East Richland County's variation of the IVC shown in Figure II-20.1 is to cure the sludge in piles on the ground instead of in a closed vessel. The system has been operational since March 1986. Five tons per day of sludge is produced by the extended aeration wastewater treatment process. The sludge is dewatered to approximately 17 percent solids by belt filter presses before entering the compost system. The compost system produces approximately 14 tons (12,6 mt) of compost per day. The county currently has a renewable one-year contract to sell the compost for $12.50 per ton. As illustrated, waste sludge is discharged to a storage bin. The sludge, a carbon source such as wood chips, and recycled compost are mixed together and fed to the bio-reactor. The mixture is held in the bio-reactor for approximately 14 days to allow complete decomposition of the sludge and to destroy disease causing organisms. The compost is then fed to a cure reactor to obtain further solids stabilization and conversion of organic materials to humus. Air is fed into the reactors to maintain an aerobic process. Composting is a thermophilic, aerobic decomposition process whereby complex organic constituents of sewage sludge are broken down microbially into simpler compounds. The composting reaction can be illustrated as follows (II-14):

$$\begin{array}{c}\text{Fresh}\\\text{Organic}\\\text{Waste}\end{array} + O_2 \xrightarrow{\text{Microbial Metabolism}} \begin{array}{c}\text{Stabilized}\\\text{Organic}\\\text{Residue}\end{array} + CO_2 + H_2O + \text{Heat}$$

Heat generated during the process reduces the number of pathogenic microorganisms in the sludge. The stabilized organic residue or end product of the composting process possesses physical and chemical properties which make it useful as a soil

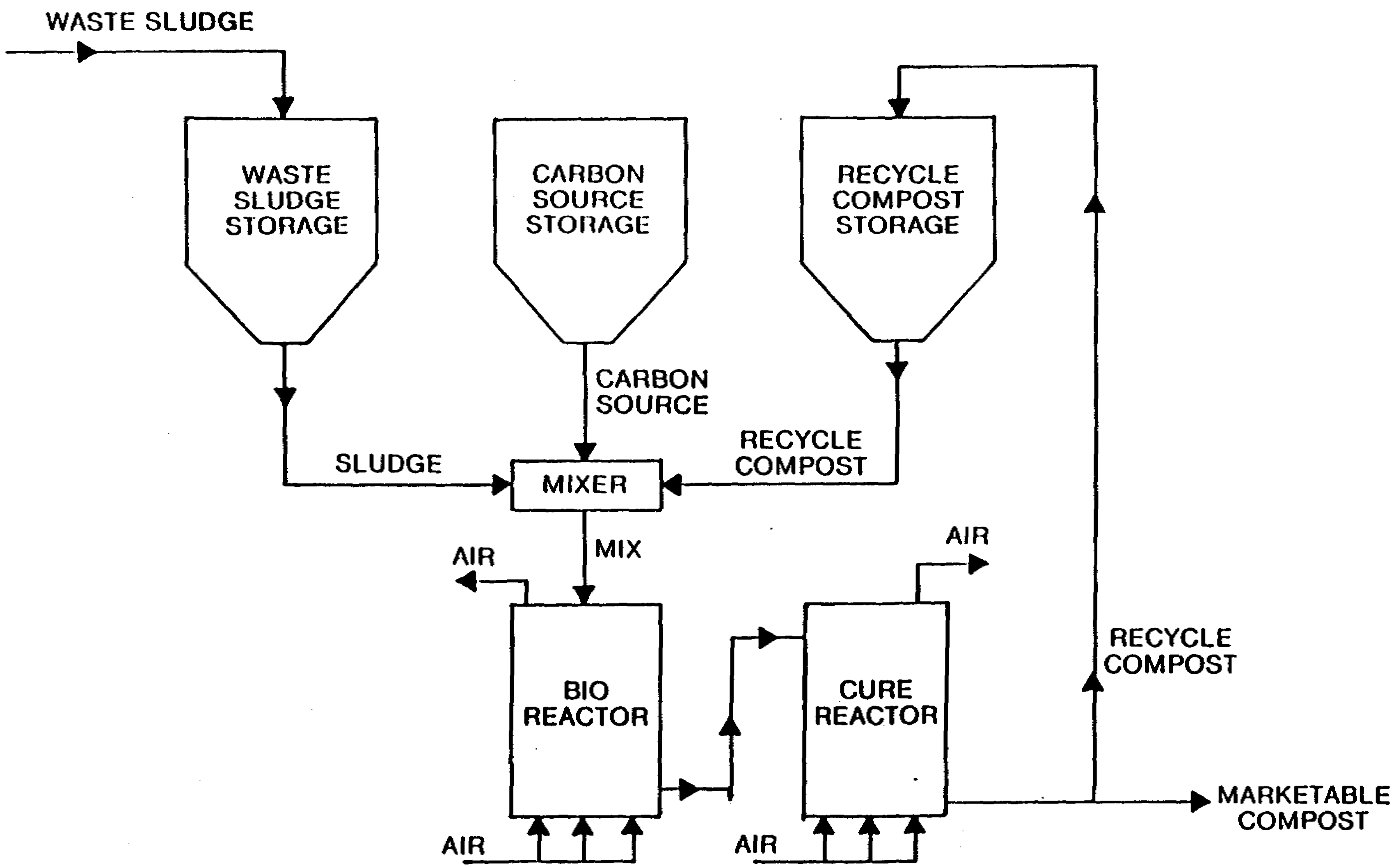


Figure II–20.1: In–Vessel Sludge Composting Schematic.

amendment in landscaping, reforestation, land reclamation and land development projects.

Until recently, composting in the U.S. has been carried out only through non-enclosed windrow, or aerated static pile methods. Problems related to odor control and land area requirements, and the reduced efficiency of these methods during adverse weather conditions have prompted the use of in-vessel, or enclosed, composting (IVC) methods.

A schematic comparison of IVC and non-enclosed composting methods is presented in Figure II-20.2. Relative to non-enclosed methods, IVC provides enhanced odor control, reduced land area requirements, and better operations control during adverse weather. IVC also offers greater potential for recovery and subsequent reuse of the heat generated during the composting process.

Process Controls:

Efficient composting requires sludge with a solids content of 18-30%, a volatile solids content exceeding 50%, a pH of 6-9, and a carbon to nitrogen ratio (C:N) of 25-35:1. The moisture content and pH are often dependent on sludge processing prior to composting.

The C:N ratio and moisture content of sewage sludge are commonly altered through the addition of bulking agents or amendments prior to composting., Bulking agents provide a carbon source to increase the C:N ratio and also increase the solids content and subsequently the porosity of the composting mass. A sludge/bulking agent ratio to provide and infeed mixture in the range of 35-40 tons (31.5 - 36 int) solids is desirable for optimal IVC efficiency. Numerous materials are used as bulking agents, including: wood chips, sawdust, peanut or rice hulls, paper, and shredded automobile tires. Sawdust and wood chips,

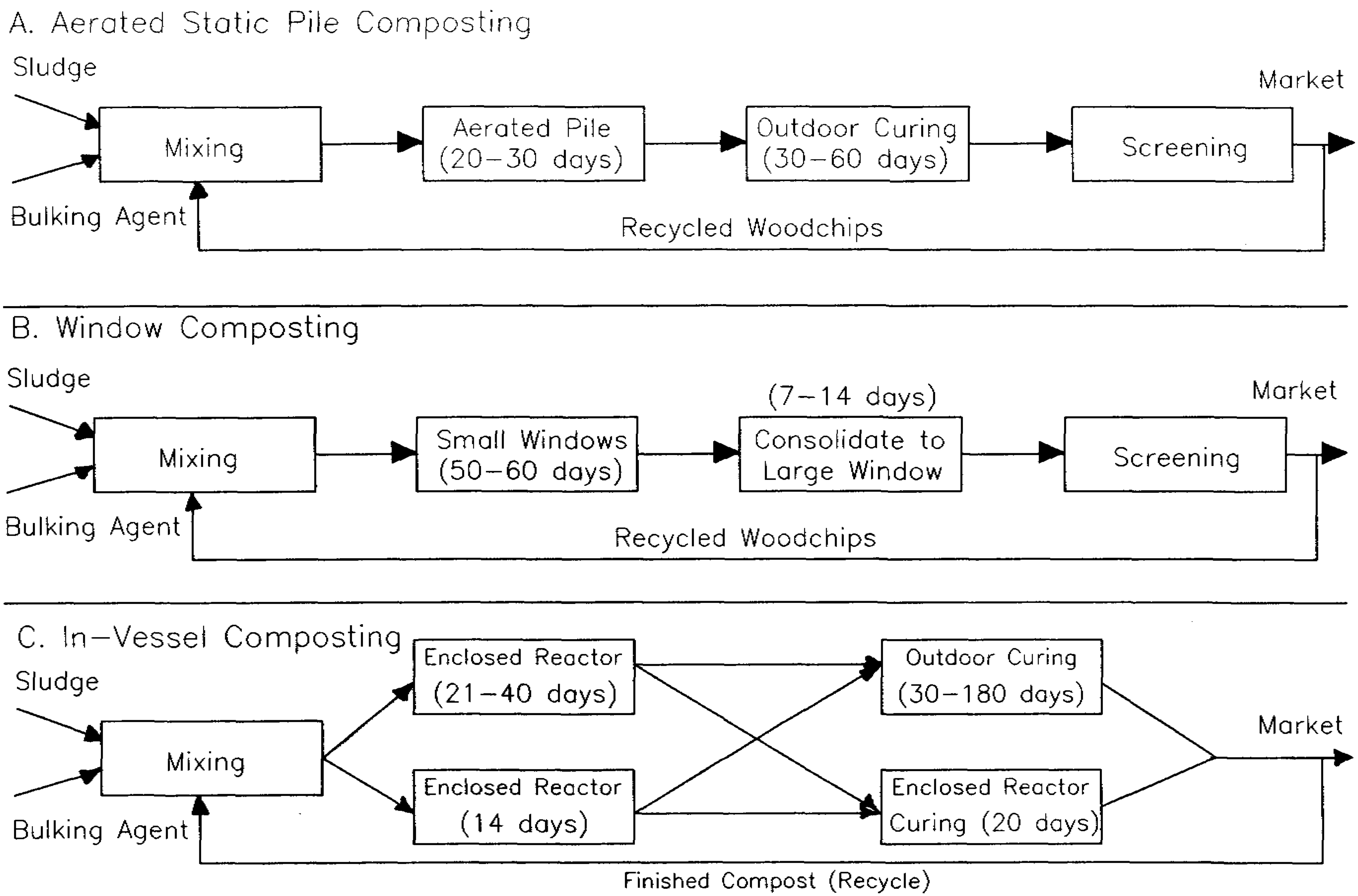


Figure II 20.2: Schematic Representation of Static Pile Window and In–Vessel Composting.

however, are the most widely used bulking agents. The choice of bulking agents is commonly determined by availability and cost. Most IVC facilities have turned to reuse of finished compost (recycle) as a carbon source and the use of a bulking agent strictly for enhancing compost porosity. This has resulted in lower operating costs but has added the need for additional equipment to mix or convey the recycle into the system.

There are numerous types and sizes of IVC systems on the market. These systems incorporate derivations of either agitated (dynamic) or non-agitated (static) reactor types (Figures II-20.3, II-20.4). Reactor sizes for static systems typically have a capacity range from 5 to 60 dry tons/day (4.5 - 54 mt/day); agitated systems are commonly designed for 30 to 90 dry tons/day (27-81 mt/day). Static systems, often referred to as plug flow reactor systems, can either be circular or rectangular. These load from the top of the reactor and discharge from the base, relying on gravity to move the composting mass through the system. Dynamic systems also include both circular and rectangular digester reactors. These systems periodically mix the sludge/amendment mixture in order to maintain uniform heat and air distribution within the composting mass. Movement of the compost through the agitated systems occurs through mechanical manipulation. Dynamic IVC systems use loading, unloading, and aeration devices similar to those of plug flow reactors.

The IVC process can be divided into factors affecting: 1) aeration and moisture removal, 2) odor control, and 3) retention, curing, and discharge.

Aeration and Moisture Removal:

Air serves three purposes in IVC. These include: 1) temperature control, 2) moisture removal, and 3) as a source of oxygen for microbial degradation (compost stabilization). Temperature control and moisture removal are achieved through

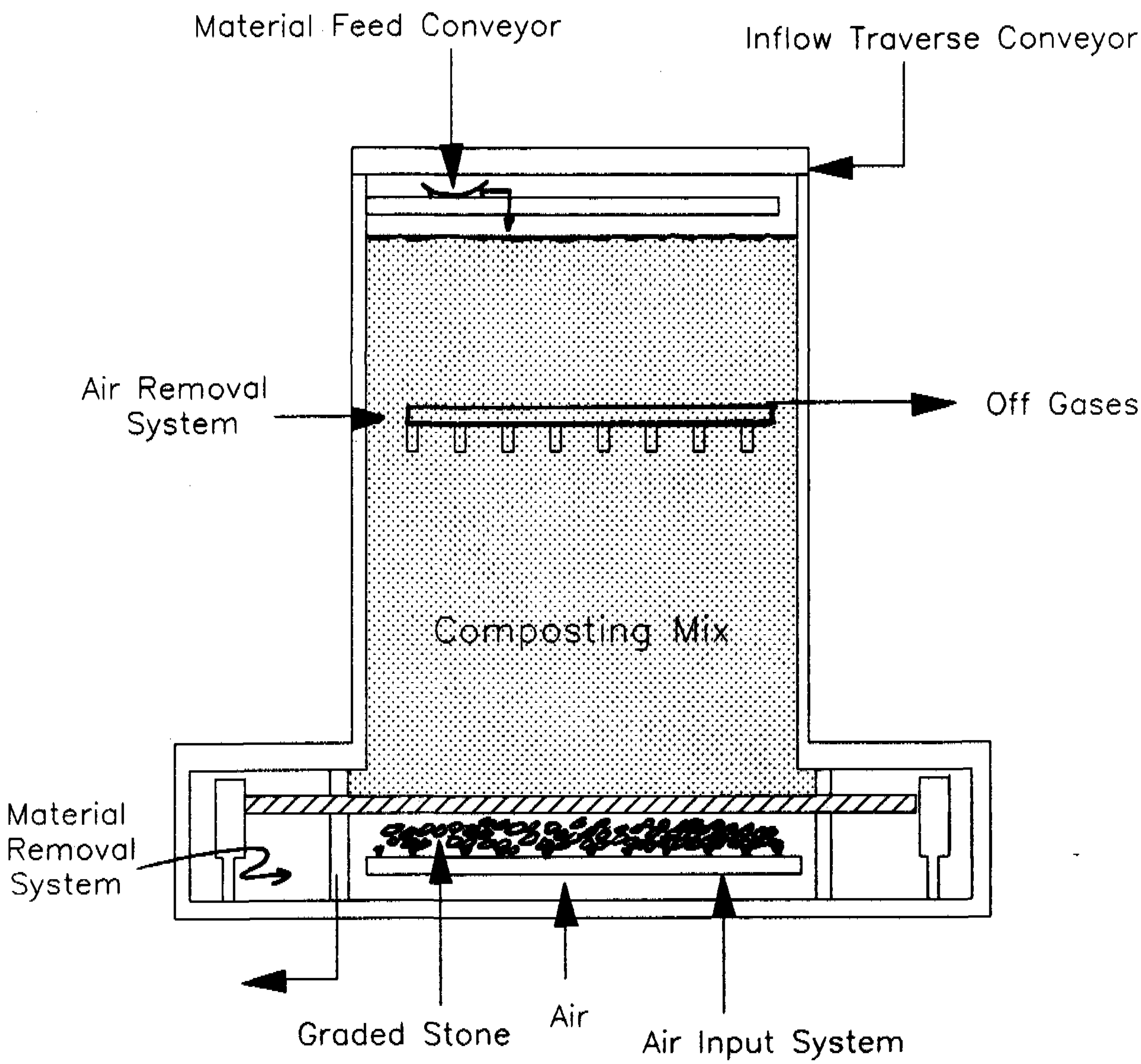


Figure II–20.3: Typical Non–Agitated IVC System.

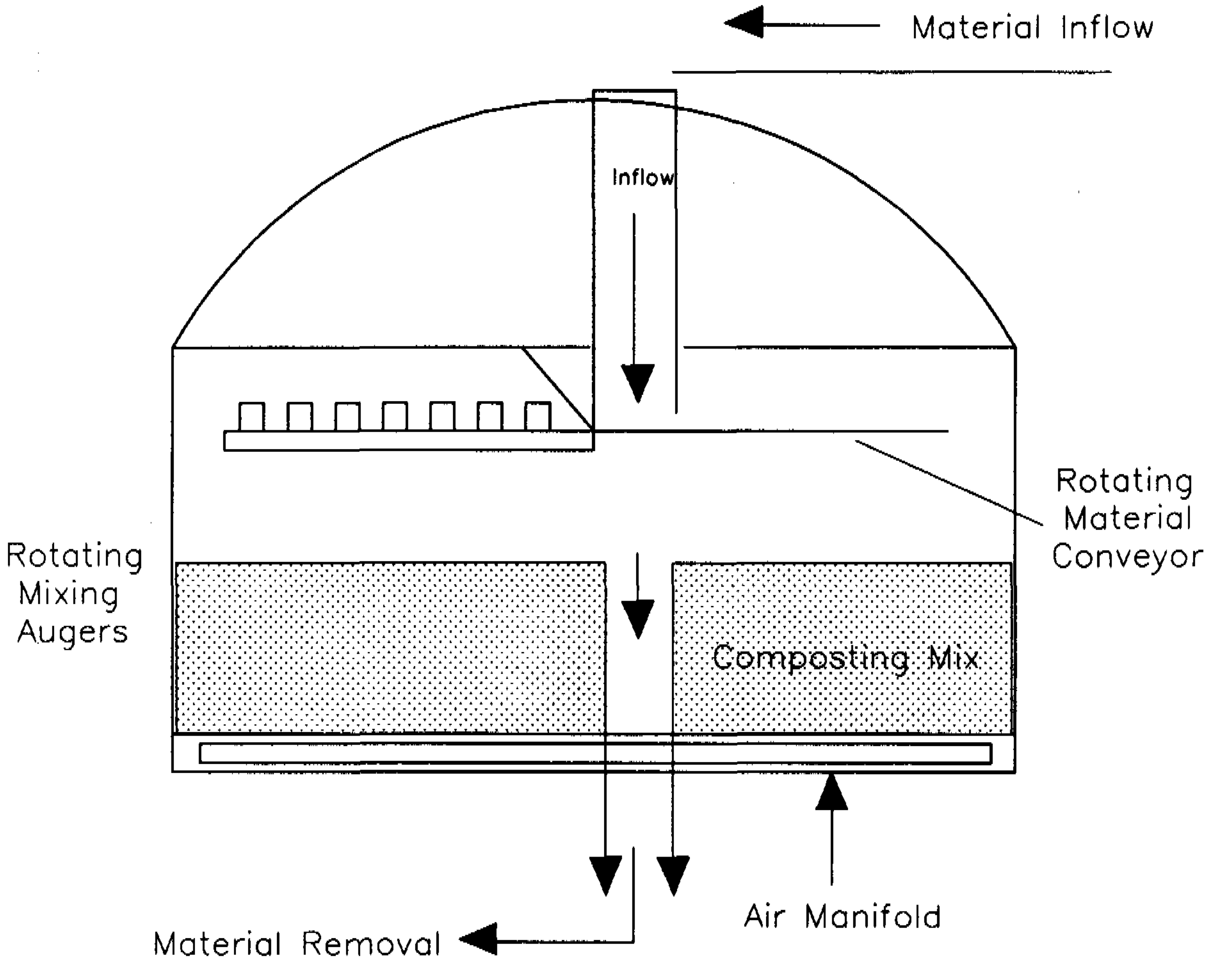


Figure II–20.4: Typical Agitated IVC System.

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regulation of air pressure and speed. Considerations to minimize or collect moisture resulting from condensation within the reactor vessel are considered during IVC design. Inadequate removal of condensate from the reactor vessel may result in the return of moisture to the composting mass, extending the time needed for composting or increasing the amount of energy needed for adequate air flow.

Five to fifteen percent oxygen is required in all zones of the reactor for complete composting. Oxygen levels below or above this range can result in significant pathogen reduction and/or odor control problems (II-13).

II-20.2 Application

Sludge composting is the decomposition of organic constituents to a stable humus-like material. In-vessel composting encases this age-old process in confined vessels. The result is a marketable compost product without the odor and storage problems sometimes associated with other composting systems. Compost is most acceptable for resource recovery when quality is satisfactory. A typical chemical analysis is given in Table II-20.1.

To produce income to help offset production costs, compost is often marketed. Examples of successful compost marketing operations are listed in Table II-20.2. However, many composting facilities rely on "give-away" programs to distribute their finished product.

II-20.3. Status

Initial planning studies to select a sludge treatment alternative for the East Richland County Public Service District wastewater treatment facilities recommended sand drying beds followed by landfilling. However, county officials wanted to

TABLE II - 20.1

TYPICAL CHEMICAL ANALYSIS FOR MUNICIPAL SLUDGE COMPOST FROM MONTGOMERY CO., MD.

CEC (meq/100 g)	40
% Moisture	60
pH	6.0-7.0
Essential Plant Nutrients*	
Calcium	2.7%
Copper	68 ppm
Iron	0.9%
Magnesium	0.2%
Manganese	383 ppm
Total Nitrogen	1.0%
Total Phosphorus	0.3%
Total Potassium	0.02%
Sodium	0.05%
Zinc	203 ppm
Non-Essential Metals*	
Arsenic	0 ppm
Cadmium	2.7 ppm
Lead	180 ppm
Mercury	0.5 ppm
Nickel	0.2 ppm

* average concentration dry weight basis

TABLE II - 20.2

MARKET VALUES FOR COMPOST

LOCATION	BRAND NAME	WHOLESALE PRICE ($/yd³)
Bangor, ME	Bangor Compost	3.75
Montgomery Co., MD	Corn Pro	4.00
Hampton Roads, VA	-	6.00
Durham, NH	-	7.00
Philadelphia, PA	Earth Life (Philorganic)	8.25
Missoula, MT	Eco-Kompost	35.00
Columbus, OH	Corn- Til	9.00
Windsor, Ontario Canada	Growth-Rich	17.00

evaluate a system that would provide resource recovery and revenue generation. A subsequent cost-effectiveness analysis determined an in-vessel composting system similar to the one shown in Figure II-20.1 to be the lowest cost alternative.

II-20.4 Summary

The conversion of present sludge management techniques to IVC has been relatively recent and on a limited basis. As a result, early IVC operations have suffered from problems relating to all aspects of composting. The strengths and weaknesses associated with IVC as compared to other sludge management options are presented in Table II-20.3. These problems, as with other emerging technologies, are expected to be corrected with time (II-13).

TABLE II - 20.3

ADVANTAGES AND DISADVANTAGES OF IVC SYSTEMS OVER CONVENTIONAL SLUDGE MANAGEMENT OPTIONS

ADVANTAGES	DISADVANTAGES
o Elimination of detrimental effects of adverse weather conditions on the composting process	o Substantial capital expenditure necessary
o Improved potential for odor control	o Materials handling and order control systems not specifically designed for sludge applications are often inadequate
o Reduced land area requirements	o Demand for compost is seasonal, this may create the need for compost storage areas and additional handling equipment
o Reduced health risks to employees and residents near composting facilities	o IVC operation may require upgrading of existing dewatering facilities
o Reduced energy consumption and elimination of air quality problems associated with sludge incineration	o Computerized nature of aeration and temperature control systems require technical expertise.
o Production of an ecologically safe and economically beneficial end product	

II-21 Methane Recovery System, Charlotte, Michigan

II-21.1 Description

Figure II-21.1 shows a typical methane gas recovery system. In this example, methane gas generated by the anaerobic sludge digestion process is captured and pumped to a gas storage tank. The gas is then used to fuel engines which generate electricity, and to fuel boilers which heat water and produce steam. The electricity is used to operate other plant equipment. The hot water and steam are used to heat raw sludge entering the digester, and to heat work areas in the treatment plant. Boilers and engines are dual-fuel equipment since a supplemental fuel is necessary. Methane has a net heating value of 970 Btu/cu.ft. (3.6×10^3 kJ/m^3) at standard temperature and pressure. Digester gas has a net heating value of approximately 600 Btu/cu.ft. (2.2×10^3 kJ/m^3) since it is only 65 percent methane.

Utilization options depend on the quantity and quality of digester gas. The characteristics of digester gas from a typical anaerobic digester are shown in Table II-21.1. Gas characteristics from a specific digester depend on the nature of the sludge, the rate at which the sludge is fed to the microorganisms (II-15).

The energy value of the methane generated from the anaerobic digestion process exceeds the energy requirements of the digestion process for mixing and heating the sludge. It is this excess energy in the form of methane gas which can be used to supply other energy needs including:

- o Production of steam or hot water
- o Fuel for internal combustion engines or gas turbines
- o Domestic or industrial gas supply

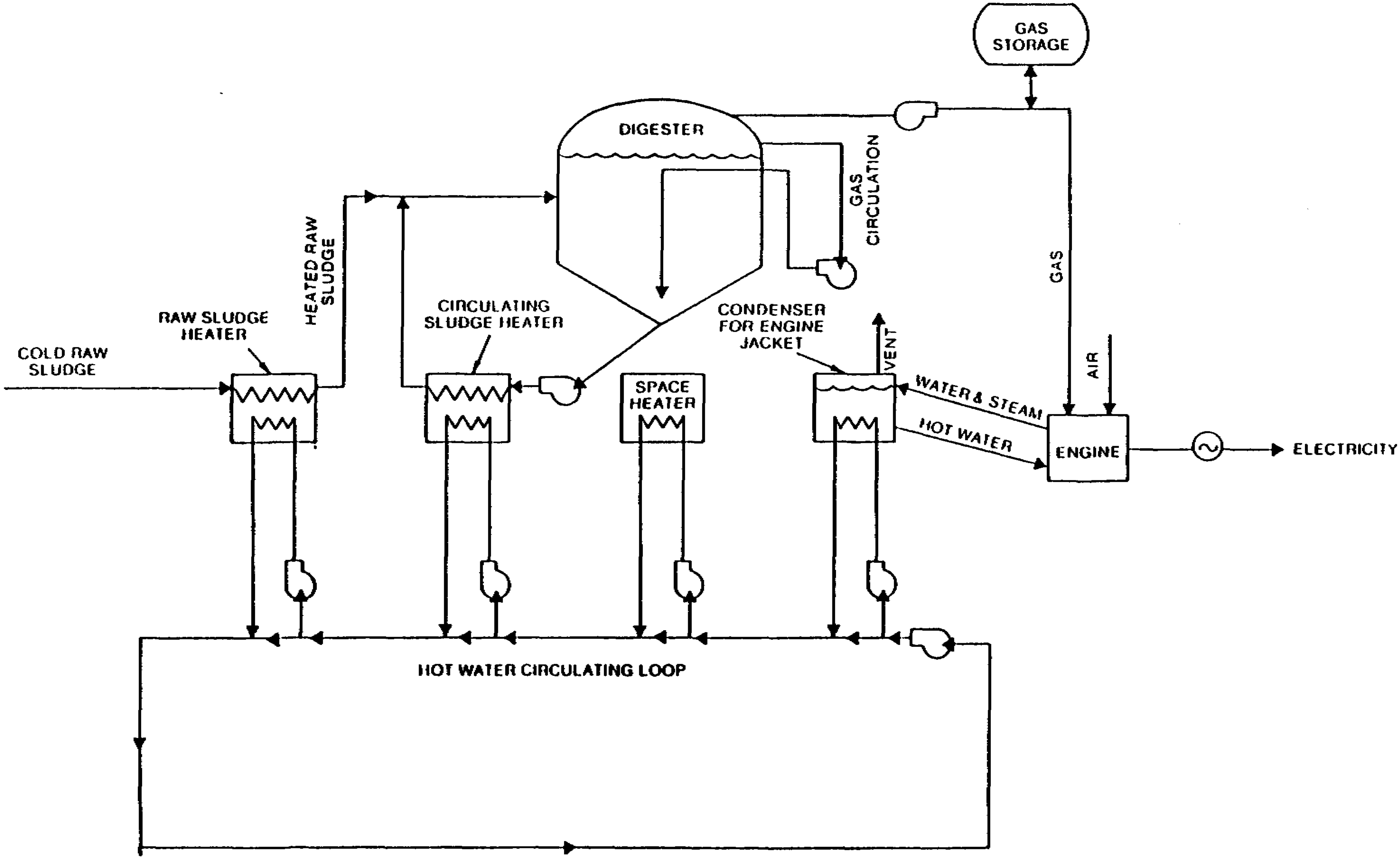

Figure II–21.1: Methane Gas Recovery Schematic.

TABLE II - 21.1

CHARACTERISTICS OF DIGESTER GAS

Digester Gas Quantity

8 to 12 ft^3/lb volatile solids added
12 to 18 ft^3/lb of volatile solids destroyed
0.6 to 1.25 ft^3/capita/day

Digester Gas Quality

Methane (CH_4)	65-70%
Carbon dioxide (CO_2)	25-30%
Nitrogen (N_2)	Trace
Hydrogen (H_2)	Trace
Hydrogen sulfide (H_2S)	Trace
Water vapor	Trace
Heat value, BTU/FT^3	550 to 600

Digester gas may require treatment depending on ultimate use of the gas. It is generally preferable to use recovery systems that can operate on untreated digester gas. Minimal treatment is required for combustion in boilers or internal combustion engines. However, gas sold to local gas utilities requires treatment to upgrade the gas to pipeline quality.

II-21.2 Application

Charlotte, Michigan, city officials selected anaerobic digestion followed by land application to farmland for treatment of the sludge produced by the city's wastewater treatment plant. Methane gas is a natural by-product of the anaerobic sludge digestion process. In order to properly operate the sludge digestion system, raw sludge must be heated which takes energy. City officials decided that use of the methane as an energy source to heat the sludge would increase the efficiency of the treatment system and save operating costs. A recovery system was designed to use the methane for heating of the raw sludge and for fueling an engine to generate electricity.

Possible limitations of methane recovery include:

- Applicability - A variety of sludges from municipal wastewater treatment can be stabilized by anaerobic digestion; however, decreased plant performance may result from additions of some chemical sludges and activated sludges because the additional solids do not readily settle after digestion.

- System Reliability - The microorganisms that generate the methane are sensitive and do not function well under fluctuating operating conditions. The process must be carefully evaluated for use at treatment plants where wide variations in sludge quantity and quality are common. A backup source of fuel is necessary to assure continuity of operation.

- o Gas Characteristics - Impurities in digester gas can cause operational problems, increase maintenance costs, and give rise to air emission problems. Hydrogen sulfide and its combustion by-products can cause corrosion in energy recovery systems if effective treatment of the digester gas is not achieved.

II-21.3 Status

Construction of the Charlotte, Michigan, wastewater treatment plant was completed in September 1980. The plant is designed for an average daily flow of 1.2 million gallons per day (0.05 m^3/sec). A total of approximately 2,500 dry tons (2268 metric tons) per day of sludge digested. This results in an average methane production of approximately 12,000 cu.ft. (340,000 l) per day. A total of approximately 8,700 cu.ft. (250,000 l) per day of methane is used, resulting in an average equivalent cost savings (natural gas) of approximately $18,000 per year.

II-22 Assessment of Dual Digestion

II-22.1 Introduction

The dual digestion system (DDS) is a sludge stabilization process utilizing both aerobic (with pure oxygen) digestion and anaerobic digestion. The major advantages of using DDS over many of the conventional sludge stabilization processes are the increased generation of biogas and the potentially higher degree of pathogen destruction. A study of three facilities utilizing DDS conducted in 1984 showed DDS to be a promising sludge stabilization alternative for those plants using the pure oxygen activated sludge process (II-16).

II-22.2 Description

The DDS technology includes the use of pure oxygen aerobic digestor with a one day detention time (the Step I reactor) followed by one or more anaerobic digesters with approximately eight days detention time (the Step II reactor). Figure II-22.1 shows a schematic representation of the process. PSA refers to oxygen generation and/or supply. The technology relies on the conservation of the heat generated in the Step I reactor by the biological oxidation of the sludge, allowing the reactor to reach mesophilic or thermophilic temperatures. The higher temperatures result in significant increases in the rate of volatile suspended solids reduction. The Step II anaerobic digestion process provides further digestion and the generation of methane gas.

The three wastewater treatment facilities studied include Hagerstown, MD; Nutbush Creek, NC; Lackawanna, NY. All of the plants previously used conventional two-stage anaerobic digestion for sludge stabilization and currently use pure oxygen activated sludge as their secondary wastewater treatment process. The oxygen is supplied by a pressure swing adsorption (PSA) oxygen

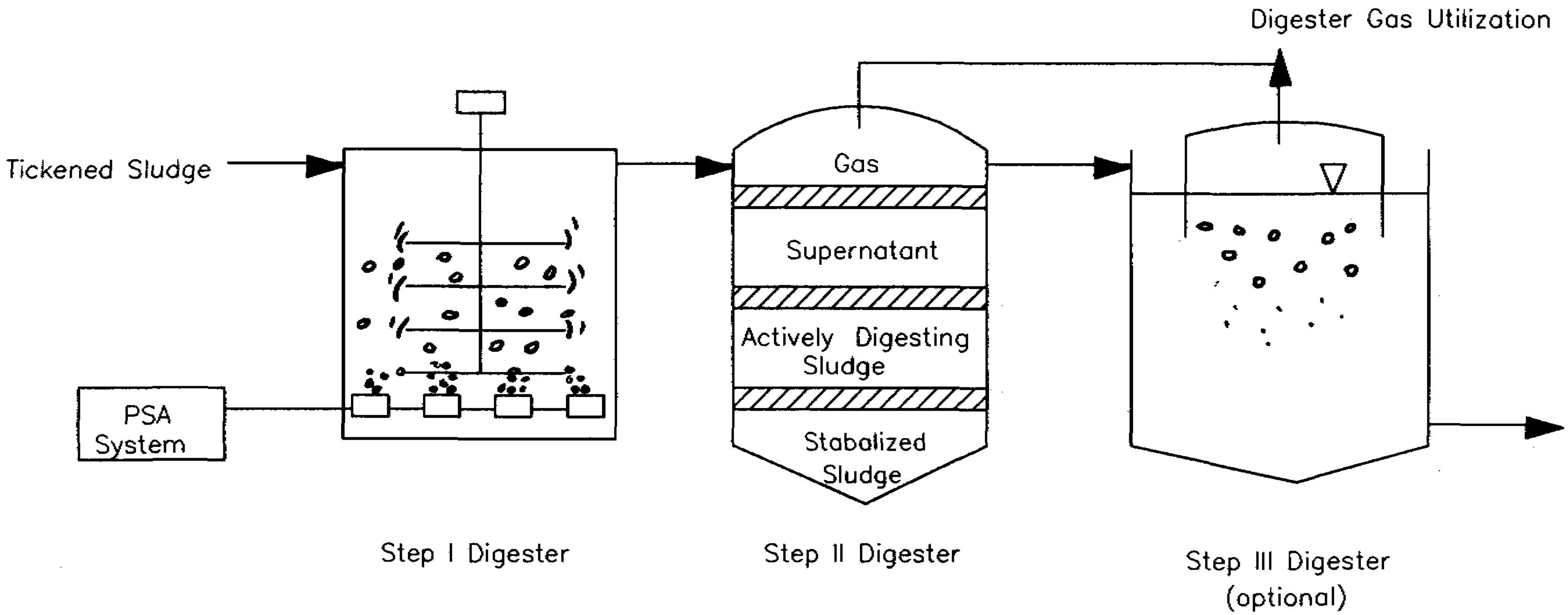

Figure II–22.1: DDS Schematic at Lackawanna WWTP.

generation system with a liquid oxygen backup. The additional oxygen requirements of the DDS are met by incrementally increasing the capacity of the PSA system.

The process modifications to incorporate DDS included the construction of a Step I reactor and modification of what was formerly the first stage digester to be the Step II reactor. The former second stage digester was retained and is used to provide additional anaerobic digestion and solid/solution separation. This digester is referred to informally as the Step III reactor but would not generally be constructed as part of a new system. A new floating cover was added to the Step III digester to provide storage for the biogas generated.

The sizes and operating conditions of the three facilities are presented in Table II-22.1

TABLE II - 22.1

DDS SUMMARIES

	Treatment Plant		
	Lackawanna	Hagerstown	Nutbush Creek
Startup Date	February, 1983	May, 1984	December, 1982
Average Daily Flow (MGD)	4.5	8.0	4.12
PSA System Capacity ton/day	3.0	17.2	6.0
DDS Feed Mode	Continuous	Intermittent	Intermittent
DDS Detention Time			
Days Step 1	1.17	0.9	1.25
Days Step 2	7.4	12.0	10.4

II-22.3 Performance

Table II-22.2 summarizes the DDS performance at the Lackawanna and Hagerstown wastewater treatment facilities.

Manufacturers' claims suggest that a 10% reduction in solids can be expected in the Step I reactor. Both Lackawanna and Hagerstown experienced even higher reductions. The overall volatile solids reductions at Lackawanna were between 26.9 and 44.4% during the study. Overall volatile solids reduction observed at Hagerstown ranged from 51.1 to 56.6%.

Using the solids reduction data and oxygen feed rate data, the specific oxygen feed rate to remove volatile solids was calculated at Lackawanna. Feed rates of 2.88 to 3.82 kg O_2 were necessary to remove one kilogram of volatile solids. The oxygen feed rate was not measured at Hagerstown.

Biogas production was measured at Hagerstown and Lackawanna. Compared to the manufacturer's estimates of 0.74 to 1.12 m^3 biogas per kg volatile solids destroyed, the values at Lackawanna were higher than expected at 1.18 to 1.51 m^3/kg volatile solids destroyed, while at Hagerstown they were lower than expected at 0.22 to 0.79 m^3/kg volatile solids destroyed. No determination could be made as to why the gas production at Hagerstown was lower than projected.

Dewaterability of the sludge was not able to be quantitatively evaluated. However, the minimum 20% total solids required by the local landfill were met with the processed sludge at both the facilities.

II-22.4 Operation

DDS requires few pieces of equipment in addition to that required as part of a conventional anaerobic treatment process.

TABLE II - 22.2

DDS PERFORMANCE

Parameter	Lackawanna	Hagerstown
Total Solids (%)	5.3	6.8
Volatile Solids Fraction (%)	62	79
Total Solids Loading (kg/d)	1,941	3,154
Temperature: (° C)		
Step 1 Effluent	50.2	49.4
Step 11 Effluent	42.4	35.9
pH		
Step 1 Effluent	7.3	6.5
Step 11 Effluent	7.4	7.6
Alkalinity: (mg/l as $CaCO_3$)		
Step 1 Effluent	3,434	NM
Step 11 Effluent	4,830	7,518
Step 1 Oxygen Feed (kg/d)	619	NM
Step II Biogas Production (m^3/d)	296	484
Total Inert Solids Reduction (%)		
Step 1	0.9-6.6	2.8-6.3
Step II	14.2-16.2	11.5-28.1
Overall	7.1-17.0	14.0-30.6
Total Volatile Solids Reduction (%)		
Step 1	13.7-19.9	10.8-16.4
Step II	14.3-30.6	45.2-48.0
Overall	26.9-44.4	51.1-56.6

NM: Not Measured

The major item is an oxygen generation system (which was already included in the plant design at both of the plants studied) to supply the requirements of the pure oxygen activated sludge process. An incremental increase in the system capacity was required for the addition of DDS to the plant design of these systems utilizing pure oxygen activated sludge wastewater treatment process.

The second major piece of equipment required by DDS is a mass transfer device to achieve enhanced oxygen transfer in the Step I digester. A motor driven, slowly rotating, custom designed vertical shaft extending the depth of the digester is used. The shaft is designed with five pairs of tapered arms to sheer the rising oxygen bubbles. Baffles are mounted on the side of the digester. The equipment is fabricated of stainless steel and was developed based on the experience at the Hagerstown demonstration project.

Mechanical reliability is a problem associated with the mass transfer device. Because of the counteracting forces of the rising gas bubbles and rotating blades, uneven forces are exerted on the shaft bearings. Failure of the shaft has occurred at Lackawanna. However, there are several options available to mitigate the potential mechanical problems. For example, at the Nutbush Creek facility, the total design volume was split between two digesters to provide some degree of backup and reliability.

II-22.5 Process Operation and Control

Operation of the DDS has been proven to be relatively simple. Compared to conventional anaerobic digestion, only one additional sample point is required for process monitoring. The oxygen fed rate is the main process control variable that must be monitored to control the Step I operating temperature. The design of a consistent sludge thickening process is also important to the performance and ease of operation of DDS. The raw sludge pumping rate depends on the solids production as well as the thickening process so that a total solids concentration of approximately 5% is maintained.

Based on actual operating experience, several modifications have been made to improve DDS operations. These include:

- Step I odor scrubbing system,
- Telescoping valve air-purge to prevent clogging,
- PSA oxygen generation totalizer,
- DDS oxygen feed totalizer,
- DDS automatic feed rate control.

II-22.6 Costs

The net present worth values of the estimated capital costs for the DDS system are summarized in Table II-22.3. Note that the capital cost for the oxygen generation system represents only the incremental cost for increasing the PSA system capacity. If the cost for a stand alone oxygen generation system had been used, there would be no advantage in using DDS.

Comparisons of costs are site specific. Table II-22.3 shows that, for Lackawanna, total costs for DDS are comparable to conventional systems, although such additional benefits as increased I/A funding and reduced sludge disposal volumes were not quantified.

Operating costs for DDS were difficult to determine since budgets for electrical power, labor, and chemicals were not broken down by process at these facilities. Measurements of electrical power were recorded during the study to calculate power consumption and yearly electrical cost. The annual power requirement can be expected to be nearly 20% greater during operation in the thermophilic mode.

TABLE II - 22.3

DDS COST ESTIMATE AT LACKAWANNA

	DDS		Conventional 2-Stage Anaerobic	
	Yearly	Present Worth[1]	Yearly	Present Worth[1]
Capital Cost[1]	-	$1,002,715	-	$818,600
Labor	$76,462	836,568	$67,809	741,888
Power	1,992	21,799	2,292	25,081
Chemicals	-	-	1,141	12,479
Maintenance & Spare Parts	-	130,026	-	151,200
Digester Heating	-	-	4,172	45,538
Digester Gas for Space Heating	10,488	114,752		
Total Present Worth		$1,876,356		$1,794,786

[1]20 years at 7%

II-22.7 Summary

The two facilities that were studied in detail demonstrated the successful utilization of DDS technology. Although solids reduction exceeded the goal, long term consistent performance was not shown at one facility. This may be attributable to sludge quality, process operation, or control.

One important advantage to using DDS is the higher degree of pathogen destruction achievable at the higher operating temperatures.

II-23 Aburra Water Treatment Plant (ref: II-17)

II-23.1 Location and Address

It is located on the right hand slopes of the Medellin River, south of Machado and near the Medellin-Bogota highway, in a place called Croacia, Columbia. The area of the lot is 15.3 ha and has elevations ranging from 1730 to 1800 m.s.n.m. The plant proper has an average elevation of 1752 m.s.n.m. The Empresas Publicas de Medellin, owners of the project, can be contacted for more information.

II-23.2 Description of the Technology

II-23.2a Source of Raw Water Supply

The source for the raw water supply will be the water originating from the new Rio Grande reservoir, with a useful capacity of 110 million m^3. From the influent works the water will flow through a tunnel 15.8 km long to the Hydroelectric Central of Niquia, from which the pumped water will flow by gravity through a pipeline to the Aburra Plant.

II-23.2b Treatment System Adopted

The system adopted for the treatment of the waters from the Rio Grande Reservoir, includes the processes of rapid mixing, flocculation, sedimentation, filtration and disinfection. If the quality of the waters requires it in the future, it is envisioned that the plant will convert to the direct filtration system, flocculation and sedimentation will not be used.

Metering

The metering for the volume of flow to the plant will be done through three Parshall flumes, each one with a capacity of 3.0 m^3/s.

Rapid Mixing and Coagulation

The coagulants will be added to the water in the Parshall flumes, which were designed to produce a hydraulic jump. The flume energy will be taken advantage of in the rapid mixing of the coagulants with the raw water. The primary coagulant will be alum, which will be applied along or in combination with some type of polyelectrolyte. Also, lime could be added before the Parshall flumes, to control the pH for optimum coagulation.

Flocculation

In the first stage, flocculation will take place in six rectangular concrete tanks. Each flocculator is made up of three zones, in each one of which there will be different velocity gradients. These could vary from between 80 and 20 sec-1. In the first and second zone the velocity gradients will be obtained through mechanical oscillating systems, such as wooden paddles. In the third zone, with a velocity gradient of 20 sec-1, flocculation is developed by means of concrete baffles that produce a horizontal flow component.

Sedimentation

In the first stage, the sedimentation process will take place in six rectangular concrete tanks. Each unit will be equipped with flat asbestos-cement plates, inclined 55 degrees to the horizontal. The entire surface area of the settlers will be used. The rate of sedimentation compared to conventional systems is much higher. The clarified water from each settler will be collected by means of six fiberglass flumes equipped with submerged orifices spaced over the length of the unit.

The sludge that is deposited over the flat plates will move by gravity as a result of the inclination of the plates, and then deposited at the bottom of the settlers. The deposited sludge will then be extracted hydraulically by means of a net work of perforated PVC pipes, interconnected with a system of two siphons. The siphons are located at the edge of the settler and adjacent to the flocculators and use water from the settlers.

Filtration

Twelve double-celled filters are used. Each cell measures 18 X 4 m, for a total of 1728 m^2. The filtering bed is composed of three layers: an upper layer of anthracite, 55 cm thick, an intermediate layer of sand, 25 cm thick, and a lower layer of gravel, 45 cm thick. The filtered water recovery system is composed of concrete beams, shaped like inverted V's, with orifices conveniently spaced, located near the apex. The beams are located above the title bottom of each filter. During filtration, discharge will occur into a central conduit located between the two cells of each filter, and then into a spillway to storage.

Flow is controlled by means of two weirs at the entrance to each filter, one for controlling the flow of water to each unit, and the other, a control weir common to all filters, that will allow the interconnecting canal of the filters to maintain a minimum level.

Backwashing uses filtered water. Backwashing is controlled by means of operating two gates: shutting off the intake of settled water, and opening the outlet for wash water. This operation could be done manually, or by means of electronic devices (operation based on water levels or predetermined schedules).

The filters could function in two modes of operation: on a constant volume of flow, or with a declining filtration rate. In both cases, the water level in the filters will be variable. Switching modes will be accomplished by means of a gate specially designed for this purpose.

Disinfection

The disinfection of the filtered water will come about by means of chlorine application at the entrance of the storage tank. The system has been designed to allow application of chlorine at the tank outlet.

Chemical Stabilization

In the event that it becomes necessary to stabilize the filtered water, a bed of lime will be applied at the entrance of the storage tank.

Chemical Products for Treatment

Alum and/or polyelectrolyte will be used for coagulation; lime will be used for control of pH for coagulation and chemical stabilization; potassium permanganate will be used to control odors and taste of water before treatment; activated carbon will be used to control odor and taste after filtration; and chlorine will be used for disinfection (postchlorination).

II-23.3 Design Criteria

Plant Capacity	
Design (1st stage)	6.0 m^3/s
Maximum (2nd stage)	9.0 m^3/s
Rapid Mixing	
Parshall Flumes	3 Units
Width of throat (gorge)	2433.6 mm
Flocculators	
Tanks	6 Units
Dimensions (LxWxH) each unit	16.2X20.7X5.2 m
Water Volume per unit	1744 m^3
Detention time	29 min.
Oscillating agitators	6 units
Range of "G" value	20-80 sec-1
Plate Settlers	
Tanks	6 Units
Dimensions (LxWxH)	33.7x20.7x5.8 m
Water Volume (c/u)	4046 m^3
Total retention time	67 min.
Retention time in plates	10 min.
Surface loading rate	159 $m^3/d/m^2$
Weir loading	183 $m^3/d/m$
Length of weirs	472 m
Plate Angle	55°

Filters

Number of units (double-celled)	12
Area (2 cells)	144 m^2
Filtration rate	300 $m^3/d/m^2$
Backwashing rate	0.90 m/min.
Volumes of Flow of backwash/filter	130 m^3/min.
Surface washing rate	0.15 m/min.
Volume of Flow of surface wash	22 m^3/min.
Filtering Medium	
Anthracite: Layer thickness	0.55 m
Effective size	0.90-1.10 mm
Uniformity coef.	1.45-1.55
Sand: Layer thickness	0.25 m
Effective size	0.50-0.55 mm
Uniformity coef.	1.45-1.55
Gravel: Layer thickness	0.50 m

Surface Wash Pumps

Number of units	2
Capacity per unit	21.5 m^3/min.
Power per unit	112.5 kw

Wash water recirculation pumps

Number of units	2
Capacity per unit	21.2 m^3/min
Power per unit	94 kw

Chemical Product Doses

Aluminum	30.0 mg/l
Coagulating polymer (optional)	0.5 mg/l
Polymer aids coag. (optional)	0.5 mg/l
Polymer aids filt. (optional)	0.5 mg/l
Chlorine (postchlorination)	4.0 mg/l
Lime	30.0 mg/l
Activated Carbon (optional)	10.0 mg/l
Potassium Permanganate (optional)	0.25 mg/l

II-23.4 Limitations

Mechanical agitation systems are used in the hydraulic flocculators to help manage high raw water solids at start up.

II-23.4.1 Advantages and Disadvantages

The design of the high rate settlers, and filters allows them to be smaller, which in turn represents a savings in labor costs. Similarly, the "autowashing" system for the filters represents a great savings by making conventional systems unnecessary.

The only identifiable disadvantage is the imbalance that the treatment process may experience as a result of the mechanical failure of the flocculating or dosing systems, although these have backup systems.

II-23.5 Costs

The approximate cost of the labor and equipment, as of May, 1988 is as follows:

- Preparation of access road and lot	$ 750,000
- Construction	4,500,000
- Supply of Equipment and materials	5,000,000
TOTAL	$10,250,000

II-23.6 History of Operation

As of March 1987, the plant was still under construction.

II-23.7 Other Criteria

For this particular plant, high rate settlers employing inclined plates, as well as high rate filters and autowashers were adopted as appropriate technology, due to the fact that this technology has extensively proven itself in Latin America and especially Columbia. In addition to the savings, simple operation of the plant is possible without the use of imported systems.

II-24 Process Name: "Unipack" Central Filtration Water Treatment Plant (ref: II-17)

II-24.1 Locations: Los Garzones, Cordoba and Croachi, Cundinamarca, Columbia

II-24.2 Description of the Technology

The system consists of a tank in which the mixing, flocculating, settling, and filtering processes all take place simultaneously and on a continuous basis.

The water to be treated is mixed with chemicals in the mixing chamber. The products of this reaction are generally insoluble solids of colloidal size, that take long to grow and settle. In "UNIPACK" this process of growing and settling is accelerated by the mixing of untreated, with previously treated water which contains suspended solids. This acceleration is much like the minimizing effect that solids have an oversaturated liquids. In "UNIPACK" this effect is obtained from the raw water inflow that keeps the previously formed sludge recirculating and makes the solid phase disperse finely into the liquid phase. The solid-liquid contact enhances precipitation of the suspended particles in the water. The special hydraulic design creates a flow regimen that results in solid-liquid mixing in a circular pattern towards the clarification zone.

The task that the balanced floating sludge blanket performs is very important, in that in addition to facilitating an excellent solid-liquid contact that provokes excellent flocculation, it acts like primary filtration. This offers greater efficiency and yield from the chemical addition. The sludge is easily controlled by a periodic manual purge.

Pretreated water is passed through a submerged high rate filter which eliminates the trace quantities of turbidity and results in a clear effluent. The filter has hydraulic collectors and diffusors balanced to avoid canalization and sludge clogs. It can be backwashed automatically.

Backwashing is accomplished automatically by means of a siphon which activates when head or the filter bed reaches a predetermined level (usually 1.20 to 1.50 meters above the filtered water outlet). The design calls for the backwash pipe to transport 21 gpm/ft.2 (5100 $m^3/m^2/min$) of filtering area. When the water level in the storage tank reaches the level of the siphon breaker, the siphon lets air into the backwash pipe; breaking the siphon, and ending the wash operation.

II-24.3 Application

The plants are designed for flow rates that oscillate between 1 l/sec and 60 l/sec. Typical applications are: water treatment for municipalities, that include removal of turbidity, softening as well as filtration and disinfection; water treatment for industries as diverse as textiles, nutrition, paper milling, mining, etc.

II-24.4 Design Criteria

Tolerances: The plant can operate at 10% of its design flow and handle overloads of up to 15% of its design flow. The design makes it an auto-operated system requiring a minimum of attention from the operator and a minimum of mechanical parts.

II-24.5 Limitations

The system design requires that the raw water arrives with a pressure equivalent to a head of 6.5 m static head.

II-24.6 Advantages of the System

The "UNIPACK" design and its operation principles are novel concepts in the field of water treatment:

1. It does not require electrical energy, a characteristic that makes it especially useful in many locations.

2. The automatic operation of the filter makes it a useful application where trained personnel are scarce, or where the budget cannot accommodate a highly trained operator.

3. The overflow (loading) rates are very low, a characteristic that makes it useful with all water sources, including rivers with changing turbidity.

4. The construction cost is lower than other treatment plants due to its compact size.

5. Maintenance cost is much lower than for conventional systems.

6. There are few mechanical parts subjected to wear and frequent replacement.

II.24.7 Costs

Capital: The estimated cost for LPS is currently (May 88) from $2,700 to $4,400 per l/sec, including laying the foundation and start-up. Plants with a greater volume of flow may have a lower unit l/sec cost.

Cost of Operation: The unit water treatment cost at the Choachi location (everything operates by gravity and there is no pumping), is estimated at 2.4¢ per m^3 (only one operator). In

Garzones (Cordoba) double pumping is required; from the river to the plant and from the plant to the elevated tanks. The average cost is 7¢ per m^3.

II.24.8 History of Operation

The first plants of this type were constructed by ACUATECNIA in Puerto Lleras in 1968 with a capacity of 15 l/sec. Today the plant continues to function adequately. Some problems which have been corrected include short circuiting and air binding in the overflow.

II.24.9 Other Criteria

Undoubtedly the system is more appropriate in small communities since it can be operated by untrained personnel. The operator can be native to the area and can be trained in less than eight days. As mentioned before, the system has no parts subjected to wear and does not require electrical energy. Also, a single operator can be in charge of the entire system.

II-25 Technology: Oxidation Pond (ref: II-17)

II-25.1 Location and Address: Municipio de Tabio, Sabana de Bogota, CAR, Carrera 10 ‡ 61-82, Bogota, Columbia, S.A.

II-25.2 Description of the Technology

In the Cota municipality in 1971 a pond was constructed to serve 2,322 inhabitants (estimated for 1980). In other words the design period was for 10 years. Today, Tabio has 3,700 habitants. Tabio is located 46km from Bogota.

The total area of the pond is 3 ha. A loading rate of 54 grams of BOD per day per capita was assumed. Two ponds were constructed in series in an attempt to achieve plug flow conditions.

The volume of flow of the combined wastewater reaches a distribution chamber of 1 m^2 area, through a 24" diameter pipe to the settler. An overflow effluent structure discharges into the Chicu river at about 20 l/sec.

The entrance volume of flow is conducted to two settlers constructed in parallel, that are operated alternately. Their dimensions are 21.5x10.4x0.42m. The retention time is 6 hours.

An inclined bar screen, at 60 degrees with rods separated by 1" spaces is used. The settled flow is distributed through 3-6" PVC pipes to the first stabilization pond. The ponds are connected with 12" diameter pipe, 13.4m long.

II-25.3 Design Criteria

- Design Load: 129 Kg/ha/day
- Retention time: 28 days

- The following two ponds were designed:

Parameter	1st Pond	2nd Pond
Length	138m	90m
Width	47m	47.5m
Area	$6193m^2$	$2100m^2$
Depth	2m	1m
Ret. time	23 days	5 days
Pond type	Facultative	Facultative

II-25.4 Limitations

Available land area was limited.

II-25.5 Advantages and Disadvantages

The first pond with a depth of 2m was not as efficient as anticipated, due to the fact that at this depth, the zones near the bottom of the pond had temperatures near 12° C. This temperature inhibits anaerobic activity.

Another disadvantage is that, since the sewage is combined, great variations in the load concentrations are possible.

II-25.6 Cost

The annual operating and maintenance costs are as follows:

Item	May 1988
Caretaking	$ 1160
Maintenance	717
Repairs	134
Others	89
Total	$ 2100

Maintenance includes sludge removal from the settlers. The pond should be cleaned every ten years.

If the pond were constructed today, its cost would be $120,000.

II-25.7 History of Operation

Between January and June of 1985, the pond achieved 83% BOD removal. Nevertheless, the system produced effluents less than or equal to 48 mg/l, 95% of the time.

Significant treatment (50% BOD removal) occurs in the primary settler. The ponds continue removing BOD,but efficiency is low because of solids accumulation. The average removal of total and fecal coliforms was in the order of 92 to 98% respectively; values that are considered high. The concentration of coliforms in the effluent is still very high, thus not appropriate for agricultural use. The removal of suspended solids was 73%.

II-26 Technology: Oxidation Ditch (ref: II-17)

II-26.1 Location and Address: Municipio de Cota, Sabana de Bogota, CAR, CRRA 10 # 16 82, Bogota, Colombia, SA

II-26.2 Description

A modified biological process using activated sludge and described as completely mixed, extended aeration. The population is 3,100, in a region described as high plains at 2659m with an average temperature of 14° C.

Experimental laboratory studies were done at three pilot plants, where a 0.16 lb BOD/lb MLVSS/day, food to microorganism ratio (F/M) was obtained. This allowed for satisfactory removals of BOD with minimal oxygen utilization. The efficiency of the system did not suffer when the organic loading was increased so as to obtain a 0.2-0.25 lb BOD/lb MLVSS/day, food to microorganism ratio (F/M). Larger quantities of oxygen and greater sludge production were needed at the increased loadings.

Adopting a concentration of solids in the aeration tank of 3000 mg/l and projecting an effluent of 219 mg/l, a hydraulic retention time of 11 hours was calculated. Afterwards, this value was revised to 18 hours. A design volume of 312 m^3 was selected. The required oxygen was calculated at 152 Kg 02/day. Settling tank dimensions are 40 m^2 area, and 3 m in depth. The drying beds are rectangular in shape; 18 m long and 6 m wide.

Sludge drying is by infiltration and evaporation. The BOD removal reached 90% and the total suspended solids 94%. Removal of total and fecal coliforms was 98% and 99%, respectively.

II-26.3 Design Criteria

- Elliptical ditch
- MLVSS: 3000-5000 mg/l
- Aeration time: 12 to 35 hours
- Cell residence time: 15 to 35 days (sludge retention time)
- Optimum F/M ration: 0.16 day-1

II-26.4 Limitations

Pretreatment facilities had to be constructed to reduce incoming solids. Better recirculation of the sludge was necessary in order to reach the MLVSS value in the design.

II-26.5 Cost

The annual operation and maintenance costs, are as follows:

	May, 1988
Caretaking and Operation	$ 2550
Energy	3800
Maintenance	230
TOTAL	$ 6580

Total construction cost of the plant: $100,000

II-26.6 History and Operation

During the first four years of operation (starting in 1981) it was necessary to construct and modify the treatment unit in order to achieve optimization for the Cota area. Good removal of organic load in terms of BOD and suspended solids only began in 1985. An extensive shake down period was required.

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TECHNOLOGY EVALUATION AND SELECTION

INTRODUCTION

This section is provided to further aid the process of evaluation and selection of technologies for application to water and wastewater treatment. There are many factors which may be used. The factors may be general in nature or specific to locations and problems.

The format of this handbook is designed to help in selection. The material is presented in two parts. The first part presents data and information which is applicable to the technology for general application. The second part is in the case study format and is given to illustrate specific applications. The fact that a technology has been used to solve specific problems lends credibility to its application elsewhere.

Each technology section is presented using evaluation and selection criteria. The criteria are: description (usually containing design criteria), limits (advantages and disadvantages when enough information is available), operation and maintenance (process control considerations), availability (ease of implementation), special factors, and recommendations. In some cases these criteria would not fit the specific technology and other factors were used. Sometimes not all of the factors were applicable to a technology.

The user of this handbook may compare one technology to another, or prepare evaluations of groups of technologies to meet a certain need or application. Cost is often over-emphasized in the evaluation process. A technically based evaluation should always be prepared. Process performance is the most important consideration. In other words, the technology must function in the application for which it is intended. Costs should only be used to compare processes when the performance of one or more

TABLE 1

TECHNOLOGY SELECTION GUIDELINES

Technologies	Life Cycle Cost	Cost Effectiveness	Reliability	Simplicity of Operation	Ease of Maintenance	Performance	Ability to Meet Water Quality Objectives	Adaptability to Change in Influent Quality	Performance Dependent on Pretreatment	Adaptability to Varying Flow Rate	Ease of Construction	Adaptability to Upgrading	Availability of Major Equipment	Equipment/Supplies Available Locally	Post-installation Service/Chemical Delivery	Personnel Skill Level	Energy Utilization	Residue Production	Cost of Residue Disposal	Potential for Effluent Use/Reuse	Importance of Air Emissions
Slow Sand Filter	L	M	H	M	L	H	H	H	Y	H	M	L	H	H	M	M	L	H	H	H	L
Rapid Sand Filter	M	M	M	L	L	M	H	M	Y	M	L	M	H	H	M	H	H	H	H	H	L
Chemical Precip. & Filtration	M	H	M	L	L	H	H	H	N	M	L	H	H	H	H	H	M	H	H	M	L
Dual Media Filter	H	M	H	L	L	H	H	H	Y	M	L	M	H	L	L	H	H	H	H	H	L
Sludge Vacuum Filter	M	M	H	L	L	M	H	H	Y	H	L	M	M	M	L	H	H	H	M	L	H
Sedimentation - Cir. Primary	M	M	H	M	M	M	L	H	N	H	L	L	H	M	M	M	L	H	M	L	M
Sedimentation - Rec. Primary	M	M	H	M	M	M	L	H	N	H	L	H	M	L	L	M	L	H	M	L	M
Upflow Filter	H	H	M	L	M	M	H	H	Y	M	L	M	H	L	L	H	M	H	H	M	L
Flocculation	M	H	H	L	M	H	H	H	N	M	L	M	H	H	M	H	L	H	H	H	M
Gravity Sewers	H	L	H	H	M	M	L	H	N	H	L	L	H	H	H	L	L	L	L	L	L
Pressure Sewers	L	M	M	M	L	H	H	M	Y	M	H	L	M	M	L	H	H	M	L	L	H
Lagoons - Facultative	L	H	M	M	M	L	L	H	N	M	M	H	H	H	H	M	L	M	L	L	M
Aquaculture	L	H	L	M	L	L	L	H	N	H	H	L	H	H	H	L	L	L	L	L	M
Preliminary Treatment	L	H	L	H	L	H	L	H	N	H	M	H	H	H	H	H	L	H	L	L	L
Rotary Screen	H	H	M	M	L	M	H	H	Y	M	L	M	M	M	L	H	H	H	M	H	L
Wedgewire Screen	H	H	M	M	L	M	H	H	Y	M	L	M	M	M	L	H	H	H	M	H	L
Trickling Filter - Plastic	M	H	M	M	H	M	L	H	N	M	M	M	M	M	M	M	M	M	L	L	M
Trickling Filter - Rock	M	H	L	M	M	M	L	M	N	M	M	H	H	H	H	M	M	M	L	L	M
Trickling Filter - Low Rate	M	H	L	M	M	M	L	H	N	L	M	H	H	H	H	M	L	M	L	M	M
Aerated Lagoons	M	M	L	L	L	M	M	H	N	M	M	M	H	H	H	M	H	M	L	L	H
Sludge Drying Bed	L	H	H	H	H	H	H	H	N	L	H	H	H	H	H	L	L	H	M	M	H

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Land Application Sludge	L	H	H	M	H	M	H	M	Y	L	H	M	M	M	M	M	L	L	L	M	H
Chlorination	H	H	H	L	L	H	H	H	Y	L	L	L	M	L	L	H	M	L	L	H	L
UV Disinfection	H	M	M	L	L	M	M	M	Y	L	L	L	L	L	H	H	M	L	L	H	L
Plate Settler	M	M	H	M	L	H	M	H	N	L	L	L	H	H	M	H	L	H	M	M	L
Land Application - Waste Water:																					
Irrigation	M	H	M	M	H	M	M	M	N	M	M	M	H	H	H	M	M	L	L	M	M
Overland Flow	L	M	L	M	L	L	L	H	Y	L	H	H	H	H	M	M	L	M	L	L	H
Perculation	H	H	M	M	H	M	M	M	N	M	M	L	H	H	H	M	M	L	L	H	H
Lagoon - Anaerobic	L	M	L	M	L	M	M	M	N	M	M	H	H	H	H	M	L	L	L	L	M
Lagoon Aerobic	M	M	M	L	L	M	M	H	N	M	M	L	H	M	M	M	H	L	L	M	M
Water Collector	M	M	H	L	L	M	-	M	Y/N	M	L	-	L	L	M	M	H	L	L	M	L
GAC Adsorption	H	VH	H	L	L	H	H	L	Y	L	L	L	L	L	L	H	H	M	H	M	M
Flotation	M	M	H	M	L	M	M	L	N	L	L	L	L	L	L	H	H	M	M	M	H
Imhoff Tank	L	H	H	M	L	M	M	M	N	H	M	H	H	H	H	M	L	M	M	L	M
Roughing Filter	L	L	L	M	M	M	M	M	N	H	M	H	H	H	H	M	L	M	L	L	M
RBC	M	M	M	L	L	M	M	M	Y	M	L	L	L	L	L	H	M	M	M	M	H
Activated Sludge	M	M	M	M	L	M	M	M	Y	M	L	H	M	M	M	H	M	M	M	M	M
Steep Slope Sewers	L	H	M	M	H	M	H	M	Y/N	M	H	L	L	L	L	L	L	L	L	L	L
SBR	M	M	M	L	L	M	M	-	Y	M	L	H	M	L	L	H	M	M	M	M	H
Intermittent Sand Filter	M	L	M	L	M	M	M	M	Y	M	L	-	M	H	L	H	M	H	H	H	L
Pulsed Bed Filtration	M	M	M	L	L	M	M	H	Y	M	L	-	H	H	L	H	M	H	H	H	L
Biological Nutrient Removal	M	L	L	L	L	M	M	M	Y	L	L	-	H	H	L	H	M	M	M	H	H
Hydrograph Release Lagoons	M	M	M	L	L	H	M	M	N	H	M	-	H	H	H	M	M	L	L	L	M
Vacuum Assisted Dewatering	M	H	M	L	L	H	H	M	N	H	L	-	H	L	L	H	H	H	H	L	H
Vacuum Sewers	L	M	L	L	L	H	L	L	Y	M	VL	-	L	L	L	H	H	M	L	L	H
Sludge Composting	L	M	M	M	M	H	M	M	N	H	H	-	H	H	H	M	L	VH	M	-	H
Methane Recovery	M	H	H	L	M	H	L	M	N	H	L	-	H	M	H	H	L	M	L	-	H
Dual Digestion	M	H	H	H	M	H	L	M	N	H	L	-	H	H	H	H	L	M	M	-	H

TABLE

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Slow Sand Filter	L	M	H	H	L	H	H	H	Y	H	M	L	H	H	M	M	L	H	H	H	L
Rapid Sand Filter	H	M	M	L	L	M	H	M	Y	M	L	H	H	H	M	H	H	H	H	H	L
Chemical Precip. & Filtration	M	H	H	L	L	H	H	H	N	M	L	H	H	H	H	H	M	H	H	H	L
Dual Media Filter	H	M	H	L	L	H	H	H	Y	M	L	M	H	L	L	H	H	H	H	H	L
Sludge Vacuum Filter	M	M	H	L	L	M	H	H	Y	H	L	M	M	M	L	H	H	H	M	L	H
Sedimentation - Cir. Primary	M	M	H	M	M	M	L	H	N	H	L	L	H	M	M	M	L	H	M	L	M
Sedimentation - Rec. Primary	M	M	H	M	M	M	L	H	N	H	L	H	M	L	L	M	L	H	M	L	M
Upflow Filter	H	H	M	L	M	M	H	H	Y	M	L	M	H	L	L	H	M	H	H	M	L
Flocculation	M	H	H	L	M	H	H	H	N	M	L	M	H	H	M	H	L	H	H	H	M
Gravity Sewers	H	L	H	H	M	M	L	H	N	H	L	L	H	H	H	L	L	L	L	L	L
Pressure Sewers	L	M	M	M	L	H	H	M	Y	M	H	L	M	M	L	H	H	M	L	L	H
Lagoons - Facultative	L	M	M	M	M	L	L	H	N	M	M	H	H	H	H	M	L	M	L	L	M
Aquaculture	L	H	L	M	L	L	L	H	N	H	H	L	H	H	H	L	L	L	L	L	M
Preliminary Treatment	L	H	L	H	L	H	L	H	N	H	M	H	H	H	H	H	L	H	L	L	L
Rotary Screen	H	H	M	M	L	M	H	H	Y	M	L	M	M	M	L	H	H	H	M	H	L
Wedgewire Screen	H	H	M	M	L	M	H	H	Y	M	L	M	M	M	L	H	H	H	M	H	L
Trickling Filter - Plastic	M	H	M	M	H	M	L	H	N	M	M	M	M	M	M	M	M	M	L	L	M
Trickling Filter - Rock	M	H	L	M	M	M	L	M	N	M	M	H	H	H	H	M	M	M	L	L	M
Trickling Filter - Low Rate	M	H	L	M	M	M	L	H	N	L	M	H	H	H	H	M	L	M	L	M	M
Aerated Lagoons	M	M	L	L	L	M	M	H	N	M	M	M	H	H	H	M	H	M	L	L	H
Sludge Drying Bed	L	H	H	H	H	H	H	H	N	L	H	H	H	H	H	L	L	H	M	M	H

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Land Application Sludge	L	H	H	M	H	M	H	M	Y	L	H	M	M	M	H	M	L	L	L	M	H
Chlorination	H	H	H	L	L	H	H	H	Y	L	L	L	M	L	L	H	M	L	L	H	L
UV Disinfection	H	M	M	L	L	M	M	M	Y	L	L	L	L	L	H	H	H	L	L	H	L
Plate Settler	M	H	H	M	L	H	M	H	N	L	L	L	H	H	M	H	L	H	M	M	L
Land Application - Waste Water:																					
Irrigation	M	H	M	M	H	M	M	M	N	M	M	M	H	H	H	M	M	L	L	M	H
Overland Flow	L	M	L	M	L	L	L	H	Y	L	H	H	H	H	H	M	L	M	L	L	H
Perculation	H	H	M	M	H	M	M	M	N	M	M	L	H	H	H	M	M	L	L	H	H
Lagoon - Anaerobic	L	M	L	M	L	M	M	M	N	M	M	H	H	H	H	M	L	L	L	L	M
Lagoon Aerobic	M	M	M	L	L	M	M	H	N	M	M	L	H	M	M	M	H	L	L	M	M
Water Collector	M	M	H	L	L	M	-	M	Y/N	M	L	-	L	L	M	M	H	L	L	H	L
GAC Adsorption	H	VH	M	L	L	H	H	L	Y	L	L	L	L	L	L	H	H	M	H	H	M
Flotation	M	M	H	M	L	M	M	L	N	L	L	L	L	L	L	H	H	M	M	M	H
Imhoff Tank	L	H	H	M	L	M	M	M	N	H	M	H	H	H	H	M	L	M	M	L	M
Roughing Filter	L	L	L	M	M	M	M	M	N	H	M	H	H	H	H	M	L	M	L	L	M
RBC	M	M	M	L	L	M	M	M	Y	M	L	L	L	L	L	H	M	M	M	M	H
Activated Sludge	M	H	M	M	L	M	M	M	Y	M	L	H	M	M	M	H	M	M	M	M	H
Steep Slope Sewers	L	H	M	M	M	M	H	H	Y/N	M	H	L	L	L	L	L	L	L	L	L	L
SBR	M	M	M	L	L	M	M	-	Y	M	L	H	M	L	L	H	M	M	M	M	H
Intermittent Sand Filter	M	L	M	L	M	M	M	M	Y	H	L	-	H	H	L	H	M	H	H	H	L
Pulsed Bed Filtration	M	M	M	L	L	M	M	H	Y	H	L	-	H	H	L	H	M	H	H	H	L
Biological Nutrient Removal	M	L	L	L	L	M	M	M	Y	L	L	-	H	H	L	H	M	M	M	H	H
Hydrograph Release Lagoons	M	M	M	L	L	H	M	M	N	H	M	-	H	H	H	M	M	L	L	L	M
Vacuum Assisted Dewatering	M	H	M	L	L	H	H	M	N	H	L	-	H	L	L	H	H	H	H	L	H
Vacuum Sewers	L	M	L	L	L	H	L	L	Y	M	VL	-	L	L	L	H	H	M	L	L	H
Sludge Composting	L	M	M	M	M	H	M	M	N	H	H	-	H	H	H	M	L	VH	M	-	H
Methane Recovery	M	H	H	L	M	H	L	M	N	H	L	-	H	M	H	H	L	M	L	-	H
Dual Digestion	M	H	H	H	M	H	L	M	N	H	L	-	H	H	H	H	L	M	M	-	H

TABLE

TECHNOLOGY SELECTION GUIDELINES

Technologies	Life Cycle Cost	Cost Effectiveness	Reliability	Simplicity of Operation	Ease of Maintenance	Performance	Ability to Meet Water Quality Objectives	Adaptability to Change in Influent Quality	Performance Dependent on Pretreatment	Adaptability to Varying Flow Rate	Ease of Construction	Adaptability to Upgrading	Availability of Major Equipment	Equipment/Supplies Available Locally	Post-installation Service/Chemical Delivery	Personnel Skill Level	Energy Utilization	Residue Production	Cost of Residue Disposal	Potential for Effluent Use/Reuse	Importance of Air Emissions
Slow Sand Filter	L	M	H	H	L	H	H	H	Y	H	M	L	H	H	M	M	L	H	H	H	L
Rapid Sand Filter	H	M	M	L	L	M	H	M	Y	M	L	H	H	H	M	H	H	H	H	H	L
Chemical Precip. & Filtration	M	H	H	L	L	H	H	H	N	M	L	H	H	H	H	H	M	H	H	H	L
Dual Media Filter	H	M	H	L	L	H	H	H	Y	M	L	M	H	L	L	H	H	H	H	H	L
Sludge Vacuum Filter	M	M	H	L	L	M	H	H	Y	H	L	M	M	M	L	H	H	H	M	L	H
Sedimentation - Cir. Primary	M	M	H	M	M	M	L	H	N	H	L	L	H	M	M	M	L	H	M	L	M
Sedimentation - Rec. Primary	M	M	H	M	M	M	L	H	N	H	L	H	M	L	L	M	L	H	M	L	M
Upflow Filter	H	H	M	L	M	M	H	H	Y	M	L	M	H	L	L	H	M	H	H	M	L
Flocculation	M	H	H	L	M	H	H	H	N	M	L	M	H	H	M	H	L	H	H	H	M
Gravity Sewers	H	L	H	H	M	M	L	H	N	H	L	L	H	H	H	L	L	L	L	L	L
Pressure Sewers	L	M	M	M	L	H	H	M	Y	M	H	L	M	M	L	H	H	M	L	L	H
Lagoons - Facultative	L	M	M	M	M	L	L	H	N	M	M	H	H	H	H	M	L	M	L	L	M
Aquaculture	L	H	L	M	L	L	L	H	N	H	H	L	H	H	H	L	L	L	L	L	M
Preliminary Treatment	L	H	L	H	L	H	L	H	N	H	M	H	H	H	H	H	L	H	L	L	L
Rotary Screen	H	H	M	M	L	M	H	H	Y	M	L	M	M	M	L	H	H	H	M	H	L
Wedgewire Screen	H	H	M	M	L	M	H	H	Y	M	L	M	M	M	L	H	H	H	M	H	L
Trickling Filter - Plastic	M	H	M	M	H	M	L	H	N	M	M	M	M	M	M	M	M	M	L	L	M
Trickling Filter - Rock	M	H	L	M	M	M	L	M	N	M	M	H	H	H	H	M	M	M	L	L	M
Trickling Filter - Low Rate	M	H	L	M	M	M	L	H	N	L	M	H	H	H	H	M	L	M	L	M	M
Aerated Lagoons	M	M	L	L	L	M	M	H	N	M	M	M	H	H	H	M	H	M	L	L	H
Sludge Drying Bed	L	H	H	H	H	H	H	H	N	L	H	H	H	H	H	L	L	H	M	M	H

TABLE 1

TECHNOLOGY SELECTION GUIDELINES

Technologies	Life Cycle Cost	Cost Effectiveness	Reliability	Simplicity of Operation	Ease of Maintenance	Performance	Ability to Meet Water Quality Objectives	Adaptability to Change in Influent Quality	Performance Dependent on Pretreatment	Adaptability to Varying Flow Rate	Ease of Construction	Adaptability to Upgrading	Availability of Major Equipment	Equipment/Supplies Available Locally	Post-installation Service/Chemical Delivery	Personnel Skill Level	Energy Utilization	Residue Production	Cost of Residue Disposal	Potential for Effluent Use/Reuse	Importance of Air Emissions
Land Application Sludge	L	H	H	M	H	M	H	M	Y	L	H	M	M	M	M	M	L	L	L	M	H
Chlorination	H	H	H	L	L	H	H	H	Y	L	L	L	M	L	L	H	M	L	L	H	L
UV Disinfection	H	M	M	L	L	M	M	M	Y	L	L	L	L	L	H	H	H	L	L	H	L
Plate Settler	M	H	H	M	L	H	M	H	N	L	L	L	H	H	M	H	L	H	M	M	L
Land Application - Waste Water:																					
Irrigation	M	H	M	M	H	M	M	M	N	M	M	M	H	H	H	M	M	L	L	M	H
Overland Flow	L	M	L	M	L	L	L	H	Y	L	H	H	H	H	H	M	L	M	L	L	H
Perculation	H	H	M	M	H	M	M	M	N	M	M	L	H	H	H	M	M	L	L	H	H
Lagoon - Anaerobic	L	M	L	M	L	M	M	M	N	M	M	H	H	H	H	M	L	L	L	L	M
Lagoon Aerobic	M	M	M	L	L	M	M	H	N	M	M	L	H	M	M	M	H	L	L	M	M
Water Collector	M	M	H	L	L	H	-	M	Y/N	M	L	-	L	L	M	M	H	L	L	H	L
GAC Adsorption	H	VH	H	L	L	H	H	L	Y	L	L	L	L	L	L	H	H	M	H	H	M
Flotation	M	M	H	M	L	M	M	L	N	L	L	L	L	L	L	H	H	M	M	M	H
Imhoff Tank	L	H	H	M	L	M	M	M	N	H	M	H	H	H	H	M	L	M	M	L	M
Roughing Filter	L	L	L	M	M	M	M	M	N	H	M	H	H	H	H	M	L	M	L	L	M
RBC	M	M	M	L	L	M	M	M	Y	M	L	L	L	L	L	H	M	M	M	M	H
Activated Sludge	M	M	M	M	L	M	M	M	Y	M	L	H	M	M	M	H	M	M	M	M	H
Steep Slope Sewers	L	H	M	M	M	M	H	M	Y/N	M	M	L	L	L	L	L	L	L	L	L	L
SBR	M	M	M	L	L	M	M	-	Y	M	L	H	M	L	L	H	M	M	M	M	H
Intermittent Sand Filter	M	L	M	L	M	M	M	M	Y	H	L	-	H	H	L	H	M	H	H	H	L
Pulsed Bed Filtration	M	M	M	L	L	M	M	H	Y	H	L	-	H	H	L	H	M	H	H	H	L
Biological Nutrient Removal	M	L	L	L	L	M	M	M	Y	L	L	-	H	H	L	H	M	M	M	H	H
Hydrograph Release Lagoons	M	M	M	L	L	H	M	M	N	H	M	-	H	H	H	M	M	L	L	L	M
Vacuum Assisted Dewatering	M	H	M	L	L	H	H	M	N	H	L	-	H	L	L	H	H	H	H	L	H
Vacuum Sewers	L	M	L	L	L	H	L	L	Y	M	VL	-	L	L	L	H	H	M	L	L	H
Sludge Composting	L	M	M	M	M	H	M	M	N	H	H	-	H	H	H	M	L	VH	M	-	H
Methane Recovery	M	H	H	L	M	H	L	M	N	H	L	-	H	M	H	H	L	M	L	-	H
Dual Digestion	M	H	H	H	M	H	L	M	N	H	L	-	H	H	H	H	L	M	M	-	H

technologies will meet the requirements of a given application. In fact, if this evaluation procedure is followed strictly, cost will often not be a consideration, because conditions from location to location are very different.

Selection of technologies can only be made once, and the utility management and plant operators must provide consistent service using the facility during the life of the plant. It is well worth the time and effort to perform a complete evaluation when a facility is being planned and when upgrading of an existing facility is being considered. Shorthand approaches to the selection process should be avoided by engineers. Leaving the technology selection to the consulting engineering company should be avoided. Selection should be a joint process.

TECHNOLOGY SELECTION GUIDELINE MATRIX

Recognizing that shorthand selection is to be avoided, it is nevertheless useful to consider evaluation and selection criteria in a summary fashion. Most technologies which are presented in Section I are included in Table 1, along with evaluation criteria. A summary assessment of each technology is given on Table 1.

Evaluations may be qualitative or quantitative. The evaluation provided on Table 1 is qualitative, using high (H), medium (M) and low (L), or yes (Y) / no (N) for Table entries. The evaluations performed for specific applications may be in more detail and quantitative. It is not possible in a report of limited scope to address specific applications. The evaluations given are to be considered general. The factors in Table 1 will be applicable to specific situations and may be used for detailed analyses, but the entries in the Table may not be applicable.

Most of the factors are self evident. The following comments may be useful as elaboration.

Life Cycle Cost - The total cost of acquisition and ownership over the full useful life of a facility. The total cost to acquire, operate, maintain, and salvage. The evaluation of life cycle cost is a technique which allows the consideration of total cost and local factors in the selection. Selections based on capital cost alone are unsatisfactory, as are those which do not consider major equipment replacements, for example, during plant lifetime. The costs presented in the handbook are capital and operating and maintenance (O&M), from which the user may construct life cycle cost estimates taking into account specific site and project factors.

Cost Effectiveness - The cost per unit of treatment and/or per unit performance. For example, $/lb of solids removed and processed. NOte that it is not sufficient to consider removal alone, but processing to the point where the pound of solids has been itself treated and permanently disposed. Cost effectiveness estimates may be developed by computing total annual costs (O&M plus amortized capital cost, for the removal treatment technology and the solids processing technology) divided by the total amount processed (in the case of the solids example, total solids removed, dewatered, landfilled, etc.). Cost effectiveness is a term often used and almost never correctly applied to process selection and evaluation. The cost effectiveness determination must consider local factors.

Performance - and - Ability to Meet Water Quality Treatment Objectives - These two factors may be considered together. Design should be based on the expectation for the production of water with required effluent quality characteristics. Expected performance is the key to the evaluation. Quantitative performance information is given later in this subsection.

Adaptability to Changes in Influent Quality -
Performance Dependability on Pretreatment -

Adaptability to Changes in Flow Rate - These three factors may be considered together. They relate to how the technology may be expected to function separately. Typically, process performance is dependent on the performance of upstream processes. How the technology relates to the operation of other - including downstream - processes is also important.

Availability of Major Equipment -
Equipment / Supplies Available Locally -
Post Installation Service -

These three relate heavily to local considerations. The qualitative summary evaluations in Table 1 may have the least accuracy in special applications.

PERFORMANCE AND REMOVAL CAPABILITY

A qualitative summary of technology performance capability is given on Table 2 (67 and other sources). The information herein should not be used as a substitute for testing. Jar tests, pilot scale evaluations, and even full scale tests at existing plants should always be used whenever capital expenditures are planned. It is likely that the tests will pay for themselves in saved capital and O&M costs.

Both water and wastewater treatment are considered in Table 2. For general purposes, the following may be considered typical:

L = 0 to 30%
M = 20 to 70%
H = 60 to 99%
L/H = widely variable performance and probably not applicable.

TABLE 2

TECHNOLOGY REMOVAL CAPABILITY

Technologies	Pesticides	Arsenic	Asbestos	Cadmium	Chromium	Copper	Cyanides	Lead	Zinc	Bis-Chloro Methyl Ether	Phthalates	Amines	Total Phenols	Benzene	Chloroform	Carbon Tetra Chloride	Xylenes	Benzo(a) Pyrene	BCs	Methylene Chloride	THMs	THM Pre-Cursors	Hardness	Taste & Odor	Color	Turbidity
Coagulation and Flocculation	M	M/H	·	H	M/H	·	M	L/H	L/H	·	L/H	·	L	·	·	L	·	·	·	M	·	H	·	·	·	M
PAC Adsoprtion with Activated Sludge	·	·	·	·	L/H	M/H	M	·	M/H	·	·	·	H	·	·	·	·	·	·	·	M	H	·	·	·	·
GAC Adsorption	·	L/H	·	M/H	L/H	L/M	L/H	L/M	L/H	·	M/H	·	M/H	H	M/H	·	·	H	·	M/H	H	H	·	·	·	·
Chemical Precipitation with Sedimentation	·	·	·	·	·	·	·	·	·	·	·	H	·	·	·	·	·	·	·	·	·	H	·	·	·	M/H
Lime	L	M/H	H	L/H	M/M	L/H	M/H	L/H	M/H	·	M/H	·	L/H	H	H	·	·	H	·	L	·	·	M/H	·	·	·
Ferric Chloride	·	H	·	·	·	·	·	·	·	·	L	·	L	·	·	·	·	·	·	H	·	·	H	·	·	·
Polymer	·	·	·	L/H	M/H	L/H	L/H	M/H	L/H	·	·	·	·	L	·	H	·	·	·	H	·	·	H	·	·	·
Sodium Carbonate	·	H	·	M/H	H	M	·	H	H	·	·	·	·	·	L	·	·	·	·	·	·	·	·	·	·	·
Barium Chloride	·	L	M	·	M	M	·	·	M	·	·	·	·	·	·	·	·	·	·	·	·	·	·	·	·	·
Sodium Hydroxide	·	·	·	L/H	M/H	L/H	·	H	H	·	M/H	·	H	H	H	·	·	·	·	H	·	·	M/H	·	·	·
Combined Precipitants	·	L/H	·	L/H	M/H	M/H	·	L/H	L/H	·	H	·	M/H	M	H	H	·	·	·	L	·	·	·	·	·	·
Alum	L	L	·	M	L/H	L/H	M	L/H	M	·	L/H	·	L/H	H	M/H	H	·	·	·	H	·	·	M/H	·	·	·
Chemical Precipitation with Filtration	·	L	·	L/H	M/H	L/H	M	H	H	·	M/H	·	H	·	·	·	·	·	·	·	·	·	·	·	·	H
Filtration	L/H	L/H	H	L/H	L/H	L/H	L/H	L/H	L/H	·	M/H	H	L/H	·	H	H	H	·	L	L/H	·	·	·	·	·	M/H
Sedimentation	L/H	L/H	H	L/H	L/H	L/H	L/H	L/H	L/H	·	M/H	·	L/H	M/H	M	H	·	M/H	·	·	·	·	·	·	·	L/H
Activated Sludge	M	L/H	·	L/H	L/H	L/H	L/H	L/H	L/H	H	L/H	M/H	L/H	M/H	L/H	H	·	·	·	·	·	·	·	·	·	·
Lagoons																										
Aerated	·	H	·	H	L/H	M/H	H	H	M/H	·	L/H	·	L/H	M	H	·	·	·	·	·	·	·	·	·	·	·
Non-Aerated	·	·	·	·	M	·	·	H	H	·	·	·	L	·	·	·	·	·	·	·	·	·	·	·	·	·
Chemical Oxidation																				·	·	·	·	H	H	·
Chlorine	·	·	·	·	·	L	H	·	·	·	·	·	·	·	·	·	·	·	·	·	·	·	·	·	H	·
Ozone	·	M	·	·	·	L	L/H	L	H	·	M/H	M	·	M	·	·	·	H	·	·	H	H	·	·	H	·
Trickling Filter	·	·	·	·	·	·	M	·	·	·	M	·	H	·	·	·	·	·	·	·	·	·	·	·	·	·
Roughing Filter	·	·	·	·	·	·	M	·	·	·	M	·	H	·	·	·	·	·	·	·	·	·	·	·	·	·
RBC	·	·	·	·	·	M	·	·	·	·	·	·	·	·	·	·	·	·	·	·	·	·	·	·	·	·
Chemical Reduction	·	·	·	·	·	L/H	·	H	H	·	·	·	·	·	·	·	·	·	·	·	·	·	·	·	·	·

TABLE 2

TECHNOLOGY REMOVAL CAPABILITY

Technologies	Pesticides	Arsenic	Asbestos	Cadmium	Chromium	Copper	Cyanides	Lead	Zinc	Bis-Chloro Methyl Ether	Phthalates	Amines	Total Phenols	Benzene	Chloroform	Carbon Tetra Chloride	Xylenes	Benzo(a) Pyrene	BCs	Methylene Chloride	THMs	THM Pre-Cursors	Hardness	Taste & Odor	Color	Turbidity
Coagulation and Flocculation	M	M/H	-	H	M/H	-	M	L/H	L/H	-	L/H	-	L	-	-	L	-	-	-	M	-	H	-	-	-	H
PAC Adsoprtion with Activated Sludge	-	-	-	-	L/H	M/H	M	-	M/H	-	-	-	H	-	-	-	-	-	-	-	H	H	-	-	-	-
GAC Adsorption	-	L/H	-	M/H	L/H	L/M	L/H	L/M	L/H	-	M/H	-	M/H	H	M/H	-	-	H	-	M/H	H	H	-	-	-	-
Chemical Precipitation with Sedimentation	-	-	-	-	-	-	-	-	-	-	-	H	-	-	-	-	-	-	-	-	-	H	-	-	-	M/H
Lime	L	M/H	H	L/H	M/H	L/H	M/H	L/H	M/H	-	M/H	-	L/H	H	H	-	-	H	-	L	-	-	M/H	-	-	-
Ferric Chloride	-	H	-	-	-	-	-	-	-	-	L	-	L	-	-	-	-	-	-	H	-	-	H	-	-	-
Polymer	-	-	-	L/H	M/H	L/H	L/H	M/H	L/H	-	-	-	-	L	-	H	-	-	-	H	-	-	H	-	-	-
Sodium Carbonate	-	H	-	M/H	H	M	-	H	H	-	-	-	-	-	L	-	-	-	-	-	-	-	-	-	-	-
Barium Chloride	-	L	M	-	M	M	-	-	M	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Sodium Hydroxide	-	-	-	L/H	M/H	L/H	-	H	H	-	M/H	-	H	H	H	-	-	-	-	H	-	-	M/H	-	-	-
Combined Precipitants	-	L/H	-	L/H	M/H	M/H	-	L/H	L/H	-	H	-	M/H	M	H	H	-	-	-	L	-	-	-	-	-	-
Alum	L	L	-	M	L/H	L/H	M	L/H	M	-	L/H	-	L/H	H	M/H	H	-	-	-	H	-	-	M/H	-	-	-
Chemical Preciptation with Filtration	-	L	-	L/H	M/H	L/H	M	H	H	-	M/H	-	H	-	-	-	-	-	-	-	-	-	-	-	-	H
Filtration	L/H	L/H	H	L/H	L/H	L/H	L/H	L/H	L/H	-	M/H	H	L/M	-	M	H	M	-	L	L/H	-	-	-	-	-	M/H
Sedimentation	L/M	L/H	M	L/H	L/H	L/H	L/H	L/H	L/M	-	M/H	-	L/H	M/H	M	H	-	M/H	-	-	-	-	-	-	-	L/H
Activated Sludge	M	L/H	-	L/H	L/H	L/H	L/H	L/H	L/H	H	L/H	M/H	L/H	M/H	L/H	H	-	-	-	-	-	-	-	-	-	-
Lagoons																										
Aerated	-	H	-	H	L/H	M/H	H	H	M/H	-	L/H	-	L/H	M	H	-	-	-	-	-	-	-	-	-	-	-
Non-Aerated	-	-	-	-	H	-	-	H	H	-	-	-	L	-	-	-	-	-	-	-	-	-	-	-	-	-
Chemical Oxidation																				-	-	-	-	H	H	-
Chlorine	-	-	-	-	-	L	H	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	H	-
Ozone	-	M	-	-	-	L	L/H	L	H	-	M/H	M	-	M	-	-	-	H	-	-	H	H	-	-	H	-
Trickling Filter	-	-	-	-	-	-	M	-	-	-	M	-	H	-	-	-	-	-	-	-	-	-	-	-	-	-
Roughing Filter	-	-	-	-	-	-	M	-	-	-	M	-	H	-	-	-	-	-	-	-	-	-	-	-	-	-
RBC	-	-	-	-	-	M	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Chemical Reduction	-	-	-	-	-	L/H	-	M	M	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

TABLE 2

TECHNOLOGY REMOVAL CAPABILITY

Technologies	Pesticides	Arsenic	Asbestos	Cadmium	Chromium	Copper	Cyanides	Lead	Zinc	Bis-Chloro Methyl Ether	Phthalates	Amines	Total Phenols	Benzene	Chloroform	Carbon Tetra Chloride	Xylenes	Benzo(a) Pyrene	BCs	Methylene Chloride	THMs	THM Pre-Cursors	Hardness	Taste & Odor	Color	Turbidity
Coagulation and Flocculation	H	M/H	-	H	M/H	-	H	L/H	L/H	-	L/H	-	L	-	-	L	-	-	-	H	-	H	-	-	-	H
PAC Adsoprtion with Activated Sludge	-	-	-	-	L/H	M/H	H	-	M/H	-	-	-	H	-	-	-	-	-	-	-	H	H	-	-	-	-
GAC Adsorption	-	L/H	-	M/H	L/H	L/M	L/H	L/H	L/H	-	M/H	-	M/H	H	M/H	-	-	H	-	M/H	H	H	-	-	-	-
Chemical Precipitation with Sedimentation	-	-	-	-	-	-	-	-	-	-	-	H	-	-	-	-	-	-	-	-	-	H	-	-	-	M/H
Lime	L	M/H	H	L/H	M/H	L/H	M/H	L/H	M/H	-	M/H	-	L/H	H	H	-	-	H	-	L	-	-	M/H	-	-	-
Ferric Chloride	-	H	-	-	-	-	-	-	-	-	L	-	L	-	-	-	-	-	-	H	-	-	H	-	-	-
Polymer	-	-	-	L/H	M/H	L/H	L/H	M/H	L/H	-	-	-	-	L	-	H	-	-	-	H	-	-	H	-	-	-
Sodium Carbonate	-	H	-	M/H	H	M	-	H	H	-	-	-	-	-	L	-	-	-	-	-	-	-	-	-	-	-
Barium Chloride	-	L	M	-	H	M	-	-	M	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Sodium Hydroxide	-	-	-	L/H	M/H	L/H	-	H	H	-	M/H	-	H	H	H	-	-	-	-	H	-	-	M/H	-	-	-
Combined Precipitants	-	L/H	-	L/H	M/H	M/H	-	L/H	L/H	-	H	-	M/H	M	H	H	-	-	-	L	-	-	-	-	-	-
Alum	L	L	-	M	L/H	L/H	M	L/H	M	-	L/H	-	L/H	H	M/H	H	-	-	-	H	-	-	M/H	-	-	-
Chemical Preciptation with Filtration	-	L	-	L/H	M/H	L/H	M	H	H	-	M/H	-	H	-	-	-	-	-	-	-	-	-	-	-	-	H
Filtration	L/H	L/H	H	L/H	L/H	L/H	L/H	L/H	L/H	-	M/H	H	L/H	-	M	H	M	-	L	L/H	-	-	-	-	-	M/H
Sedimentation	L/H	L/H	H	L/H	L/H	L/H	L/H	L/H	L/H	-	M/H	-	L/H	M/H	M	H	-	M/H	-	-	-	-	-	-	-	L/H
Activated Sludge	H	L/H	-	L/H	L/H	L/H	L/H	L/H	L/H	H	L/H	M/H	L/H	M/H	L/H	H	-	-	-	-	-	-	-	-	-	-
Lagoons																										
Aerated	-	H	-	H	L/H	M/H	H	H	M/H	-	L/H	-	L/H	M	H	-	-	-	-	-	-	-	-	-	-	-
Non-Aerated	-	-	-	-	H	-	-	H	H	-	-	-	L	-	-	-	-	-	-	-	-	-	-	-	-	-
Chemical Oxidation																				-	-	-	-	H	H	-
Chlorine	-	-	-	-	-	L	H	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	H	-
Ozone	-	H	-	-	-	L	L/H	L	H	-	M/H	M	-	M	-	-	-	H	-	-	H	H	-	-	H	-
Trickling Filter	-	-	-	-	-	-	H	-	-	-	H	-	H	-	-	-	-	-	-	-	-	-	-	-	-	-
Roughing Filter	-	-	-	-	-	-	H	-	-	-	H	-	H	-	-	-	-	-	-	-	-	-	-	-	-	-
RBC	-	-	-	-	-	H	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Chemical Reduction	-	-	-	-	-	L/H	-	H	H	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

Table 3 (3) contains largely qualitative information also, mostly pertinent to water treatment.

COMPARING GROUPS OF PROCESSES

An example of comparing a group of technologies is given in Table 4 (67), in this case for biological treatment processes and land treatment. The values may be considered average removals. Care should be exercised in using such average numerical values, because of variations in system variations, influent quality, sampling and analytical measurements, and a variety of other factors. Example conclusions for the comparison on Table 4 are: a) RBC and land treatment are better for ethylbenzene (and similar organics) removal than the other processes - same for nickel, and b) preliminary treatment is not effective for removal of most of the contaminants shown.

The user may make similar comparisons for other groups of technologies using actual local data. Another example is given for filtration on Table 3. It is impossible to provide all groupings which might be of interest.

Median values for removals of common contaminant parameters and some technologies are given on Table 5.

TABLE 3

PROCESS SELECTION GUIDELINES

Water Quality Parameters	Process and Components	Applicability	Comments
Turbidity	In-line filtration Coagulation Filtration	Low turbidity, low color	Greater operator attention required; shorter filter run than with direct filtration and conventional treatment; additional sludge-handling facilities may be required; pilot plant studies may be required; lower capital and O & M costs
	Direct filtration Coagulation Flocculation Filtration	Low to moderate turbidity, low to moderate color	Greater operator attention required; greater sludge-handling facilities may be required; pilot plant studies may be required; lower capital and O & M cost; better filter run time than in-line filtration but shorter than conventional treatment
	Conventional Coagulation Flocculation Sedimentation Filtration	Moderate to high turbidity, moderate color	Detention time in sedimentation basins allows for adequate contact time for T&O and color removal chemicals; more operational flexibility and less operation attention required
	Microscreening	Removal of gross particulate matter (e.g., algae)	Process relies on straining mechanism; process could not meet water quality objective if used alone
Bacteria/virus	Chemical Disinfection Chlorine Chloramine Chlorine dioxide Ozone Other chemicals Bromine Iodine Potassium permangate	Disinfection of surface and ground waters	THM potential with chlorine needs to be assessed, chloramine treatment not as potent as chlorine but does eliminate THM formation; cost of treatment: $Cl_2 <$ Chloramine $< ClO_2 <$ ozone; other chemicals such as bromine, iodine, etc., limited to small application
	Nonchemical disinfection Ultraviolet Ultrasonic	Disinfection of surface and ground waters	Advantage of UV is that no residual is left, which is applicable to fish aquariums and hatchery disinfection; ultrasonic is expensive, some success when used with ozone in tertiary disinfection
Color	Coagulation	High color levels	Use of high coagulant dose and low pH (5-6) is cost-effective when high color levels exist

	Adsorption GAC PAC Synthetic resins	Moderate to low color levels	GAC bed life is in the order of 1-6 weeks; synthetic resins are expensive (capital and regeneration costs); PAC is used for handling short-term color problems (however, it is costly as a routine color control method)
	Oxidation Chlorine Ozone Potassium permanganate Chlorine dioxide	Low, consistent color levels	Effectiveness: Ozone>Cl^2>ClO_2>$KMnO_4$; Cl_2 and $KMnO_4$ typically used for other purpose (disinfection and T&O control) but are effective for color control
Taste and order	Source control Copper sulfate Reservoir Destratification	Used to prevent any T&O problems in plant	Most satisfactory way of controlling is to control the problem at the source; copper sulfate may require chelation at certain pH values
	Oxidation Chlorine Ozone Potassium permanganate Chlorine dioxide	Low T&O levels	Chlorine may result in increased odor problems where odors are of industrial or algal origin; $KMnO_4$ widely used and very effective for odor control (however, overdosing may result in slightly pink color)
	Adsorption GAC PAC	Low to moderate levels, GAC used for industrial odor sources	PAC, in slurry form, is usually added at coagulation process for moderate T&O levels and ahead of filters for low T&O levels; GAC commonly used for odors caused by industrial sources - bed life usually very long
Hardness	Lime-soda softening Ion exchange	Extremely hard waters Removes not only hardness but also selective constituents THMs	Most common method of removal of hardness. Very expensive, especially in large-scale facilities
Organics (THM)	Alternative disinfectants Chloramine Chlorine dioxide Ozone Removal of precursors Chlorine dioxide Ozone GAC PAC Coagulation Removal of THM Ozone GAC PAC	THMs	Chloramines are not as powerful a disinfectant as free chlorine; ozone offers no appreciable residual protection in distribution system; modification of chlorine addition points may reduce THM formation; PAC has been found to yield only partial removals at very high doses

TABLE 4

REMOVING POLLUTANTS BY TREATMENT PROCESSES (PERCENT)

Parameter	Primary Treatment	Trickling Filter	Activated Sludge	Lagoon	Rotating Biological Contractor	Land Treatment Infiltration Percolation
BOD	-	82	91	90	96	95
COD	8	74	83	75	93	93
Oil and Grease	38	74	78	71	86	90
Total susp. soilids	48	88	90	92	93	98
Ammonia nitrogen	-	4	25	53	37	87
Benzene	-	92	81	75	96	96
Bis(2-ethylberyl) phthalate	44	60	50	-	94	91
Butyl benzyl phthalate	44	55	-	-	72	72
Cadmium	35	93	80	-	96	96
Chloroform	44	78	74	56	97	94
Chromium	35	85	87	72	98	98
Copper	9	72	86	86	99	93
Cyanide	-	62	72	79	82	94
Diethyl phthalate	-	-	-	-	-	-
Ethylbenzene	10	87	78	72	95	94
Lead	-	58	57	-	56	56
Mercury	11	40	26	76	71	71
Methylene chloride	-	96	-	99	99	99
Nickel	23	66	36	72	92	92
Phenols, total	-	75	76	60	86	97
Solids, total	-	25	26	62	8	30
Solids, volatile, total	4	55	53	76	-	55
Tetrachloroethylene	11	92	80	95	98	96
TOC	5	70	73	65	84	94
Toluene	-	98	94	92	100	99
Trichloroethylene	16	96	81	92	99	98
Zinc	-	90	78	87	98	98

Removal is calculated on the basis of average influent to the POTW and effluent of each particular process.

TABLE 5

MEDIAN REMOVAL EFFICIENCIES
(Percent)

	BOD_2	COD	TKN	TOC	TSS
Sedimentation	23	93	-	32	97
Polymer	50	71	-	82	99
Lime	52	32	-	18	71
Lime Polymer	-	99	-	22	99
Alum, Coag., Aid	37	59	-	47	66
Alum	61	10	-	63	84
Alum, Lime	41	86	-	80	93
Filtration	24	24	-	13	67
Activated Sludge	93	67	-	69	44
Aerated Lagoons	86	62	77	45	45
Granular Carbon	52	50	-	55	38
Powdered Carbon with Activated Sludge	-	-	-	-	-
Ozonation	-	50	-	9	15