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WASTE STABILIZATION PONDS

EARNEST F. GLOYNA

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WASTE STABILIZATION PONDS

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WORLD HEALTH ORGANIZATION

GENEVA

1971

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PREFACE

With increasing urbanization and industrialization, the volume of domestic sewage, industrial effluents, agricultural wastes, and urban run-offs is steadily growing. All these waste-waters must be assimilated into the environment without impairing the health and well-being of man. It is therefore often necessary to assist the natural processes of purification through the use of biological waste treatment plants. The type of facility required will depend upon the assimilative capacity of the environment and upon the use that will be made of the receiving water or, in some cases, of the treated waste-water itself.

Waste stabilization ponds provide a useful method of waste-water treatment and disposal for growing communities where both funds and trained personnel are in short supply. In these ponds, beneficial organisms stabilize the waste-water into a liquid that can be released to the environment without endangering man directly or affecting the environment adversely, and that does not place an undue cost burden on a downstream user.

Up to now the technical literature on the management and design of low-cost waste-water treatment systems has not been readily available in usable form. The purpose of this monograph is to summarize the available information on waste stabilization ponds : it defines acceptable design criteria based on public health considerations, suggests alternative approaches to design, provides data on pond design, and deals with the operational problems that ultimately determine the success or failure of any waste-water treatment facility. Some of the background information was obtained through a survey conducted by WHO of the use of waste stabilization ponds in a number of countries. In addition, the monograph provides useful information on the theory of biological waste treatment.

The monograph is directed specifically to the design engineer who has limited funds at his disposal, who has to rely on modest construction facilities, and who has to provide operating instructions that can be followed by unskilled personnel. It will assist him in selecting the most suitable design in the light of economic considerations, availability of raw materials, climatic conditions, and social factors.

A preliminary draft of the monograph was circulated to experts in different parts of the world, and the present publication incorporates many of their suggestions. A list of these reviewers will be found on pages 7–9. To them, and particularly to the author, Dr E. F. Gloyna, the World Health Organization expresses its sincere gratitude.

CHAPTER 1

General Considerations

The activities of man give rise to a wide range of waste products, many of which become waterborne and must be carefully treated before being released to the environment. Such waste-waters may contain excreta, household wastes, industrial discharges, agricultural run-offs, and urban storm drainage. All these wastes, individually or collectively, can pollute and contaminate[1] the environment.

There is an increasing need for low-cost methods of treating waste-waters, particularly municipal sewage and industrial effluents. The operation of such methods, and the maintenance of the necessary plant and equipment, must be within the capability of developing urban centres and industrial complexes.

As migration from rural to urban areas continues, the control of waste-water[2] will become increasingly difficult. This problem has two aspects: water must be supplied for the carriage of household and industrial wastes, and the waterborne wastes from a community that has an adequate water supply must be safely disposed of.

The protection of water resources against pollution is basic to the development of a sound economy. For both the maintenance of public health and the conservation of water resources, it is essential that pollution be controlled.

As urbanization continues household privies, septic tanks, sand-filter drains, and other methods of disposing of excreta may create economic and health problems. Such practices as the disposal of night-soil by burial or the disposal of waste-waters in cesspools (underground pits) can also contribute significantly to the pollution of soil and water if attempted on the scale required for a large urban centre.

Newly developing areas must determine the waste-water treatment measures that are required, and government agencies should proceed as

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[1] A *polluted water* contains organic materials or other wastes to the extent that it is not suitable for domestic use, whereas a *contaminated water* contains disease-causing agents. A water polluted with excreta will undoubtedly be contaminated.

[2] The term "waste-water" is used in preference to "sewage". The excreta and other substances in domestic waste-waters may be suspended, dissolved, or in the colloidal state.

rapidly as possible with planned waste-control programmes. Problems usually arise in allocating funds for water supply and waste-water disposal and expenditures must be equitably distributed between the collection and the treatment of community wastes. An elaborate treatment scheme, whether biological or chemical, is impractical without a satisfactory collection system.

It is generally recognized that some form of biological treatment provides the most economical solution for handling domestic and most industrial waste-waters. Waste stabilization ponds—the form of biological treatment with which this monograph is concerned—are most suitable for locations where land is inexpensive, organic loadings fluctuate, currency restrictions are in force, and there is a shortage of trained operating personnel.

The design of a waste stabilization pond depends on the treatment objectives. A pond system is usually designed to receive untreated domestic or industrial wastes, but may also be designed to treat primary or secondary treatment plant effluents, excess activated sludge, or diluted night-soil. The ponds may be used to pretreat wastes, to remove most of the biochemical oxygen demand (BOD),[1] and to reduce the concentration of disease-causing agents.

In waste stabilization ponds, the decomposable organic wastes are stabilized by micro-organisms and the numbers of disease-causing agents are reduced significantly, primarily due to the long detention period required for stabilization. In some types of pond, aerobic conditions can be maintained by the natural photosynthetic processes of algae. These green plants provide most of the oxygen required for aerobic stabilization. The remainder of the oxygen is transferred from the air to the water by natural surface mixing processes.

An inspection of costs will show that it is considerably cheaper to treat waste-water in stabilization ponds than by other methods, provided land costs are not prohibitive. The cost of reducing the putrescible content of waste-water in waste stabilization ponds is usually less than half that of other methods of treatment (South Africa, National Institute for Water Research, 1965). Furthermore, the costs per caput of the population for small pond systems do not increase rapidly with decreasing size, as they do with other methods of treatment.

Types of Pond

Many different terms have been applied to the different types of waste stabilization pond, including sewage lagoons, oxidation ponds, redox ponds, maturation ponds, facultative lagoons, anaerobic lagoons, aerobic stabilization ponds, and mechanically-assisted oxidation ponds.

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[1] The concept of biochemical oxygen demand is explained on p. 51.

For the purposes of this monograph, ponds are defined as follows. The term *waste stabilization pond* is used to describe any pond or pond system designed for biological waste treatment. A pretreatment *anaerobic waste stabilization pond* is essentially a digester that requires no dissolved oxygen, since anaerobic bacteria break down the complex organic wastes. An *aerobic waste stabilization pond* is one in which aerobic bacteria break down the wastes and algae, through photosynthetic processes, provide sufficient oxygen to maintain an aerobic environment. A *facultative waste stabilization pond* is one in which there is an upper aerobic zone (maintained by algae) and a lower anaerobic zone. Aerobic, facultative, and anaerobic organisms might be found in a facultative waste stabilization pond. A *mechanically aerated waste stabilization pond* is one in which mechanical aerators either supplement or replace algae as a means of providing the required dissolved oxygen. This type of pond may function as an aerobic or facultative system. In some mechanically aerated ponds the turbulence may not be sufficient to keep all solids in suspension; therefore, sludge may settle and undergo anaerobic decomposition while most of the pond remains aerobic.

Ponds receiving untreated waste-waters are referred to as raw or *primary waste stabilization ponds*. Ponds receiving effluents from primary settling tanks or secondary biological treatment units are called *secondary waste stabilization ponds*. Similarly, a second or third pond in a series of ponds functions as a secondary facultative or aerobic treatment unit. A pond whose primary function is to reduce the number of disease-causing microorganisms through extended detention time is called a *maturation pond*. A maturation pond may also be used to rear fish such as carp and it may then be termed a fish pond. The physical layout and mode of operation can also be used to categorize a pond system. Ponds may be designed to operate singly, in series, or in parallel.

Most waste stabilization ponds, as at present used, are facultative treatment units. In this respect, they resemble rivers and lakes. Aerobic conditions are maintained near the surface and sometimes throughout most of the depth of the pond. However, an anaerobic environment persists near the bottom, where there will always be some settled organic debris.

History of Pond Development

Ponds have been used for centuries to store and treat animal and household wastes. However, only within the last two decades have specific design criteria been developed in terms of volumetric requirements, organic loading rates, and detention periods.

In 1901 the city of San Antonio, Tex., constructed an impoundment of 275 ha with an average depth of about 1.4 m. This pond, now known as Mitchell Lake, is still in use. Following this successful experiment, other cities in Texas, California, North Dakota, and elsewhere in the USA used ponds as a means of treating sewage (Caldwell, 1946). However, the use of ponds prior to the last two or three decades seemed to be more by accident than design (Giesecke & Zeller, 1936; Pearse et al., 1948). As an example, in 1924 the city of Santa Rosa, Calif., attempted to avoid the cost of a waste-water treatment plant by uncovering gravel beds which, it was believed, could be used as natural filters prior to discharging waste-water into the then highly polluted Santa Rosa Creek (Gillespie, 1944). As might be expected the gravel bed became partially sealed, resulting in an impoundment of sewage to a depth of about 90 cm. The effluent from this pond was reported to resemble that from a trickling filter. The first pond in North Dakota was put into operation in 1928 because there was no nearby stream to dilute or even carry away the waste-water collected by the newly constructed sewer system. It was decided to empty the waste-water into a natural depression some distance from town, in the hope that this would prevent odour problems. This pond remained in operation for over 30 years (Svore, 1961). During the second world war, significant operating experience was gained at military installations in the USA (Mohlman et al., 1946).

Little engineering or research went into the construction of early ponds, some of which failed. The geometry and loading of the pond varied with the "lie of the land". Receiving-stream requirements were frequently not considered. Finally, however, the acceptance of ponds in the southwestern USA brought a semblance of design into being and the operation of these ponds has been steadily improving. A similar series of events occurred in Europe (Imhoff, 1926).

It seems that the first use of a pond system specifically designed to treat raw waste-water occurred in North Dakota (Van Heuvelen & Svore, 1954). This pond received the unconditional approval of the State Health Department in 1948. By 1960 there were over 100 such installations in North Dakota and about 300 pond systems, mostly of the secondary treatment type, in Texas.

Following a period of field studies in 1940-50, the development of rational design criteria for pond systems was undertaken. After 1950 significant research and field data began to appear in the literature (Gotaas et al., 1954; Hermann & Gloyna, 1958; Wennstrom, 1955; Towne et al., 1957; Parker et al., 1959), and many reviews of such literature are available (e.g., Fitzgerald & Rohlich, 1958; US Public Health Service, 1961a, 1961b; India, Central Public Health Engineering Research Institute, 1964). By 1962, there were 1647 stabilization ponds in use in the USA for the treatment of municipal wastes (Porges & Mackenthun, 1963) and possibly an equal number for the treatment of industrial or agricultural wastes (Porges, 1963). Successful experiences have been reported in Australia (Parker, 1962); New Zealand (Collom, 1965); Israel (Wachs et al., 1961; Watson, 1962); Brazil (Azevedo-Netto, 1967); South Africa (Shaw, 1962; Stander & Meiring, 1962;

South Africa, National Institute for Water Research, 1965); India (India, Central Public Health Engineering Research Institute, 1964); and Canada (Fisher et al., 1968). Plants have been operated successfully in arctic regions such as Alaska and northern Canada with the assistance of aerators (Reid, 1965; Pattison, 1966).

Within the last few years a number of design recommendations have been made by various health departments in the USA. These have been in the form of suggested loading or detention[1] standards. For example, design criteria have been adopted by the 10 Missouri Basin states (Van Heuvelen et al., 1960) and the Oregon State Board of Health (1965). Some states in the southwestern USA recommend an upper BOD_5 loading[2] of 56 kg/ha per day until better design experience becomes available for a given waste and location.

Basic research on waste stabilization ponds has also grown rapidly in recent years. Pond designs based on laboratory and field experiences have shown increasing approximation (Oswald, 1963; Marais, 1966; Hermann & Gloyna, 1958). Significant studies on bacteria reduction in ponds (Coetzee & Fourie, 1965; Yousef, 1962), operational practices (Ullrich, 1967), and the toxicity of industrial wastes (Gloyna & Espino, 1967; Huang & Gloyna, 1968) have produced basic data for better designs and operation.

It is the opinion of a growing number of engineers that waste stabilization ponds have undergone sufficient study and development to be classified as one of the major types of waste-water treatment system.

Modes of Operation

It is difficult to classify ponds by wastes received, size, shape, mode of operation and treatment objectives; however, some typical modes of operation are discussed below.

Where BOD reduction is a major consideration, the practice is to use either a combination of anaerobic and facultative waste stabilization ponds or facultative ponds independently. However, when it is important to reduce the numbers of pathogenic organisms, series-connected ponds produce the best results. A series-connected system might include anaerobic, facultative, and maturation ponds or the latter two types only.

The layout and mode of operation will depend upon the objectives and the degree of flexibility required. Series design is usually used where the organic load is great and where it is desirable to reduce the coliform count. Parallel systems are used where it is desirable to provide considerable flexibility of operation. Recirculation, although rarely used, provides a

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[1] Throughout this monograph, the term "detention" is applied to liquid, and the term "retention" is applied to solids such as grit and sludge.

[2] BOD_5 (biochemical oxygen $demand_5$) is the biochemical oxygen demand as determined by standard laboratory procedures for 5 days at 20°C ± 1°C.

means of bringing some of the oxygen-rich waters from near the effluent area to the part of the system where oxygen must be supplied at the greatest rate.

Wastes containing large amounts of solids and those containing toxic or coloured substances require special treatment. In contrast to domestic waste-water, each industrial waste must receive special attention.

The biological processes are controlled essentially by the detention time and the temperature. For ideal operation, it is usually desirable that the effluent and influent flow rates be equal. Although an imbalance in flows will not destroy the system, excessive percolation and evaporation may exert a powerful influence on a waste stabilization pond system. Since inorganic solids may accumulate in ponds from which extensive evaporation occurs, special attention must be given to the design of systems in arid regions. Similarly, excessive percolation may adversely affect nearby groundwaters and reduce the treatment efficiency of a pond.

The maturation pond is becoming an integral part of waste-water treatment systems in South Africa (Van Eck, 1961). In terms of *E. coli* count the effluents from these ponds are comparable with those obtainable by the chlorination of sand-filtered effluents (Stander & Meiring, 1962).

Waste stabilization ponds are increasingly being built in the USA to receive the effluents from overloaded biological waste-water treatment units. These ponds are designed to improve the effluents of activated sludge plants, biological filters, anaerobic and facultative ponds, etc. Usually, the objective is to prepare the water for reuse by reducing the BOD.

In several countries increasing attention is being given to the recharging of groundwater with treated waste-water. In Israel, where waste-water treatment in stabilization ponds has considerable engineering and economic advantages over treatment in conventional plants, plans have been developed for the possible use of this method for a city having a population of more than 1 000 000. The design recommendations include the use of both anaerobic and facultative ponds (Amramy, 1965). The effluent would be pumped to spreading basins and the reclaimed water would be withdrawn through wells.

At Santee, Calif., effluent from an activated-sludge treatment plant is detained in a waste stabilization pond for 30 days; the waste-water is then chlorinated at a level of about 15 mg/litre and is permitted to travel about 2.5 km through a natural sand and gravel formation 3–4 m thick. The water is collected in the first of two recreational lakes (Askew et al., 1965). During the test period from March to November 1963, all samples of the raw sewage, primary effluent, and activated sludge effluent were found to contain 1 or more of 13 different viruses. During the same period, only 25 % of the samples of the effluent of the stabilization ponds were found to contain viruses. Samples collected from the recreational lakes have been consistently negative for virus. These results confirm other California studies that have indicated that travel of enteropathogens with percolating water is not always a critical factor in determining the method of waste treatment and the feasibility of recharge.

In a number of areas night-soil or wastes intermediate between night-soil and raw waste-water (i.e., wastes containing more water than night-soil but less water than that obtained from a water carriage system) must be treated, and such treatment can be carried out in waste stabilization ponds. Tentative results indicate that ponds receiving night-soil and conserving tank effluents will remain aerobic if the loading does not exceed the contribution from 3 000–5 000 persons per hectare per day and if the minimum depth is about 1 m (Shaw, 1962). Evaporation and seepage losses must be compensated for if ponds of this type are to continue operating.

Instructions for the design of farm ponds, particularly for the treatment of hog wastes, are now available for some US farming communities (Bay, 1962). It is reported that there are over 200 ponds for the treatment of hog wastes in the State of Missouri alone. Unfortunately, few data are available on the performance of these farm systems.

CHAPTER 2

Extent of Waste Stabilization Pond Usage

This chapter briefly discusses the results of a survey of pond usage, present practices in some countries that have reported operational experience, and costs.

Worldwide Survey

From 1964 to 1967, WHO carried out a survey of the extent to which ponds were being used for waste-water treatment and of the problems associated with their operation. Through the assistance of the WHO Regional Offices, ministries of health, and local health establishments, it was possible to assemble a vast amount of design and operational data.

The information obtained from this survey and other sources showed that waste stabilization ponds are in use in at least 39 countries:

Argentina [1]
Australia
Bolivia
Brazil
Canada
Colombia
Costa Rica
Cuba
Ecuador
Federal Republic of Germany
Finland
German Democratic Republic
Ghana
Guatemala
India
Israel
Japan
Kenya
Mauritius
Mexico
Netherlands
New Zealand
Nicaragua
Nigeria
Pakistan [1]
Peru
Romania
Saudi Arabia
South Africa
Southern Rhodesia
Sweden
Thailand
Trinidad and Tobago
Uganda
Union of Soviet Socialist Republics
United Arab Republic [1]
United States of America
Venezuela
Zambia

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[1] Experimental ponds.

Such ponds are in use from the polar areas to the equator. Small hamlets that have recently received the benefits of a water supply, as well as some large metropolitan areas, find waste stabilization ponds both economical and practical. Some typical results are summarized below. However, this summary does not cover North American ponds, which are too numerous and in any case are discussed in some detail on pp. 31–39.

As might be expected, the organic and volumetric loadings were found to vary considerably. Some of the pretreatment anaerobic ponds, especially those receiving industrial wastes, receive several hundreds or even thousands of kilograms of BOD_5 per hectare per day.

The removal efficiencies for comparable areas and loadings seem to be fairly uniform throughout the world. It is not uncommon to obtain better than 90 % BOD removal in waste stabilization ponds.

Excluding North America, reports were received from the operators of 49 plants. Of these, 28 % reported no problems, 16 % indicated occasional or frequent odours, 5 % noted problems with chironomid midges, 4 % were concerned with flies, and 4 % reported mosquito problems. Of those reporting, 36 % indicated that the most prevalent odours occurred during the summer. This is indicative of the geographical location of most of these ponds (i.e., they are probably located in regions where there is no ice cover in the winter and consequently odours associated with a spring readjustment period do not occur). Some operators have controlled all these nuisances by well-planned maintenance programmes.

Of the 49 plants that reported effective operational control of various problems, 50 % maintained some control of mosquitos, 40 % were active in removing accumulated scum, and 31 % practised erosion control of the embankments. About 65 % of the ponds were fenced.

The populations served by the ponds varied from less than 1 000 to several hundred thousand in number. The manpower requirements were most significant: the man-hours per month needed for plants serving populations in the range 1 000–10 000 varied from 2 to 180. It would appear that manpower requirements are wholly dependent on the social and economic status of the area.

The design criteria varied considerably. Some countries were concerned with the removal of BOD, coliform and other micro-organisms, and suspended solids. Other countries based designs on BOD removal only, and these designs usually specified depth, surface area, and organic loading. Colour, dissolved oxygen, and certain components of industrial wastes were also specified as important design criteria by some countries.

Almost all the plants on which reports were received had the ability to carry out some laboratory analyses or could call upon a central authority for such service. Excluding North America, 38 plants in 10 countries kept records. Of these plants, 94 % measured pH, 79 % analysed for BOD, and 68 % measured dissolved oxygen. The MPN index (most probable number of coliform organisms per 100 ml of sample) was determined by only 19 plants in 7 countries.

The physical arrangements and auxiliary equipment varied greatly. The majority of the ponds were constructed by cut-and-fill methods. The preferred shape seemed to be rectangular, and the most common depth was 1–2 m. On the basis of surface area, the ponds fell into 3 approximately equal groups: those of less than 4 000 m^2, those of 4 000–20 000 m^2, and those larger than 20 000 m^2. The detention time was generally greater than 10 days. About 50 % of the plants had no type of flow-measuring device. Most of the inlets were constructed at the edge of the pond, although some were located at the centre.

Since it is impossible to describe in detail all the plants and the special designs used in all the countries, a few countries representing a geographical distribution, certain designs, and unique modes of operation have been selected.

India

India, with more than 35 waste-water treatment ponds, has been working for the past decade through the Central Public Health Engineering Research Institute, Nagpur, on different aspects of waste-water treatment by ponds. The climate in India—considerable sunshine and generally warm weather—is ideal for such treatment. The solar radiation is apparently of the order of 250–500 langleys per day, according to season. Loadings of 22–440 kg BOD_5/ha per day may be used, depending upon the climatic conditions. A pond area of 1 ha is sometimes used for each 5 000–10 000 people. The recommended pond depth is 1–2 m.

Table 1 shows that the BOD removal efficiency is usually well above 85 %. However, it must be assumed that many of the ponds are anaerobic. The cost of a pond is about 7–10 % of the cost of conventional alternatives. Fig. 1 shows the relationship between cost per caput, population served, and latitude. The cost data would apply to many developing countries. The annual maintenance costs of ponds in India are 3–5 % of the construction costs. It may be noted that mechanical parts are more costly in developing countries, and maintenance of the parts is both costly and difficult owing to the scarcity of skilled workers.

Israel

Since 1960 detailed studies have been made of several waste stabilization ponds in Israel (Wachs et al., 1961; Watson, 1962). During the decade preceding 1960, more than 60 waste stabilization ponds were built, the construction costs being only 5–20 % of those of conventional biological waste treatment plants, excluding land costs.

TABLE 1

MONTHLY AVERAGE DATA SHOWING BOD REMOVAL EFFICIENCY OF PONDS AT BHANDEWADI, NAGPUR, INDIA *

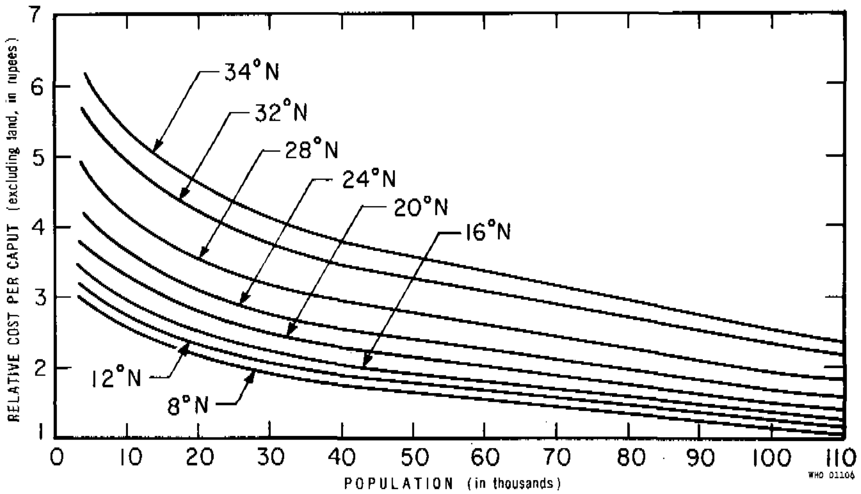
Month	Detention period (days)	BOD of raw sewage (mg/litre)	BOD of effluent (mg/litre)	Load (lb BOD_5/ac per day)[a]		BOD reduction (%)
				Applied	Removed	
1965:						
March	4.6	113	9.6	590.0	540.0	91.0
April	4.6	115	6.0	596.0	565.0	94.8
May	5.6	70	12.0	298.0	247.0	82.9
June	4.9	80	15.0	387.0	315.0	81.3
July	5.0	210	28.0	1 000.0	865.0	86.5
August [b]	3.9	235	60.0	961.0	716.0	74.5
September [b]	4.7	235	58.0	785.0	591.0	75.3
October [b]	5.0	264	63.0	832.0	633.0	76.1
November [b]	4.7	272	57.0	923.0	730.0	79.0
December	4.5	149	37.3	778.0	583.0	75.0
1966:						
January	5.7	252	37.0	1 028.0	877.0	85.3
February	4.5	245	28.0	1 281.0	1 135.0	88.5
March	3.9	418	34.0	2 800.0	2 389.0	85.3
April	5.6	417	33.0	1 766.0	1 626.0	92.1
May	5.5	351	36.8	1 509.0	1 350.0	89.4
July	6.7	282	22.0	1 007.0	928.0	92.2

* Reproduced, by permission, from Mehta (1966).

[a] 1 lb BOD_5/ac per day = 1.1 kg BOD_5/ha per day.

[b] One pond was out of commission for repairs.

FIG. 1. COST OF STABILIZATION PONDS IN INDIA IN RELATION TO SIZE OF POPULATION SERVED AND TO LATITUDE*



* Data from Arceivala & Desai (personal communication, 1968).

These experiences have been summarized by Watson (1962, pp. 35–36):

> Experience shows that after primary treatment the BOD of domestic sewage usually varies from 0.08 lb to 0.12 lb per capita. A BOD of 250 p.p.m. can be reduced after 5 days in a pond to 50 p.p.m., and to about 30 p.p.m. after a further 5 days' detention.
>
> *Primary treatment more than $^1/_4$ mile from nearest house*
>
> Open earth basins built in duplicate or triplicate have the following properties:
> Detention: 20 hours after 6 months' operation
> Reduction of BOD: 40 %–55 %
> De-sludging: Annually
> Depth of basin: 8 ft excluding freeboard
> Average depth of sludge after 1 year: 3 ft
> Average depth of scum after 1 year: 16 in.
> Average depth of liquid after 1 year: 3 ft 8 in.
>
> *Oxidation pond, first stage*
>
> Loading: Maximum 4 000 persons per acre, which equals about 250 lb BOD/acre/day.
> Detention: 5 days at constant depth.
> Depth: 3 ft 6 in. + 3 ft freeboard (possibly 4 ft 6 in. in summer).
> Cleaning: No cleaning so far. Provision must be made to drain a section of the pond without upsetting the operation of the plant.
> Storage reservoir: Necessary, as pumping direct from first stage oxidation pond is undesirable.
> Overflow: Necessary from oxidation pond and from reservoir.
>
> *Oxidation pond, second stage*
>
> At a somewhat reduced loading and with detention for a further 15 days in aerobic oxidation ponds, an effluent can be produced which will be little, if at all, inferior to full treatment in the conventional activated sludge or high-rate bio-filtration plant. A good effluent requires that the depth of the pond does not fluctuate.
>
> A compromise was found by using the storage reservoir for second-stage oxidation. The depth will naturally fluctuate, depending on the extent of use. The design of the reservoir is based on at least 5 days' additional detention, if the reservoir is to be thought of as an oxidation pond where the algae will continue their purification process. More often the reservoir provides 1 day's storage and is in use only during the summer months.[1]

In 1958 the Ministry of Health established tentative guide-lines for the design of waste stabilization ponds whose effluent is to be used for agricultural irrigation. The minimum detention periods required were 18–24 hours in first-stage ponds and 5 days in aerobic[2] second-stage ponds, with a BOD of 20–25 kg/1 000 m^2 per day. Most early ponds in Israel were built to these requirements, although some were even more heavily loaded (H. I. Shuval—personal communication, 1967). Recent studies have shown that it is desirable to reduce the BOD loading of ponds to a maximum of 13–15 kg/1 000 m^2 per day in order to overcome anaerobic conditions during

---

[1] Quoted by kind permission of the Council, Institution of Civil Engineers, London, England.

[2] According to the definitions used in this monograph (see p. 15), these ponds would be regarded as facultative.

the winter months. Recent practice in Israel has tended towards increasing detention periods and reducing loads to this level in order to provide a greater safety factor in pond design (Samsonov, 1965).

The key to successful operation and maintenance has been simplicity in design and careful attention to the details of maintenance. For example, primary pretreatment ponds must be desludged periodically, repairs must be made to dikes and other structures, and weeds must be cut.

Recent reports indicate that thought is being given to deeper ponds and to the problem of maintaining an effluent that is chemically suitable for irrigation. The greater depth assists in the maintenance of a suitable anaerobic-facultative environment; it also reduces evaporation losses, resulting in a lower salinity of the effluent, which is consequently of better quality for irrigation.

Experience with the extra-deep pond at Ashkelon indicates that when loadings acceptable for Israel's climate are used during the warm months, a pronounced thermal gradient causes the upper part of the pond to behave as a conventional facultative pond "floating" on an anaerobic one (Wachs & Berend, 1968). Such stratification is enhanced by the fact that both the inlet and the outlet are located at the surface.

During the warmer months effluent with a lower concentration of algae can be obtained through an outlet at a depth of 1 m, although such an effluent is deficient in dissolved oxygen at all times.

For anaerobic-facultative pond systems, which are popular in Israel, the construction costs (excluding land costs) are roughly US$ 9 000–45 000 per 1 000 m^3 per day (A. Amramy—personal communication, 1966).

Central and South America

It has been reported that ponds in Brazil, Colombia, Costa Rica, and Peru have performed well with no nuisances at loadings of 3 000 population equivalents[1] per ha (230 kg of BOD_5/ha per day) (E. Ribeiro—personal communication, 1966). It appears that anaerobic ponds can be loaded at rates of 6 000–15 000 population equivalents per ha. In Brazil the facultative ponds have been designed on the basis of 100 kg of BOD_5/ha per day, which represents 1 750 population equivalents per ha. For systems consisting of an anaerobic pond followed by a facultative pond, the designs of the two ponds are based on 720 and 100 kg of BOD_5/ha per day, respectively.[2] The average waste-water flow is about 185 litres *per caput* per day (Azevedo-Netto, 1967). Some plants in Costa Rica have been designed on the basis

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[1] "Population equivalents" are sometimes used to express loads of putrescible organic matter and suspended solids, particularly when non-domestic sewage is involved. Frequently a load of 54 g of BOD and 90 g of suspended solids *per caput* daily is used to represent one population equivalent.

[2] Strictly speaking, it is incorrect to refer to anaerobic pond design on the basis of area (i.e., in terms of BOD/ha per day), but the system is used in this monograph in order to compare the capabilities of facultative and anaerobic waste stabilization pond systems.

of 55–83 kg of BOD_5/ha per day. In terms of population this represents a load of the wastes from 1 000–1 600 persons/ha. The average sewage flow is about 0.4 m^3 *per caput* per day. The operating depth of the ponds is about 1 m (F. Saenz, unpublished data, 1965).

South Africa

Extensive work has been carried out by the South African Council for Scientific and Industrial Research (Stander & Meiring, 1965). Developments in the design of maturation ponds, primary waste stabilization ponds, facultative waste stabilization ponds, and ponds for the treatment of night-soil and conserving tank effluent are well advanced.

The maturation pond is used to reduce the enteric bacterial count in the effluent from facultative ponds and conventional biological treatment plants. Effluents from humus-tanks,[1] sand-filters, or other treatment systems are allowed to flow at a continuous predetermined rate into maturation ponds, which improve their biochemical and bacteriological quality. The factors that affect the performance of maturation ponds are the extent of mixing, the amount of oxygen that is introduced, the intensity of solar radiation, the temperature, short-circuiting, and the loading. The extent of mixing is dependent on the wind, density stratification, and pond turnover resulting from changes in surface temperature and influent arrangements. Oxygenation is primarily dependent, as in other ponds of this type, on the amount of algae present. In addition to supplying heat and energy for photosynthesis. solar radiation may contribute to the reduction of bacteria. Temperature appears to be one of the most important factors affecting the performance of maturation ponds. It is also noteworthy that the total quantity of dissolved solids is reduced in maturation pond systems. Ponds can be designed according to a formula that ensures that the pond will remain aerobic and that BOD and, more important still, the enteric bacteria content are reduced to a satisfactorily low level.

Shaw et al. (1962) suggest the following criteria as guides for the design of primary waste stabilization ponds in South Africa:

General principles

Topography : For the sake of economy, advantage should be taken of topographical features to reduce the cost of constructing stabilization ponds. Investigations should, therefore, start with the selection of the most suitable site.

Pond arrangement : A single pond will generally not provide a satisfactory effluent and is, therefore, not recommended. Three or four cells in series are normally recommended, depending on the quality of waste being treated and the effluent quality desired.

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[1] *Humus* is the solid material left after biological treatment of waste-water (sewage), when decomposition has advanced to the point where the original form of the material can no longer be distinguished.

Flexibility : In view of the scant knowledge of all the factors affecting pond performances, a flexible system should be planned, i.e., units initially designed for operation in series should, if possible, have provision for conversion to units in parallel, and provision should be made for extension of both primary and secondary ponds. This will enable operating experience gained *in loco* to be used to the best advantage in future extensions.

Character of influent

Domestic effluents : Ponds will effectively stabilize raw sewage, settled sewage, septic tank effluent, as well as partly treated sewage effluent.

Industrial effluents : Where industrial effluents are accepted into sewers and mixed with domestic sewage, due regard must be given to the possible toxic or other detrimental effects on the biological processes in the ponds.

Primary pond loading

As a general guide a loading of approximately 145 lbs BOD per acre per day can be adopted for domestic raw sewage in a four-foot deep pond. This is approximately equivalent to 1 200 persons per acre,[1] or 40 000 gallons per acre per day, with normal domestic sewage. For exceptionally strong sewages, or for effluents from aqua privies, septic tanks or settlement tanks the loadings can be estimated...

Arrangement of secondary ponds

The retention time in the secondary ponds should be approximately twenty days total. The secondary pond area should be divided into 3 cells in series, having detention times of ten days, five days and five days respectively. Where topography is favourable to a different subdivision of area, this may be used but the minimum retention in any pond should be at least five days.

Physical features

Depth : Ponds should have a mean depth of three to six feet; the depth need not be uniform but limits of three feet minimum to eight feet maximum are suggested.

Inlet : Discharge into primary should be on bottom of pond to prevent thermal short-circuiting. Influent velocity should not be directed towards outlet. If this is unavoidable, a baffle should be provided to dissipate incoming velocity.

Outlet : The outlet draw-off should be provided with a scumboard to prevent surface short-circuiting and to prevent wind-blown scum going out with effluent.

Embankments : Slopes of embankments should be dictated by normal engineering practice for small dams. Details at fringes should be designed for preventing ingress of vegetation. Capital investment on weed-prevention, by stone pitching or using soil-cement at verges may well be repaid by savings on maintenance.

Pond bottoms : Bottoms may be level or graded, but hollow pockets should be avoided.

Pond shape : The shape of the ponds will normally vary with topography. The main consideration is the siting of inlets and outlets so as to avoid short-circuiting or dead spots.

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[1] For metric conversion factors, see Annex 3.

Measuring devices

Some form of flow measuring and/or recording device should be installed ahead of the primary pond. Besides providing a check on the pond loading, this will furnish valuable data for use when the pond system is extended. A weir or similar device should also be installed, where possible, between ponds and at the final outlet, to afford a check on seepage losses.

Coarse screens and detritus channels

It is recommended that coarse screens [1] and detritus channels be installed ahead of stabilization ponds.

Evaporation and seepage

Evaporation is normally unimportant as it will generally not exceed $\frac{3}{16}$″ per day.[2]

Initial seepage will normally be reduced by bacterial and organic clogging. An average value for seepage after a year would be about $\frac{1}{2}$″ per day.

Groundwater contamination from seepage: A careful investigation should be made to assess possibility of dangerous groundwater pollution by seepage from ponds.

Sludge accumulation

At the loadings recommended for raw sewage, sludge accumulation is unlikely to exceed 18″ in 5 years.

Mosquito breeding

To prevent mosquito breeding all verges must be kept free of vegetation.

The anaerobic-facultative pond system, whereby the conventional facultative pond is preceded by an anaerobic pond, is also receiving considerable attention. Effluents from the facultative pond are recirculated and mixed with the sewage entering the anaerobic pond. This recirculation appears to control odours. Unfortunately, during the winter months when the temperature varies between 15°C and 20°C, BOD removal decreases and an unsightly thick scum that promotes fly-breeding forms on the anaerobic ponds. However, this problem can be overcome by increased maintenance.

The use of ponds for treating concentrated wastes such as night-soil has been studied in considerable detail (Shaw, 1962). For climatic conditions similar to those of Pretoria, a pond loading of wastes from 2 740 persons/ha per day appears suitable. In South Africa the organic concentration is higher and the volume of water lower than in geographical areas where water is abundant and water carriage systems are predominantly used. The pond must be filled with water, and liquid must be added periodically to offset evaporation and seepage losses. The recommended

---

[1] Because of maintenance problems, many authorities do not recommend any form of screen.

[2] Excessive evaporation and seepage can present a problem in certain arid areas.

depth is 60–150 cm. Of considerable importance is the fact that the sludge accumulation is only about 4 cm per year. There has been no evidence of the breeding of flies or mosquitos in these special ponds if they are well maintained, but the degree of maintenance required is high.

Concrete approach ramps must be provided to ensure that all dumped material is washed into the pond. A pump should be available to assist with washing down the ramps and breaking up floating solids.

Europe

In Europe, as in China, some form of waste stabilization pond was in use long before modern times. It appears that fish ponds were built by the early Greeks at Agrigantum, Sicily. During the nineteenth century fish ponds were used throughout central Europe with considerable success (Hickling, 1962).

Aerobic and facultative waste stabilization ponds have been used extensively in various European countries for treating wastes from small communities. This type of waste-water treatment with recirculation has proved most successful in Sweden (Wennstrom, 1955). For the most part, operating data of European facilities have not been used in empirical design equations.

Various kinds of anaerobic pond have been used to treat beet-sugar mill wastes, paper-mill effluents, and domestic sewage (F. Pöpel—personal communication, 1968).

Australia

Some of the first detailed studies on the treatment of raw waste-water were undertaken near Melbourne (Parker et al., 1950). These studies led to the construction of "lagoons" (waste stabilization ponds) for two-stage treatment. Since 1947, 28 000–90 000 m^3 of raw sewage has been treated in this way. Much of the first pond, which receives the solids, is essentially anaerobic, and this results in removal of about 70–85 % of the BOD, compared with the removal of 30–40 % that might be expected from solids removal alone. The destruction of organic matter, as measured by changes in volatile solids, is shown in Table 2. In the absence of digestion and assuming that 7.5 % (by weight) of the sludge consists of solids, an accumulation of 1 cm represents 7 200 kg/ha and a rate of accumulation of 1 cm per year represents a suspended solids loading of 19.3 kg/ha per day.

Where high-rate anaerobic ponds receive raw waste-water and the solids loading is of the order of 500 kg/ha per day in the first pond, the rate of build-up of solids is substantial. The solids deposition in two Australian ponds is given in Tables 3 and 4. This build-up raises the question of whether, when, and how the accumulated solids should be removed.

Experience at Werribee indicates that at installations using several ponds in series, effective operation can be continued until the sludge layer occupies at least three-quarters of the pond depth. The sludge layer can be dried only during the hot summer months. The semi-dry sludge may be pushed out by a bulldozer. In smaller installations, sludge has been pumped out wet.

TABLE 2

COMPOSITION OF SLUDGE SOLIDS IN AN ANAEROBIC POND, MELBOURNE, AUSTRALIA *

Characteristic	Raw sludge	Lagoon sludge
Total solids (by weight)	5.3 %	14.9 %
Moisture (by weight)	94.7 %	85.1 %
Volatile solids (by weight)	74.0 %	38.3 %
pH	5.8	7.5

* After Parker (1966).

TABLE 3

SOLIDS DEPOSITION IN EXPERIMENTAL ANAEROBIC PONDS, BACCHUS MARSH, AUSTRALIA *

Characteristic	Pond 1	Pond 2	Pond 4
Average inflow (UKgal/d) [a]	3 500	1 000	4 000
Solids content of inflow (mg/l)	323	323	323
Period of inflow	18 months	18 months	18 months
Solids added (lb/d) [a]	5 660	1 615	6 460
Depth of sludge (in) [a]	7	7	29
Solids content (by weight)	2.49 %	2.79 %	4.81 %
Total sludge (lb dry weight)	182	204	125
Sludge solids accumulated (by weight)	3.2 %	12.1 %	1.9 %

* After Parker (1966).

[a] For conversion factors, see Annex 3.

TABLE 4

SOLIDS DEPOSITION IN A SERIES OF 7 ANAEROBIC PONDS, EACH APPROXIMATELY 10 ACRES (4 HA) IN AREA, WERRIBEE, AUSTRALIA *

Characteristic	Results [a]
Average inflow	4 500 m^3/day
Solids content of inflow	400 mg/l
Period of inflow	5 years
Solids added	4 500 lb/day; 1 640 000 lb/year; 8 200 000 lb over 5 years
Depth of sludge, 1st pond	24 in
Total sludge accumulation in all ponds	3 170 000 lb
Sludge solids accumulated in all ponds	37 % by weight

* After Parker (1966).

[a] For conversion factors, see Annex 3.

Anaerobic ponds are not recommended for highly saline and mineralized waste-waters, since massive sulfide generation occurs in the first lagoon and normal methane fermentation is completely inhibited. It is true that organic matter is destroyed even in the absence of methane fermentation, since it is used for sulfate reduction, and BOD is thereby removed from the supernatant water; but the resulting high concentrations of sulfide impose an additional load on the subsequent anaerobic units.

New Zealand

A number of small ponds are being used in New Zealand, but a system that merits special attention is the Manukau Sewerage Scheme, constructed by the Auckland Metropolitan Drainage Board (Collom, 1964).

The size of the pond complex is 530 ha. Provision has been made for primary settling, facultative pond treatment, partial anaerobic digestion, and sludge "lagoons" (stabilization ponds). The 26-ha sludge lagoons are operated in a manner similar to facultative waste stabilization ponds. After the sludge has been digested to a level where about 93 % of the available methane has been collected, the remaining sludge is pumped to one of three ponds about 3 m deep. This is done in the morning to take advantage of photosynthetic oxygenation. The solids settle, and the liquid above the sludge is dark in colour and contains algae and oxygen. In 1963 the minimum loading was 224 kg BOD_5/ha per day, while the weight of dry solids put into the ponds was about 3 900 000 kg. The rate of build-up of sludge has proved significantly lower than expected. The residual solids appear to be uniformly deposited over the pond bottom, and are then either brought into solution or re-suspended and passed on to the facultative ponds.

The waste stabilization ponds receive primary settled waste-water. BOD reduction has been most satisfactory, but *Chironomus zealandicus*, a non-biting midge indigenous to New Zealand that reproduces in shallow, slightly polluted ponds, has created a problem. It has been controlled by (*a*) increasing the organic load, which appreciably reduces the concentration of midge larvae (bloodworms), (*b*) applying certain chemicals around the perimeters of the pond, and (*c*) deepening the shallow areas of the pond.

United States of America

In 1962, 1 647 waste stabilization ponds were being used in the USA to treat municipal waste-waters (Porges & Mackenthun, 1963), and some 1 600 ponds were being used for industrial wastes (Porges, 1963). It is only within the last 10 or 20 years, however, that there has been a concerted effort to come to a better understanding of the critical design factors.

Industrial ponds

Porges (1963) prepared an inventory of waste stabilization pond practices in industry on behalf of the US Public Health Service. Detailed information was obtained regarding the type of facility, method of operation, pretreatment, efficiency, waste characteristics, and operational reports. The findings were summarized as follows (Porges, 1963, p. 467):

> No discharges were reported for 151 ponds. Land application of pond effluent was practiced at 18 installations. Supplemental treatment, such as aeration, screening, trickling filters, and the use of chemicals was indicated for 45 installations. Thirty-nine installations reported that the industrial waste was combined with sewage before treatment.
>
> The data indicates widespread use of 827 ponds by 31 industrial groups in the United States. The industry showing the greatest number of ponds is the canning group with 29 percent of the reported installations. Second is meat and poultry with 20 percent. Chemical industry is next with 7 percent.

As an example of industrial usage, the waste stabilization pond is the form of secondary treatment most widely used by the pulp and paper industry. At least 25 large mills and many smaller mills use this technique (Gehm, 1963; Gellman & Berger, 1968), operating at pond loadings from 11.4 to 345 kg BOD_5/ha per day. Loadings of 57 kg BOD_5/ha per day result in removal of at least 85 % of the BOD_5, and no odours develop. The use of ponds as pretreatment units (primarily for the physical separation of solids) followed by odour-free facultative ponds has met with equal success.

The National Council for Stream Improvement and its member pulp and paper mills have readily accepted the mechanically aerated waste stabilization pond. In 1966 there were existing and planned facilities for treating 2 440 000 m^3 waste per day from 26 mills by this technique. Both fixed and floating mechanical aerators of a wide range of sizes are in use. The efficiency of BOD removal can be varied from 50 % to 95 % by control of nutrients, air, detention time, and secondary clarification.

Many other industries are beginning to use mechanically aerated ponds. These ponds are frequently designed to provide preliminary treatment and are followed by facultative waste stabilization ponds. In general, surface aerators are used to provide the dissolved oxygen.

Domestic ponds

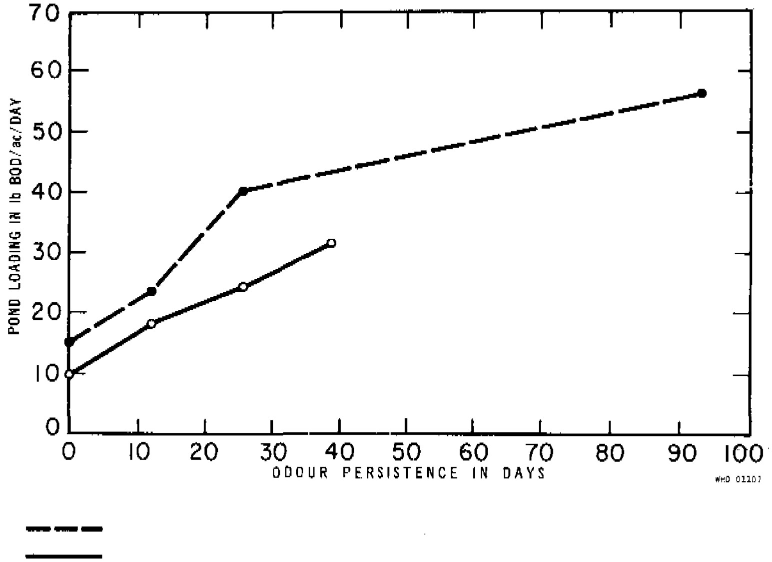
Since waste stabilization ponds are widely used to treat municipal wastewaters, only a few typical examples are given in this section.

In the southern and southwestern states of the USA the organic loading usually recommended is 56 kg BOD_5/ha per day. There is no problem of ice formation on ponds in this area, but long hot summers tend to promote excessive concentrations of blue-green algae, particularly when the water temperature exceeds 30°C. The floating "mats" caused by algal growths or by gasification of benthic sediments can be controlled by building deeper ponds and by wetting down the mats once or twice a week along the pond edges. The mats must be broken up before they have an opportunity to decay.

In Texas in 1966, there were 317 waste treatment plants employing ponds and several more with experimental ponds, primarily anaerobic waste stabilization ponds. The average organic loading to the facultative waste stabilization pond is 56 kg BOD_5/ha per day, based on primary settled waste-water. The detention time is roughly one month. Anaerobic pretreatment is not encouraged, for it is believed that the insect vectors of encephalitis may multiply in ponds that are septic. During the last 20 years or more there has been a great deal of experimentation, and the trend is towards greater pond depths ranging from 1.5 to 2.7 m.

An organic loading of 22.4 kg BOD_5/ha per day has been set as the design standard in the Missouri Basin, Upper Mississippi Basin, and the Great Lakes states. The principal reason for this low value is the need to control odours during the spring ice break-up. Odour control is the most important single item the design engineer in this area has to consider. As shown in Fig. 2, a loading of 11.2 kg BOD_5/ha per day with a depth of almost 1 m usually eliminates odours during the spring ice break-up (Porges,

FIG. 2. STABILIZATION POND LOADING IN lb BOD/ac/DAY* IN RELATION TO ODOUR PERSISTENCE AFTER BREAK-UP OF ICE



Reproduced, in modified form, from Svore (1968), p. 428, by permission of the publishers.
* 1 lb BOD/ac = 1.12 kg/ha.

1964). However, loadings of less than 22.4 kg BOD_5/ha per day may result in an insufficient supply of water during the hot, dry summer months. In all cases, evaporation losses must be considered.

The state of Michigan built its first waste stabilization pond in 1961, and seven years later 72 installations were in operation or planned. The ponds operate with a minimum of nuisance and provide effective pollution control (Pierce & Richmond, 1966). Treatment of waste-water by waste stabilization ponds has proved at least as effective as conventional secondary treatment with regard to removal or reduction of BOD, total suspended solids, and coliform organisms. Michigan has some highly porous soils, and clay admixtures have been used successfully for sealing the bottom and sides of ponds.

The state of Colorado (Horn, 1965) has divided defects of design, construction, and operation into three main categories: (1) liquid depth not maintained; (2) water surface area inadequate; and (3) settleable solids not evenly distributed.

The current practice of some midwestern and southwestern states is to use two ponds in series. The first pond is designed for loadings of 22.4–56 kg BOD_5/ha per day, while the second pond is considerably smaller in size. This system provides a degree of flexibility and is similar to the maturation pond system used in South Africa.

In western areas of the USA, many ponds are built as percolation and evaporation ponds. The advantages of this practice must be weighed against the disadvantages of a possible loss of a surface water resource.

Many states require disinfection of all domestic wastes. For example, Oregon requires disinfection of such wastes between May 1 and November 1 or as directed by the State Sanitary Authority. Consequently, some ponds are built with sufficient capacity to hold all wastes discharged during this period.

The State of Iowa, which has over 100 operational ponds, has found that ponds of over 2.5 ha each are more efficient in the winter because the larger size permits the wind to remove snow. A pond design based on 250 population equivalents per hectare is recommended for winter operations (Stouse, 1964).

In Alaska, experiments by the Arctic Health Research Center, US Public Health Service, have demonstrated that some of the disadvantages of pond operations in very cold climates can be overcome by the use of submerged aeration systems. Aeration permits higher loadings and shorter detention times (Reid, 1965). At detention times of 15–20 days and loading rates of 85–112 kg BOD_5/ha per day, the BOD removal rate stayed between 75 % and 80 %. Ice formation during the winter helps to conserve heat. The equipment and ponds have functioned at temperatures of —46°C. Fig. 3 shows the ice cover over an aerated sewage lagoon at Eielson, Alaska. The stalagmites are formed by the freezing of moist air from the pond.

FIG. 3. AERATION UNDER ICE COVER, EIELSON, ALASKA*

* Reproduced from Reid (1965) by courtesy of the Hinde Engineering Company, Highland Park, Ill., USA.

Canada

Ponds were first used for waste treatment in Canada in the prairie provinces. Now they are found in every part of the country, serving communities with populations ranging from only a few hundred to more than 200 000. The largest pond systems are located near Regina and Winnipeg. It is estimated that in 1966 there were about 500 waste stabilization facilities in Canada (Fisher et al., 1968).

The geographical location and size of Canada give rise to climatic extremes of temperature, humidity and solar radiation. Because these factors greatly affect the design and operation of ponds, no single design criterion can be applied (Stanley, 1959). The most commonly used criterion is the organic loading, usually expressed in terms of kg BOD/ha per day. To a lesser extent, designs are based on volumetric loadings, in which case a theoretical minimum detention time is used, together with an estimated or observed volumetric *per caput* flow rate. In some provinces both volumetric and organic loading factors are used to determine dimensions. The following description of ponds in the various provinces illustrates some of the special features used in the design of Canadian ponds.

Maritime provinces

The suggested design of waste stabilization ponds in the eastern provinces is based on organic loadings of 17–39 kg BOD_5/ha per day. Since most ponds are designed to serve small communities under moderately severe winter conditions, design procedures have been kept simple, and ponds are generally of the facultative type. The Atlantic Ocean keeps the climate of these provinces from becoming really severe, and only in a few cases is storage throughout the winter required. Data for spring and summer operations indicate removal rates of 85 % for BOD and 70 % for suspended solids. At most installations the reduction in coliform organisms appears to be in excess of 99 %. Offensive odours, if they occur at all, are observed during the spring thaws.

Most ponds in the maritime provinces have centre-discharging inlets located about 30 cm above the bottom. Variable overflow weirs are often used to maintain desired liquid levels.

Ontario and the prairie provinces

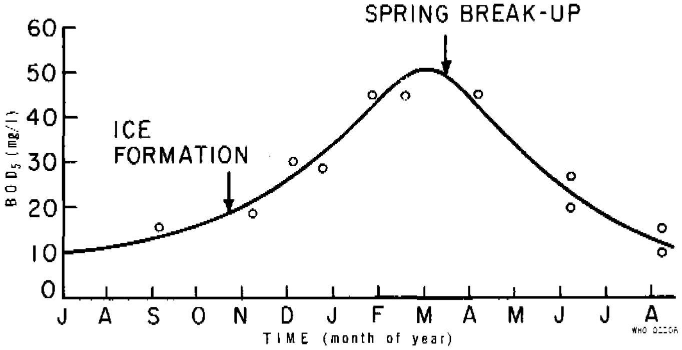
The criteria for pond design in Ontario are somewhat similar to those in the maritime provinces. Regulations of the Ontario Water Resources Commission require that the organic loading rate should not exceed 22 kg BOD_5/ha per day, or 250 population equivalents per ha per day.

In Manitoba, the recommended organic loading rate is 45–55 kg BOD_5/ha per day (500–620 population equivalents per ha per day). Higher organic loadings are permitted for installations that are operated only during the summer. The recommended depths vary between 0.6 and 1.5 m. Multiple ponds, either in series or in parallel, are recommended when the total pond area is greater than 4 ha, and provision must be made for a winter detention period of 120 days where there is a small dilution factor in the receiving stream. In Ontario many facilities are operated in parallel during the winter and in series during the summer. Multiple ponds may be used to avoid discharges during the summer months when the green effluent would adversely affect recreational areas.

The seasonal variation in effluent BOD is highly dependent on temperature. Fig. 4 shows BOD_5 values for summer and spring of 10 and 50 mg/litre respectively. An annual average BOD reduction of 90 % is reported, although annual figures for individual ponds range from 65 % to 96 %. Reduction of coliform organisms exceeds 99.5 % in some ponds, including those receiving raw (untreated) domestic waste-water.

At some plants that receive waters from both sanitary sources and storm sources, grit accumulations in the inlet works have created problems. In addition, silt banks have been built up in many ponds. At low depths of liquid, short-circuiting and odours have developed, and difficulties with weeds have been encountered at nearly half the ponds on which data are available. Erosion is a constant problem at some installations, and is generally associated with large surface areas or with dike walls having inside slopes greater than the recommended 25 %.

FIG. 4. SEASONAL VARIATIONS IN BOD OF EFFLUENT FROM WASTE STABILIZATION PONDS



Reproduced, in modified form, from Fisher et al. (1968), p. 443, by permission of the publishers.

It is recommended that the inlet pipe be located near the centre of the pond. For gravity flow, the inlet may be allowed to discharge horizontally. When the flow is under pressure, a vertical discharge is required, but the velocity must not exceed 60 cm per second. Adjustable overflow weirs are used to control depth.

On the prairies the summers are hot and dry, while the winters are extremely cold and long with a mean January temperature of about —10°C. In Saskatchewan, ponds consist of two units, operated in series with a total detention time of 180 days. The detention time is based on the full volume of the second-stage unit plus an amount equivalent to a depth of 60 cm in the first-stage unit. The first pond is designed for a maximum organic loading rate of 45 kg BOD_5/ha per day. However, most ponds have storage capacity for 4–6 months, so they can be partially emptied during the autumn prior to the winter freeze-up and again in the spring when dilution water is available. BOD removal rates are reported to exceed 90 % in summer and to average 54 % in winter. Throughout the prairie, reduction of coliform organisms in ponds is in excess of 99 %, an important factor because many of the ponds discharge across farmland and receive little or no dilution.

A diffused aeration system was constructed throughout the existing lagoons at Regina, Saskatchewan when the facility became grossly overloaded. The system consists of aeration tubing, weighted feeder tubing, and blowers. Results so far indicate a high degree of treatment, and the BOD of the treated effluent is well below the level of 40 mg/litre stipulated by the Department of Public Health. Temperatures as low as —40°C do not appear to halt the aeration process.

In the province of Alberta, facultative waste stabilization ponds are used extensively. The ponds are large enough to contain the waste inflow over a period of 6–18 months. Some facilities in this province use anaerobic pretreatment ponds with detention periods of 2–4 days, followed by second-stage ponds with detention periods of 3–6 months. The observed organic loadings of facultative ponds vary from 21 to 25 kg BOD_5/ha per day for single-unit systems and from 56 to 280 kg BOD_5/ha per day for multiple-unit systems. The loading rates for anaerobic installations range from 0.04 to 0.28 kg BOD_5 per m^3 per day. The BOD removal rates are between 45 % and 90 %, the lowest levels occurring during the colder periods. The anaerobic ponds are some 2.5–3.5 m deep, the secondary ponds 1.0–1.6 m deep.

The removal of suspended solids by anaerobic ponds during the winter has ranged from 64 % to 91 %. Where ponds are operated in series, more than 55 % of the suspended solids are removed in the first pond. In both Alberta and Saskatchewan the sludge accumulation was less than 0.34 litres *per caput* per day. As a rule, accumulation in summer is about one-third of the winter value.

The thawing of the ice cover in the spring is often accompanied by objectionable odours. These odours are most prominent around facultative ponds. Anaerobic pretreatment ponds tend to exhibit less intensive spring odours because of their thinner ice cover and shorter periods of coverage. However, these pretreatment ponds produce low-intensity odours throughout the year, often accompanied by floating sludge, scum, and grease.

Dikes that enclose large ponds may be eroded by wave action and by storm water run-off. Studies involving dikes with inside slopes varying from 16% to 11 % showed that flattening the slopes does not necessarily prevent erosion.

Mosquitos have been controlled effectively through the use of insecticides and other measures. The severity of mosquito breeding should be determined by field surveys.

British Columbia

Most ponds in this province are of the facultative type. Recently, however, an installation consisting of four anaerobic pretreatment ponds and three facultative ponds has been constructed, designed to permit operation in series or in parallel. The anaerobic ponds are loaded at a rate of 220 kg BOD_5/ha per day, the facultative ponds at a rate of 6 kg BOD_5/ha per day. The BOD_5 removal rates vary from 75 % to 90 %, the higher levels being obtained with the system using anaerobic pretreatment. The only odours reported have come from the anaerobic ponds during the spring thaw.

Since the severity of the winters is moderated by the Pacific Ocean, cold weather storage is not necessary; consequently, there is little difference in this region between winter and summer pond operating practices.

North-west Territories

A pond located at latitude 68°12′ N is used to treat the wastes from 1 500 people. The average monthly temperatures range from 14.6°C to —28.6°C, while daily temperatures in summer range from about 21°C in the afternoon to 7°C at night. Ice may occur from September to the middle of June, and odours have been noticed during the spring thaw. The surface area of this pond is 17 ha, and its depth varies between 30 cm and 2 m; the volumetric loading rate is 820 m^3 per day and the organic loading rate about 10 kg BOD_5/ha per day. The BOD_5 level is about 210 mg/litre in the influent wastes and about 40 mg/litre in the effluent, giving a removal rate of about 80 %. The dissolved oxygen level has been reported to be high during the long-daylight summer months.

Cost

Waste stabilization ponds offer considerable economic advantages over other forms of municipal and industrial waste-water treatment. Although ponds naturally require more space than most other methods, the construction, operating and maintenance costs are lower and may even be less than half. Furthermore, the cost *per caput* of small installations does not increase so rapidly with diminishing size. When reduction of BOD and bacteria is the only criterion, therefore, the waste stabilization pond has little competition. Obviously, however, there are other criteria that must be considered in the appraisal of alternative treatment systems. For example, the effluent from a waste stabilization pond may not be of adequate chemical quality to meet the requirements, or there might be some aesthetic reason for not using ponds. Such decisions involving projected water uses and comparisons of alternatives must remain the responsibility of the engineer. He must design treatment facilities within the framework of local, regional, and national regulations and needs, taking both health considerations and water resources into account.

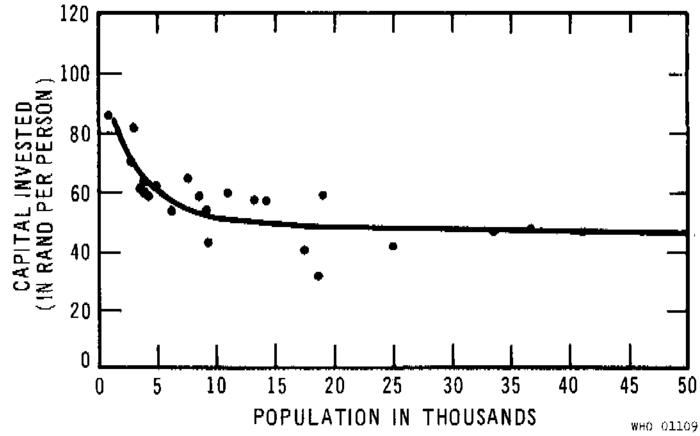
Waterborne sanitation

Unless the initial cost of a treatment facility and the costs of operation and maintenance are reasonable, waste treatment will be out of the financial reach of a large number of small communities. Usually, the cost *per caput* of waste-water purification increases sharply as the volume of water to be treated decreases (Stander & Meiring, 1965). A survey carried out in

South Africa indicates that the average annual cost per site of a sewerage system in a low-income area and the cost of a pail-latrine (night-soil) removal system are roughly the same. The adjusted total annual cost per site for waterborne sanitation, including the cost of water and one water closet per site, ranged from $11.53 to $21.22. The annual cost of sanitary pail removal and waste-water disposal into sewer inlets 5 km away ranged from $13.39 to $22.32 per site. The cost of treatment was not included. As a further yardstick for comparison, the total annual cost of a pit latrine system combined with septic tanks and a subsurface waste-water disposal system was $5.32 (Kieser, 1964).

Finally, the provision of a central collection system may involve costly outfall lines, pumps, etc. In a typical sewerage scheme, the sewer network accounts for about 75 % of the total cost, and the cost of the network *per caput* increases sharply as the size of the community decreases (Fig. 5). Through the use of waste stabilization ponds, it is often possible to provide a treatment facility in each drainage basin and thus avoid the costly transfer of waste-water from one basin to another.

FIG. 5. CAPITAL INVESTED IN SEWERS IN A NUMBER OF SOUTH AFRICAN TOWNS, IN RAND*



Data from Stander & Meiring (1965), p. 1026.
* 1 Rand = US $ 1.40.

General treatment costs

The cost of sewage treatment can be illustrated by comparing various treatment and construction costs.

Table 5 illustrates the earlier statement that there is a sharp increase in the cost of treatment per unit of volume as the size of the community decreases, except in the case of waste stabilization ponds where the estimated costs remain the same.

TABLE 5

ESTIMATED COST OF WASTE-WATER TREATMENT PER PERSON *

Population	Capital cost (US $)		Annual running costs (US $)	
	Conventional purification works [a]	Stabilization ponds without sealing of bottom	Conventional purification works	Stabilization ponds
240	86.80	5.60	—	0.84
900	44.80	5.60	—	0.56
1 000	50.40	5.60	—	0.42
3 000	36.40	5.60	4.20	0.42
5 000	33.60	5.60	—	0.42
10 000	21.00	5.60	—	0.42
30 000	22.40	5.60	2.80	0.42
50 000	16.80	5.60	—	0.42
200 000	22.40	5.60	—	0.42
1 000 000	9.80	5.60	0.84	0.42

* Data from Stander & Meiring (1965), p. 1026.

[a] *The values in this column are specimen costs taken from the literature. Consequently the decrease in cost for increasing population size is not regular.*

According to the US Public Health Service, construction costs represent about 80 % of the total estimated cost of sewage treatment facilities. Other cost items include: (*a*) facilities such as interceptors and outfall sewers; (*b*) pumping stations not contiguous to the treatment plant; (*c*) land; and (*d*) administrative, engineering, and legal services.

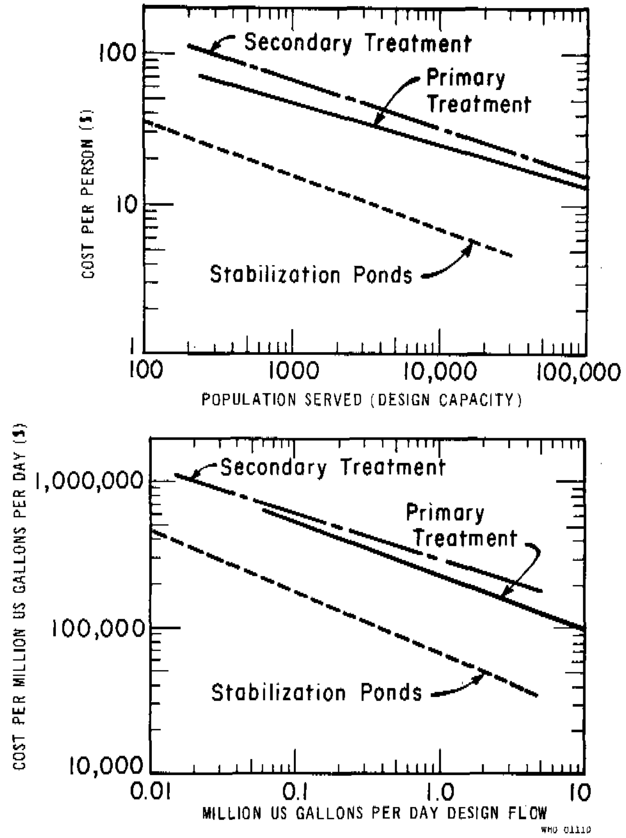
Comparative costs of secondary treatment plants in the USA are shown in Fig. 6. Here again, the cost *per caput* for treatment plants other than ponds is appreciably higher for the smaller plants.

Separate cost comparisons for five types of treatment in the USA are shown in Tables 6–10. These costs are based on the design capacity. Since these cost data are derived statistically, lower and upper confidence limits are given. The construction cost of a treatment system can be expected to be within these limits about two-thirds of the time.

Waste stabilization pond costs

A study of the economics of waste stabilization ponds in the midwestern states of the USA provides a basis for comparisons (Tables 11 and 12). The designs were based on loadings ranging from 150 to 48 000 population equivalents. In these calculations, one population equivalent (PE) is equal to 75 g BOD_5 per day. The data were derived from 262 treatment plants, comprising 13 primary sedimentation and digestion plants, 81 secondary biological plants, and 168 waste stabilization plants. Of the latter, 160 received raw waste-water and 8 received effluent from conventional treatment plants. No differentiation is made between waste stabilization ponds according to loading and size, layout (whether single-unit or multiple unit),

FIG. 6. COMPARISON OF CONSTRUCTION COSTS FOR DIFFERENT TYPES OF WASTE TREATMENT, IN US DOLLARS



Data from US Public Health Service (1965), pp. 18 and 33.
Note: 1 million US gallons = 3785 m^3.

TABLE 6

CONSTRUCTION COST *PER CAPUT*, IMHOFF TANK PLANTS *

Design population	Construction costs (US $)		
	Lower limit	Expected value	Upper limit
100	80.23	146.80	268.59
1 000	24.89	45.54	83.33
10 000	7.72	14.13	25.85
100 000	2.40	4.38	8.02

* Reproduced, in modified form, from Clare & Weiner (1965), p. 80, by permission of the publishers.

TABLE 7

CONSTRUCTION COST *PER CAPUT*, PRIMARY TREATMENT PLANTS WITH SEPARATE SLUDGE DIGESTION *

Design population	Construction costs (US $)		
	Lower limit	Expected value	Upper limit
100	77.12	114.96	171.36
1 000	37.52	55.93	83.37
10 000	18.25	27.21	40.56
100 000	8.88	13.24	19.73

* Reproduced, in modified form, from Clare & Weiner (1965), p. 80, by permission of the publishers.

TABLE 8

CONSTRUCTION COST *PER CAPUT*, WASTE STABILIZATION PONDS *

Design population	Construction costs (US $)		
	Lower limit	Expected value	Upper limit
100	19.96	36.31	66.10
1 000	8.85	16.10	29.31
10 000	3.92	7.14	13.00
100 000	1.74	3.17	5.77

* Reproduced, in modified form, from Clare & Weiner (1965), p. 81, by permission of the publishers.

TABLE 9

CONSTRUCTION COST *PER CAPUT*, TRICKLING FILTER PLANTS WITH SEPARATE SLUDGE DIGESTION *

Design population	Construction costs (US $)		
	Lower limit	Expected value	Upper limit
100	109.83	161.91	238.70
1 000	50.58	74.57	109.93
10 000	23.29	34.34	50.63
100 000	10.73	15.81	23.32

* Reproduced, in modified form, from Clare & Weiner (1965), p. 81, by permission of the publishers.

TABLE 10

CONSTRUCTION COST *PER CAPUT*, ACTIVATED SLUDGE TREATMENT *

Design population	Construction costs (US $)		
	Lower limit	Expected value	Upper limit
100	84.39	124.98	185.07
1 000	44.47	65.85	97.52
10 000	23.43	34.70	51.39
100 000	12.35	18.29	27.08

* Reproduced, in modified form, from Clare & Weiner (1965), p. 82, by permission of the publishers.

or earthwork and land costs. Table 11 presents a comparison of the actual contract costs, including construction, land and outfall sewer costs. Table 12 shows the land costs for treatment works.

It is estimated that the operation and maintenance costs for waste stabilization ponds range from $0.20 to $1.00 per population equivalent per year. These costs have been estimated for ponds with different design loadings, and very little difference has been found (Frankel et al., 1965).

TABLE 11

COMPARISON OF CONSTRUCTION COSTS OF 262 WASTE TREATMENT PLANTS IN THE MIDWESTERN STATES OF THE USA, 1957-1959 *

Population equivalents [a]	Cost per design population equivalent (US $)			
	Primary settling and digestion plants	Secondary biological treatment plants	Secondary waste stabilization ponds	Raw sewage ponds
100	77.91	127.30	105.20	29.36
1 000	40.05	64.83	53.34	18.52
10 000	20.58	33.02	27.04	11.69
100 000	10.58	16.82	13.71	7.37

* Data from Clare & Weiner (1961), p. 164.

[a] One population equivalent = 75 g BOD_5 per day.

TABLE 12

LAND COSTS FOR WASTE TREATMENT PLANTS IN THE MIDWESTERN STATES OF THE USA, 1957-1959 *

Type of plant	Cost in US $ per population equivalent [a]
Primary treatment	1.25
Secondary treatment	1.20
Raw sewage waste stabilization ponds	3.10
Secondary waste stabilization ponds	2.30

* Data from Clare & Weiner (1961), p. 64.

[a] The figures are based on loadings of approximately 22 kg BOD_5/ha per day.

Summary of Practices

The benefits that can be derived from the use of waste stabilization ponds in rural areas, small communities, and even in large cities are measurable and significant. The effective use of low-cost facilities in waste-water treatment is an investment in the improvement of environmental health.

While stabilization ponds cannot rival an efficiently operated activated sludge treatment plant in producing a turbidity-free effluent, the thousands of ponds in operation attest to their usefulness. Only when a cleaner effluent is required does it become necessary to adopt a more expensive method of treatment.

CHAPTER 3

Waste-water Characteristics and Biological Stabilization

Waste stabilization ponds can be made to function most effectively if the designer and operator have a basic understanding of waste-water characteristics and of the principles of biological stabilization.

The nature of pollution, the characteristics of wastes, biological reactions, oxygen demand, and the stoichiometry of algae and bacterial systems are briefly discussed in this chapter. A more detailed description of pond organisms and other factors affecting waste stabilization ponds is provided in Annex 2. For further details the reader is referred to the technical literature on biological waste-water treatment.

Nature of Pollution

Pollutants may be classified as biodegradable or non-biodegradable. Pollutants such as inorganics are not biodegraded, and once they enter the receiving water they may be diluted but are not necessarily reduced in quantity.

Other pollutants are changed in character by biological, chemical, and physical forces, and because of these changes the unstable organic waste in domestic waste-water can be converted to inoffensive substances. Likewise, some industrial wastes can be converted into stable effluents.

The substances and organisms contained in wastes may include: (*a*) infectious agents, (*b*) oxygen-demanding wastes, (*c*) plant nutrients, (*d*) organic chemicals (including many pigments), (*e*) inorganic chemical and mineral substances, (*f*) sediments, and (*g*) radioactive materials.

Waste Characteristics

The amount and composition of the wastes, whether waterborne or not, are of prime importance to the designer and operator of a treatment facility. Although there is considerable similarity in basic content, the volume and character of wastes obviously vary from one country to another. Climatic conditions and social customs are but two of the many factors that cause the type and volume of wastes to differ.

Table 13 shows the quantities of domestic sewage normally produced in different areas of the world. However, caution should be exercised in using these figures as a basis for any specific design.

TABLE 13

QUANTITY OF DOMESTIC SEWAGE PRODUCED PER HOUSEHOLD, IN SELECTED COUNTRIES *

Country	Water carriage in litres *per caput* per day
Brazil	150-300
Costa Rica	262-379
Cuba	190-225
Ecuador	100
Ghana	90-145
India	100-250
Israel	80-415
Japan	250
Mauritius	63-144
New Zealand	226-230
Nigeria	80
Peru	140
Saudi Arabia	158
Southern Rhodesia	7-77

* Author's survey.

In general, the amount of water accompanying the organic material increases as the community water supply and industrial complex become more highly developed. The solids content of waste-water will be greater in villages having limited water supplies than in cases where a highly developed water carriage system is used to carry away the sewage.

The organic matter in domestic waste-water can be divided into three main classes: proteins, carbohydrates, and fats. The proteins, which comprise 40–50 % of the organic matter, are complexes of amino acids and constitute the major source of bacterial nutrients. Roughly 50–60 % of the protein may be found in the dissolved fraction of domestic waste-water and 20–30 % in the settleable fraction. The carbohydrates consist of readily degraded starches and sugars and of cellulose, which is less readily degraded. The percentages of carbohydrates found in the dissolved and settleable fractions are similar to those for protein. The fats, including fatty acids, are usually not very soluble and are degraded more slowly.

Gotaas (1956, p. 35) gave the following figures for the quantity and composition of human body wastes:

Human faeces without urine

Approximate quantity

135–270 g *per caput* per day moist weight
35–70 g *per caput* per day dry weight

Approximate composition

Moisture content	66–80 %
Organic matter content (dry basis)	88–97 %
Nitrogen " "	5.0–7.0 %
Phosphorus (as P_2O_5) " "	3.0–5.4 %
Potassium (as K_2O) " "	1.0–2.5 %

Carbon	(dry basis)	40–55 %
Calcium (as CaO)	” ”	4–5 %
C/N ratio	” ”	5–10

Human urine

Approximate quantity

Volume: 1.0–1.3 litres *per caput* per day
Dry solids: 50–70 g *per caput* per day

Approximate composition

Moisture content		93–96 %
Organic matter content	(dry basis)	65–85 %
Nitrogen	” ”	15–19 %
Phosphorus (as P_2O_5)	” ”	2.5–5 %
Potassium (as K_2O)	” ”	3.0–4.5 %
Carbon	” ”	11–17 %
Calcium (as CaO)	” ”	4.5–6 %

The composition of wastes from animals is shown in Table 14. The values are based on a 2.25-kg chicken (usual laying hen), a 454-kg bovine (small dairy cow or marketable beef animal), and a 45-kg pig (half-grown to market size) (Hart & Turner, 1965). In terms of population equivalents, the BOD_5 of animal wastes is as follows (Butler et al., 1964, p. 12):

Source	*Population equivalents*
Man	1.0
Cow	16.4
Horse	11.3
Chicken	0.014
Sheep	2.45
Pig	3.0

TABLE 14

CHARACTERISTICS OF LIVESTOCK MANURE *

Factor	Poultry	Cattle	Swine
Total solids in faeces and urine (g per day)	30	4730	360
Solids content of freshly collected total sample (%)	23	18	19
Volatile solids, as percentage of total solids	77.5	80.3	78.5
Volatile solids (g per day)	23	3800	282
BOD_5 (mg BOD_5/mg volatile solids)	0.288	0.183	0.320
BOD_5 (g O_2 per day)	6.8	700	91
COD[a] (mg COD/mg volatile solids)	1.11	1.0	1.2
COD[a] (g O_2 per day)	25.8	3800	340
Nitrogen, as N, in fresh manure (percentage of total solids)	5.4	3.7	4.0
NH_4 nitrogen (percentage of total N)	74	—	75
Phosphorus, as P_2O_5 (percentage of total solids)	4.6	1.1	3.1
Potassium, as K (percentage of total solids)	2.1	3.0	1.4
Volatile acids, as acetic acid (percentage of total solids)	5.8	3.2	4.8

* Data from Hart & Turner (1965), p. 1580.

[a] Chemical oxygen demand.

Table 15 shows the approximate quantity and composition of sewage and sewage sludge from various sources. The wastes from each community, particularly where industries are present, require careful analysis, for flow patterns and waste characteristics are likely to vary considerably. For example, toilet paper usage may vary from nil to 20 g per person per day, and the concentration of such paper in domestic waste-waters may be as much as 20–80 mg/litre. Home clothes-washing devices may contribute 55–110 mg/litre solids and 15–30 mg/litre BOD_5 (Durfur & Becker, 1962). In dishwater the solids content and BOD both vary considerably.

Mode of decomposition

The decomposition of organic material may take place under aerobic or anaerobic conditions (Gloyna & Eckenfelder, 1968). The aerobic process requires a continuous supply of free dissolved oxygen and is the most efficient method for reducing the organic content of dilute liquid wastes. However, where solids must be liquefied and where the wastes are highly concentrated, as in the case of settled organic solids from domestic waste-waters, night-soil, and wastes from abattoirs, the anaerobic process is extremely effective.

Aerobic process

In the aerobic metabolism of organic matter, much of the carbon serves as a source of energy for the organisms and is respired as carbon dioxide (CO_2). The organisms involved are mostly bacteria, but also include fungi and protozoa. They use the remainder of the carbon, together with phosphorus and nitrogen, to form new cells. In typical domestic waste-waters, the weight of cells produced is roughly equal to 40 % and 60 %, respectively, of the weight of chemical oxygen demand (COD) and BOD_5 removed. The major reactions likely to occur in an aerobic waste stabilization pond system are as follows:

$$(CH_2O)_x + xO_2 \rightarrow xCO_2 + xH_2O$$

Protein (organic N) → ammonia → nitrite → nitrate

Organic sulfur → sulfate

Organic phosphate → H_3PO_4 → $CaPO_4$

The quantity of oxygen required to stabilize the organic material in the waste depends on the BOD satisfied during treatment. This BOD is the oxygen that must be supplied to waste stabilization ponds by photosynthetic means, transferred across the air-water interface, or obtained from oxygen-containing compounds such as nitrates, phosphates, and sulfates. Similarly, the rate at which oxygen is used is an important factor in the design of a treatment plant.

TABLE 15

APPROXIMATE QUANTITY AND COMPOSITION OF VARIOUS TYPES OF SEWAGE AND SEWAGE SLUDGE *

Item	Quantity of solids, dry basis (g per head per day)	Composition on dry basis (%)				
		Organic matter	Minerals	Nitrogen (N)	Phosphate (P_2O_5)	Potash (K_2O)
Fresh domestic sewage	81.5-100	60-85	15-40	5.0-10.0	2.5-4.5	3.0-4.5
Imhoff tank, well digested	22.6-36.2	30-45	55-70	2.0-3.0	1.2-3.5	0.1-0.5
Primary, fresh	45.4-63.4	60-80	20-35	1.5-4.0	0.8-4.0	0.1-0.5
Primary, digested	27.0-40.5	35-60	40-65	1.0-3.5	1.2-4.0	0.1-0.5
Primary and trickling-filter humus, fresh	58.4-76.3	50-75	25-50	2.0-4.5	0.8-3.6	0.1-0.5
Primary and trickling-filter humus, digested	36.0-49.5	35-60	40-65	1.0-3.5	1.0-3.8	0.1-0.5
Primary and activated sludge, fresh	72.0-90.0	50-80	20-50	2.3-5.2	1.2-4.0	0.2-0.6
Primary and activated sludge, digested	45.4-46.3	35-55	45-65	2.0-4.8	1.3-4.0	0.2-0.6
Primary sludge, digested, and fresh activated sludge	54.0-72.0	40-60	40-60	2.2-5.0	1.3-4.0	0.3-0.8

* After Gotaas (1956), p. 40.

Aerobic micro-organisms have the ability to synthesize new cell material from wastes containing complex organic compounds. Thus some of the organic material is used to make protoplasm and some of the waste is degraded into low-energy compounds. Oxygen must be supplied continuously during the aerobic process, for it acts as the final hydrogen acceptor during the oxidation of organic matter and the reaction ceases if it is not available. Liberation of energy occurs during this hydrogen transfer.

Anaerobic process

Putrefactive breakdown of organic material takes place during anaerobic fermentation. This is a two-step process. First, a special group of acid-producing bacteria known as facultative heterotrophs degrade organic matter into fatty acids, aldehydes, alcohols, etc. Then a group of methane bacteria convert the intermediate products to methane (CH_4), ammonia (NH_3), carbon dioxide (CO_2), and hydrogen (H_2). Like the aerobic process, the anaerobic process converts carbon, nitrogen, phosphorus, and other nutrients to cell protoplasm. However, the mechanisms of anaerobic decomposition are exceedingly complex and are not thoroughly understood.

Oxygen is also required for the anaerobic process, but its source is chemical compounds, not free dissolved oxygen. In anaerobic decomposition the end products are quite complicated; the reactions are slower and the products may be odoriferous.

There is always some anaerobic activity in the bottom muds and sediments, even in ponds designed to operate aerobically. In deep ponds, too, there is likely to be a layer of liquid near the bottom that will support anaerobic organisms.

The biochemical reactions that occur in the anaerobic decomposition of wastes might be expressed as follows:

$$(CH_2O)_x \rightarrow xCH_3COOH$$

$$CH_3COOH \rightarrow CH_4 + CO_2$$

Organic nitrogen → ammonia

$$2\ H_2S + CO_2 + \text{light} \rightarrow (CH_2O)_x + S_2 + H_2O$$

Organic phosphate undergoes reduction

Biological Reactions

As has been made clear in the previous section, the biological organisms in a waste-water treatment plant may produce many biochemical changes. The major biological reactions occurring in waste stabilization ponds include: (*a*) oxidation of organic material by aerobic bacteria, (*b*) nitrification of protein and other nitrogenous material by aerobic bacteria, (*c*) reduction of organic material by anaerobic bacteria living in bottom deposits and liquids, and (*d*) oxygenation of surface liquids by algae.

Certain basic facts regarding the oxidation of organic wastes are now accepted: (*a*) the dissolved oxygen is reduced during stabilization of organic material; (*b*) the rate of oxidation is independent of the amount of dissolved oxygen available; (*c*) the type and number of organisms present is important; and (*d*) changes in oxygen content can serve as a measure of the quantity and character of oxidizable organic matter.

Oxygen Demand

The oxygen demand of waste-water may be expressed in terms of the biochemical oxygen demand (BOD) and the chemical oxygen demand (COD). These values do not necessarily measure the same things, but both are useful indicators of the total oxygen demand that may be imposed on a treatment plant or waterway by a waste. The total oxygen demand is exerted by three groups of matter: carbonaceous material, oxidizable nitrogen, and certain reducing compounds. The oxygen demand of the carbonaceous and nitrogenous material is usually measured in terms of a BOD test, while the total demand is generally expressed by the COD.

BOD test

The BOD test measures the amount of dissolved oxygen required by living organisms for the aerobic destruction or use of organic matter. Thus the BOD represents the amount of dissolved oxygen that will be required to develop and sustain the biological activity needed to degrade a given quantity of waste; if the aerobic treatment is prolonged, the BOD may also include the amount of oxygen required to degrade part of the biological cell material that was produced from the initial waste.

As shown in Fig. 7, the BOD of freshly polluted waste-water develops in two stages. During the first stage, the carbonaceous material is largely oxidized, while in the second stage a significant amount of nitrification takes place. During the first stage, the rate of organic breakdown at any time is assumed to be directly proportional to the amount of biologically degradable material present. The biological stabilization of a waste may take a long time, and for practical purposes a 5-day incubation period, BOD_5, has been accepted as a standard reference.

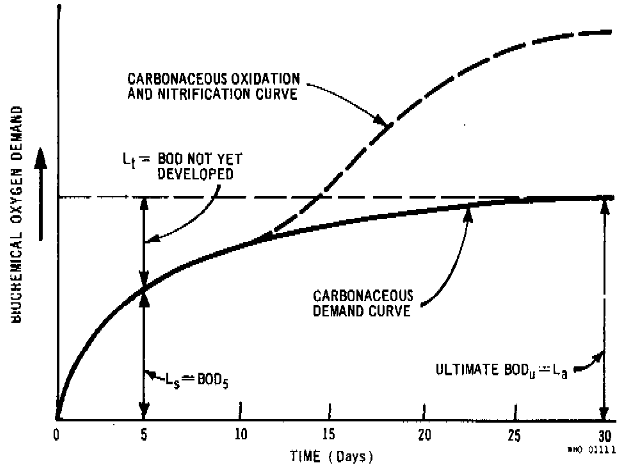
The tests involved in the evaluation of BOD have been thoroughly discussed by the American Public Health Association et al. (1965) and Åkerlindh (1953).

Particular care must be taken when neutralizing the wastes and when seeding the dilution water with sewage, polluted stream water, or special biological cultures that have become adapted to specific industrial wastes. In order to obtain the maximum BOD value, it is also important to use the proper dilution to ensure that any toxic effect is removed and that not all the dissolved oxygen is used. When increasing dilutions show increasing

BOD values, the dilutions should be increased until the BOD levels off at its maximum.

The ultimate BOD, BOD_u, is the sum of the carbonaceous BOD already satisfied and the carbonaceous BOD remaining (Equation 1). In the case of domestic waste-waters, the ultimate demand may be attained after about 20 days of incubation at 20°C. After 5 days of incubation at this temperature, about 60–90 % of the ultimate (first-stage) demand of typical domestic waste-waters is satisfied.

FIG. 7. DEVELOPMENT OF BIOCHEMICAL OXYGEN DEMAND



$$L_a = L_s + L_t \tag{1}$$

where

L_a = ultimate BOD, BOD_u (mg/litre)[1]
L_s = BOD satisfied after t days (mg/litre)
L_t = BOD not yet developed after t days (mg/litre)

Equations 2–4 describe the rate at which the carbonaceous BOD will be satisfied. The rate constant, k, is dependent upon the waste characteristics and temperature. In Equations 3 and 4, typical values of k may vary from 0.10 to 0.30 for domestic waste-waters, from 0.15 to 0.25 for untreated domestic waste-waters and high-rate treatment effluent, from 0.04 to 0.07 for high-level biological treatment plant effluent, and from 0.04 to 0.06 for streams with minor pollution.

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[1] In most cases, the symbols used in this monograph are explained only where they first appear. A full list of the symbols and their meanings is provided in Annex 2 (p. 160).

$$-\frac{\mathrm{d}L_t}{\mathrm{d}t} = k_1 L_t \tag{2}$$

$$\frac{L_t}{L_a} = \mathrm{e}^{-k_1 t} = 10^{-kt} \tag{3}$$

$$L_s = L_a - L_t = L_a(1 - 10^{-kt}) \tag{4}$$

where

t = time (days)
L_a = BOD_u or initial BOD (mg/litre)
k_1 = reaction constant, $\log_e$ (breakdown/day)
k = reaction constant, $\log_{10}$ (breakdown/day)

The use of Equation 4 is complicated by the fact that L_a and k are generally not known. The values must be estimated from laboratory data by one of several methods that are commonly used for determining L_a and k from a series of observations of L_s and t.

COD test

As at present carried out, the chemical oxygen demand determination provides a measure of the oxygen equivalence of that portion of the organic matter in a sample which is susceptible to oxidation by a strong chemical oxidizer. If wastes contain toxic substances, this test may be the only practical method for determining the organic load. Because of interpretation problems, the COD test becomes really meaningful only after many values of both BOD and COD have been collected and correlated with each other.

Nitrification

In some waste-water treatment requirements, emphasis is placed on the removal of nitrogen from the treated effluents in order to minimize the release of plant nutrients to the receiving water, for a build-up of such nutrients in the waterway or impoundment accelerates the aging process known as eutrophication. As yet, waste stabilization ponds have not been used successfully for denitrification, but this might be possible. Denitrification is carried out by facultative, heterotrophic, anaerobic bacteria that reduce nitrates to N_2, N_2O, and NO. Before this can be done the nitrogenous material must first be converted aerobically to nitrates. The level of dissolved oxygen should be near zero when reduction takes place. As the pH value of a pond rises during high light intensity, significant quantities of ammonia may be liberated from the pond surface; where significant anaerobic lysis is operating, breakdown protein residues are directly converted to ammonia, nitrogen gas may be released, and the conversion of ammonia into protein in newly synthesized algal cells may provide a useful path for further denitrification.

Anaerobic reduction

The residual products remaining after anaerobic decomposition will diffuse into the upper liquid of a waste stabilization pond, so increasing the oxygen demand. This factor is sometimes overlooked in the design and operation of waste stabilization ponds. The settleable solids will eventually undergo anaerobic decomposition and thus create a BOD in the upper, aerobic sections of the pond. The total oxygen requirement for the aerobic zone will be the amount needed to stabilize the dissolved and suspended organic material together with that fraction of the settleable organic material that is resuspended.

Stoichiometry of Algal and Bacterial Systems

A quantitative chemical relationship exists between the organic wastes treated and the micro-organisms produced. The decomposition of organic material by heterotrophic bacteria (decomposers) and the ingestion of organisms by animals (consumers) result in the return of soluble inorganic components to the water. This decomposition may be expressed by Equation 5:

$$\begin{aligned} & C_aH_bN_cO_dP_e + (a + \tfrac{1}{4}b - \tfrac{1}{2}d + \tfrac{3}{2}c + 2e)\,O_2 \\ & \quad = aCO_2 + \tfrac{b}{2}H_2O + cNO_3^- + ePO_4{}^{3-} \end{aligned} \tag{5}$$

Conversely, the algae, acting as autotrophic organisms, produce organic material from inorganic substances. The reconstitution of simple materials to cellular substance might be represented by Equation 6:

$$\begin{aligned} & 106CO_2 + 90H_2O + 16NO_3^- + 1PO_4{}^{3-} + \text{light} \\ & \quad = C_{106}H_{180}O_{45}N_{16}P_1 + 154\tfrac{1}{2}O_2 \end{aligned} \tag{6}$$

As shown in Equations 7 and 8, the effluent from a conventional waste-water treatment plant contains about 50 % of the phosphorus and nitrogen initially present in the raw waste-water, as well as significant amounts of carbon dioxide. The exact amount will depend on the design and operation of the biological treatment plant. These ingredients, together with the trace elements normally present in domestic effluents, constitute a suitable nutrient for the development of algal growths. Equation 7 depicts the available nutrients in a maturation pond following a conventional biological treatment plant. The data in Equations 7 and 8 are expressed in mmol per litre and are fairly typical of domestic waste-waters in the USA (Stumm & Morgan, 1963).

Untreated waste-water + oxygen → bacterial sludge + treated effluent (7)

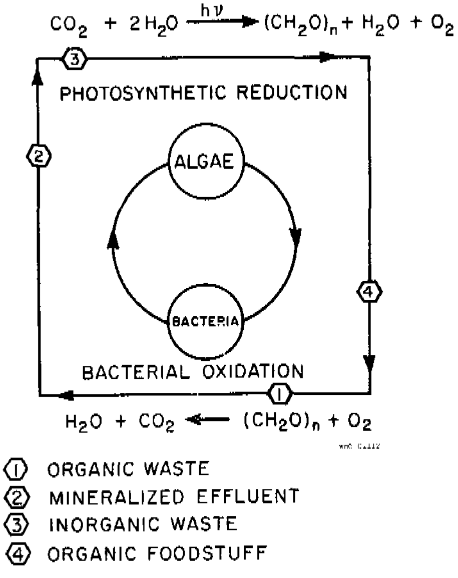
$$\begin{array}{c}\boxed{\begin{array}{l} P = 0.14 \\ N = 2.50 \\ C = 9^a \\ C = 1^b \end{array}} \\ C : N : P \text{ ratio} \\ = 70 : 17 : 1 \end{array} + 11O_2 \rightarrow \begin{array}{c}\boxed{\begin{array}{l} P = 0.075 \\ N = 1.2 \\ C = 8^a \end{array}} \\ C : N : P \text{ ratio} \\ = 106 : 16 : 1 \end{array} + \begin{array}{c}\boxed{\begin{array}{l} P = 0.065 \\ N = 1.3 \\ C = 1^a \\ C = 1^b \end{array}} \\ C : N : P \text{ ratio} \\ = 30 : 20 : 1 \end{array} + \text{energy}$$

Treated effluent + carbon dioxide + light → algal mass + oxygen (8)

$$\boxed{\begin{array}{l} P = 0.065 \\ N = 1.3 \\ C = 1^a \\ C = 1^b \end{array}} + [C = 6] + \text{light} \rightarrow \begin{array}{c}\boxed{\begin{array}{l} P = 0.065 \\ N = 1.05 \\ C = 7^a \end{array}} \\ C : N : P \text{ ratio} \\ = 106 : 16 : 1 \end{array} + \boxed{\begin{array}{l} O_2 = 10 \\ C = 1^b \end{array}}$$

The algae-bacteria relationship found in a pond is shown in simplified form in Fig. 8. These two groups of micro-organisms may live with one another effectively; they are not parasitic to each other.

FIG. 8. SIMPLIFIED DIAGRAM OF ALGAE-BACTERIA COMMENSALISM-CARBON CYCLE



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[a] Assimilable carbon.

[b] Non-assimilable carbon.

CHAPTER 4

Process Design Procedures

Previous chapters of this monograph have provided essential background information on the needs of waste-water management, the history of waste stabilization pond development, and biological treatment. This and the following chapters are concerned with the design and operation of ponds in relation to BOD removal, disease transmission control, physical facilities, and operation and maintenance needs. The examples provided in these chapters are intended as guides: as in the design of any waste-water treatment plant, the engineer's judgement must play a leading part in the overall planning process.

This chapter deals with the establishment of the physical size of various types of individual waste stabilization ponds on the basis of recognized concepts of biological degradation. Although the main emphasis in this monograph is on facultative ponds, attention is also directed to the design of anaerobic and maturation ponds. Some mention is made of the pretreatment of wastes by aqua privies because of their possible use in low-income housing developments. Finally, a brief account is given of the use of mechanically aerated waste stabilization ponds as a pretreatment measure, followed by facultative ponds; where waste discharges are large, the combination of these two types of pond may be a realistic method of waste-water treatment.

The design engineer may wish to use the examples presented in this chapter for establishing base-line comparisons or to use a particular example to develop a complete design. Where climatic conditions and local customs are known to be similar to those applying to a particular example given below, that example should be used. Where basic data are not available, however, the designer should compare the various alternatives and select the design with the largest pond volume or the longest detention time. Equations, graphs and specimen calculations are provided for the engineer's use.

Predesign Considerations

Before a design for a specific installation is attempted, it is necessary to obtain as much background information concerning the problem as possible. Where low-cost waste treatment plants are contemplated, most of the basic data will not normally be available and sound engineering judgement will be needed. Where there are no records, for example, the engineer may have to estimate the anticipated BOD load, the solids content, and the amount of waste-water, and even seasonal temperatures and solar radiation. The following list of data requirements may be useful; the items in parentheses are of particular significance.

Waste classification

Domestic (type, age, mode of delivery)
Industrial (type, age, mode of delivery)
Agricultural (type, age, mode of delivery)
Combination (type, age, mode of delivery, relative percentages)

Waste characteristics

Volume (minimum, average, maximum daily)
Concentration of organic materials (BOD_5, BOD_u, COD)
Solids (special industrial wastes; total, settleable, suspended, dissolved, total volatile, and fixed solids)
Nutrient concentrations (nitrogen, phosphorus)
Colour materials (suspended, dissolved)
Toxicity (BOD reduction rates, bioassays, and specific aquatic micro-organisms)
pH (minimum, average, maximum)
Quality of tap water (mineral analyses)

Hydrology and meteorology

Evaporation (average, seasonal variations)
Rainfall (average, seasonal variations)
Air and water temperatures (average, seasonal average, average hottest month, average coldest month)
Groundwater (average depth, permeability of formation)
Percolation rates (preferably measured with waste to be treated)
Wind (strength and direction for time of day, seasonal average)
Cloud cover (percentage of daytime, season, and year)

Light

Solar radiation (minimum monthly, average annual, seasonal variations)

Topography

Soil characteristics (ease of excavation, embankment use, percolation, compaction)
Flood stages (high-water marks)
Availability of contour map
Location of houses, industries, agriculture
Streams (drainage of all types)

Regulations, public health data

Potential odour problems
Potential insect problems
Community data (local zoning restrictions, effluent and stream standards, ownership of lands, location of waterworks)
Health regulations (effluent and stream standards, coliform bacteria standards, special quality regulations for drinking-water and food processing, bathing water criteria, regulations on sea-food, freshwater fish)

Regulations of water resource authorities (stream and estuarine standards; effluent criteria; wildlife, recreation, fisheries regulations; and prior appropriated rights to wastewaters)

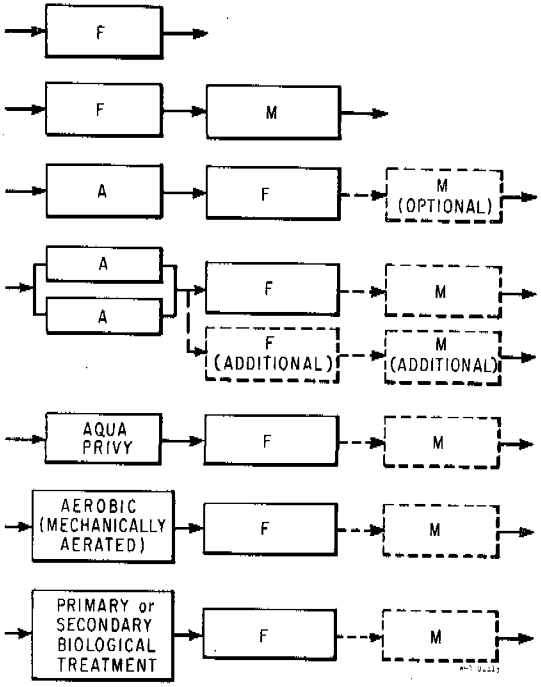
Use of effluent

Groundwater supplementation
Surface water supplementation
Immediate irrigation
Industrial re-use
Development of wildlife
Recreational use

Typical Pond Layouts

The waste stabilization pond system may comprise one pond only (facultative) or several types of pond in series (anaerobic, facultative, and maturation) (Fig. 9). In addition, it may be desirable to construct a number of series of the same type so as to permit parallel operation. For example, facultative or anaerobic ponds may be designed to operate

FIG. 9. A NUMBER OF TYPICAL WASTE STABILIZATION POND SYSTEMS



A = Anaerobic pond.
F = Facultative pond.
M = Maturation pond.

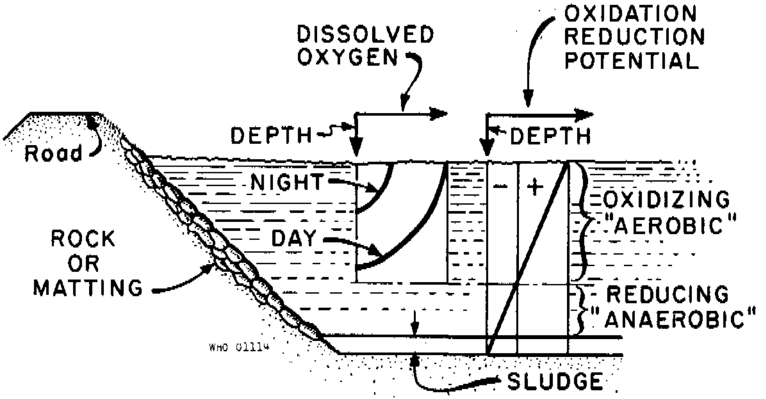
singly or in parallel. If a higher degree of treatment is desired, a maturation pond might be added beyond the facultative pond. Similarly, wastes from aqua privies may be further treated in facultative ponds. In some cases, mechanically aerated waste stabilization ponds may be used for preliminary aerobic treatment of biological wastes.

Facultative Ponds

Facultative waste stabilization ponds are those in which the upper layer is aerobic, the central area supports facultative bacteria, and the bottom sludge zone is truly anaerobic. At present most primary waste stabilization ponds treating raw waste-waters are of the facultative type. The settleable solids cause a layer of sludge or benthos to form on the bottom of the pond. In a facultative pond receiving raw waste-waters, the build-up of settleable solids is a function of temperature, but in all cases the degradable solids exert a major influence on the behaviour of the pond. Where a facultative pond follows an anaerobic pond the sludge layer is negligible. Similarly, the detritus in a maturation pond following a facultative pond consists principally of cell debris.

The facultative pond is oxygenated principally by the photosynthetic activity of algae under the influence of solar radiation, although in the larger ponds surface aeration by wind action contributes significantly to the total oxygen budget. The dissolved oxygen and the oxidation-reduction potential patterns of a typical facultative waste stabilization pond are indicated in Fig. 10. The dissolved oxygen concentration is greater during daylight periods than at night. The measurement of oxidation-reduction potential shows the tendency towards true aerobic or anaerobic conditions. The reducing environment will be found near the bottom, indicating true anaerobic conditions.

FIG. 10. A FACULTATIVE WASTE STABILIZATION POND



Because of sunlight absorption by the algal cells, effective light penetration may be less than 1 m; consequently, oxygen generation is frequently confined to the upper layer. If there is no mixing, gradients of dissolved oxygen develop, descending from a maximum at the surface of the pond to zero or near zero in the deeper sections. There is concomitant development of temperature gradients, which may effectively inhibit mixing of the pond contents by wind action.

Wind is usually the principal source of energy for mixing the water in facultative ponds, but a secondary cause of mixing that may be of significance in tropical areas when the wind velocity is low is differential heating. Mixing is an important physical parameter affecting the growth of algae, for many algae are non-motile and mixing is necessary to bring them into the zone of effective light penetration. Reduction of the duration of mixing during the usual diurnal cycle may lead to a reduction in the quantity of algae and a shift from one dominant species of algae to another. Moreover, mixing during daylight contributes to the distribution of dissolved oxygen.

Temperature is of great importance because it affects the rate of biochemical degradation. The average temperature, daily fluctuations, and yearly variations all influence the biological, physical, and chemical processes in the pond.

An empirical equation based on the kinetics of degradation of waste within a facultative pond was first presented by Hermann & Gloyna (1958); the idea was further developed by Marais & Shaw (1961), Marais (1966), Huang & Gloyna (1968), and Gloyna & Espino (1969).

Kinetic model of pond breakdown action

If it is assumed that all the influent BOD is stabilized by facultative organisms, that complete mixing occurs, and that breakdown takes place according to a first-order reaction, the effluent and influent BOD concentrations may be described by Equation 9. Although it provides a useful basis, this is an idealized equation that does not take into account the difference between the biological breakdown rates of the soluble material and those of the settleable solids.

$$L_p = \frac{L_o}{K_T R_T + 1} \tag{9}$$

where

L_p = pond and effluent BOD_5 (mg/litre)
L_o = influent BOD_5 (mg/litre)
K_T = breakdown rate at temperature T
R_T = detention time at temperature T

The breakdown rate, K_T, depends on the temperature as follows:

$$\frac{K_{35}}{K_T} = \theta^{(35-T)} \tag{10}$$

where

T = pond operating temperature (°C)
θ = temperature reaction coefficient = 1.085
and K_{35} = breakdown rate at 35°C

For a fixed percentage reduction of BOD, the symmetry of K_T and R_T in Equation 9 permits Equation 10 to be expanded as follows:

$$\frac{K_{35}}{K_T} = \theta^{(35-T)} = \frac{R_T}{R_{35}} \tag{11}$$

where

R_{35} = detention time at 35°C

Data obtained by Suwannakarn & Gloyna (1964) from a series of laboratory-scale ponds treating a synthetic non-settling waste at a number of different temperatures were analysed by Marais (1966), who obtained values for K_{35} and θ of 1.2 and 1.085 respectively. Extensive laboratory and field studies (Hermann & Gloyna, 1958) have shown that certain beneficial green algae cease to function effectively at water temperatures in excess of 35°C. Using Equation 9 and a measured value for K_{35}, therefore, the detention time for any percentage reduction at 35°C can be determined. At any other temperature (T), the detention time (R_T) for the same percentage reduction can be determined by using Equation 11.

The correlation between Suwannakarn & Gloyna's experimental BOD_5 data and theoretical values is shown in Fig. 11. These data verify the mathematical model under idealized conditions of waste composition, temperature, and mixing.

The projected efficiency of a facultative pond might be established by tabulating various values of K_T, R_T and T. Table 16 provides values of K_T for different temperatures, assuming the reaction rates remain constant.

TABLE 16

VALUES OF K_T*

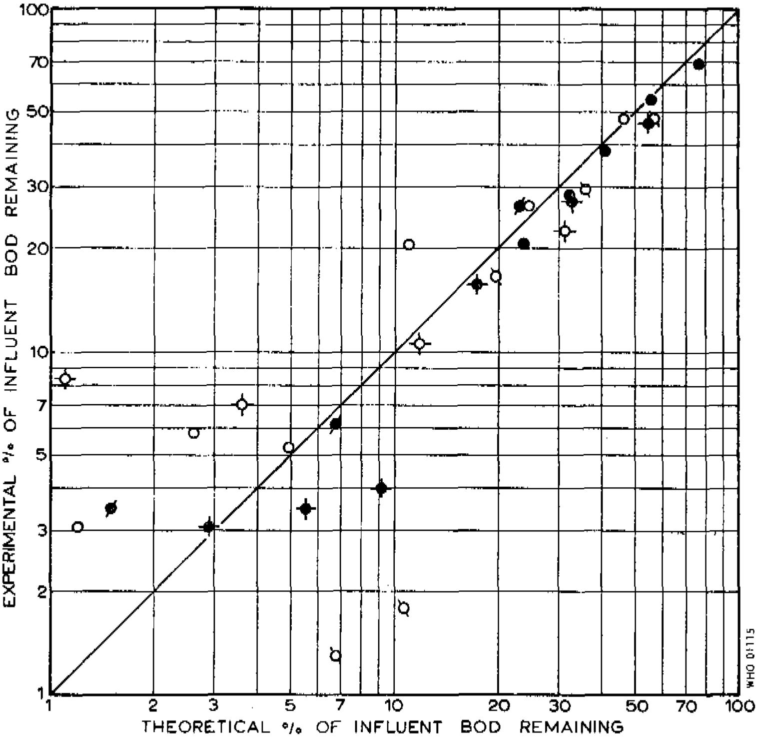
Temperature (°C)	5	10	15	20	25	30	35
K_T per day	0.103	0.12	0.24	0.35	0.53	0.80	1.2

* After I. Duarte (personal communication, 1968).

On the basis of Equation 9, the BOD removal efficiencies can be calculated for various values of R_T and K_T:

$$\text{Removal } (\%) = \left(\frac{L_o - L_p}{L_o}\right) 100 = \left(1 - \frac{1}{K_T R_T + 1}\right) 100 = \left(\frac{R_T}{\frac{1}{K_T} + R_T}\right) 100 \qquad (12)$$

FIG. 11. BOD REMOVAL IN LABORATORY-SCALE SERIES OF OXIDATION PONDS: CORRELATION OF EXPERIMENTAL AND THEORETICAL RESULTS



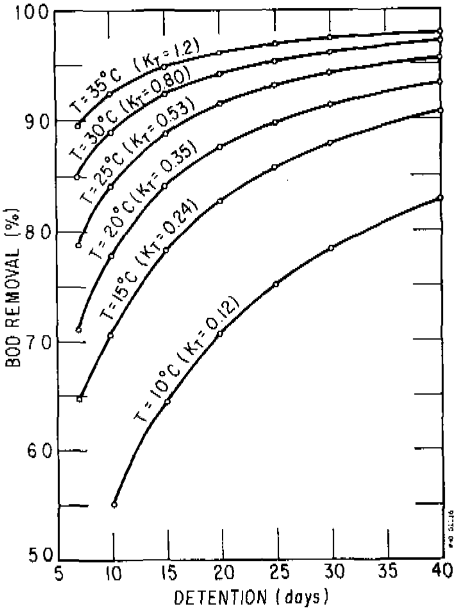
	Temperature (°C)	Flow (litres/day)
◐	35	6
-○-	35	9
○	24	6
ϙ	24	9
-●-	20	6
●	9	6

From Marais (1966), based on experimental data obtained by Suwannakarn & Gloyna (1964).

In Fig. 12, idealized removal efficiencies are plotted against various detention periods. While this graph does not provide all the information required to design a pond, it does demonstrate the important relationship between temperature and detention time.

It is evident, however, that the equations suggested above may fail to describe the effects produced by settleable solids. For example, they do not take into account the influence of soluble BOD released from the sludge layers and the toxicity effects exhibited by certain industrial wastes (Gloyna & Espino, 1969; Huang & Gloyna, 1968).

FIG. 12. IDEALIZED BOD_5 REMOVAL RATES IN FACULTATIVE PONDS



Data from I. Duarte (personal communication, 1968).

Design methods

The actual design of a facultative pond depends on a great variety of local conditions, but a number of useful and rational design procedures are available. Four such procedures are described in this section. They are based on (1) load per unit area; (2) the empirical procedure; (3) South African practice; and (4) suggestions made by the Indian Central Public Health Engineering Research Institute (CPHERI). Where little is known about the climatic conditions, the empirical procedure is recommended.

1. *Load per unit area procedure*

Experience has shown that certain generalizations can be made concerning the acceptable organic load of a facultative waste stabilization pond. Table 17 shows BOD loading values that have been used successfully in various geographical areas, but obviously great care must be exercised in using these values for design purposes.

Unpleasant odours may occur as a result of extremes in seasonal temperatures, inadequate surface area, uneven distribution of settleable solids, or inadequate liquid depth (Fisher et al., 1968; Horn, 1965; Svore, 1968). The distribution of settleable solids can be improved through the use of multiple inlets and deeper ponds (1.5–2 m). Maintenance of liquid depth depends principally on the control of seepage and evaporation. Understandably the operation of the entire pond system is dependent on the volumetric loading, BOD loading per unit area, and the concentration of organic matter in the waste-water. Where the water content is low it may be difficult to maintain sufficient liquid over the settled solids. In this case, water must be added or the pond should be designed to function without any overflow.

TABLE 17

GENERALIZED BOD LOADING PER UNIT AREA PER DAY UNDER VARIOUS CLIMATIC CONDITIONS

Surface loading (kg BOD_5/ha per day)[a]	Population per ha[b]	Detention time (days)[c]	Environmental conditions
Less than 10	Less than 200	More than 200	Frigid zones, with seasonal ice cover, uniformly low water temperatures and variable cloud cover
10- 50	200-1000	200-100	Cold seasonal climate, with seasonal ice cover and temperate summer temperatures for short season
50-150	1000-3000	100- 33	Temperate to semi-tropical, occasional ice cover, no prolonged cloud cover
150-350	3000-7000	33- 17	Tropical, uniformly distributed sunshine and temperature, and no seasonal cloud cover

[a] These estimates are based on the assumption that the effluent volume is equal to the influent volume, i.e., the sum of the evaporative and seepage losses is not greater than rainfall.

[b] Assuming a contribution of 50 g BOD_5 per person per day in developing areas.

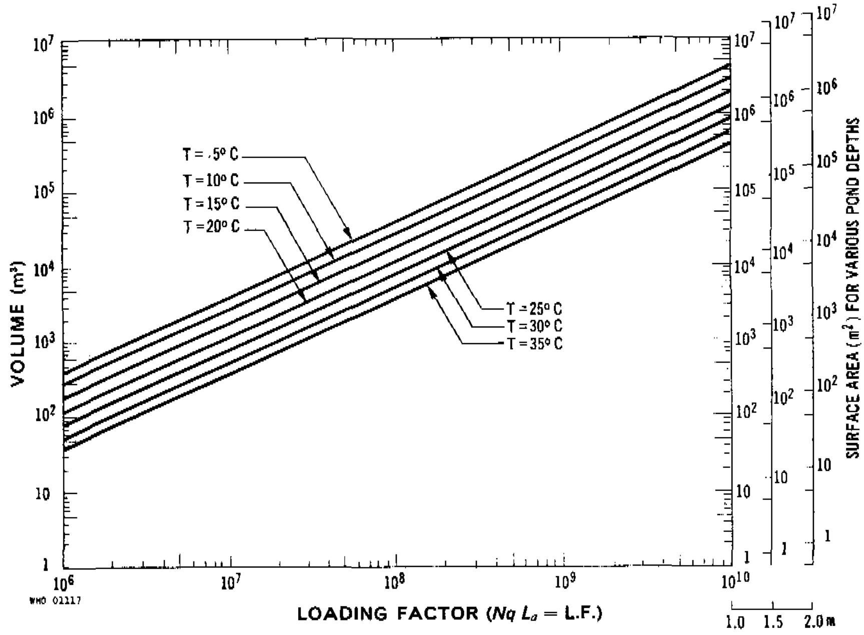
[c] Based on an influent volume of 100 litres of waste per person per day.

2. *Empirical procedure*

After observing the results of many small laboratory ponds, larger pilot plants, and over 200 operating ponds, the author of this monograph and his collaborators have developed formulations that can be used to relate pond volume to temperature, BOD, influent volume and toxicity. The first of these equations, in which the estimated BOD removal efficiency is 85–95 %, was published by Hermann & Gloyna (1958).

The required volume of a facultative pond can be estimated with the aid of Fig. 13. A loading factor (L.F.) is computed by multiplying the number of persons contributing waste (N) by the product of the *per caput* contribution in litres per day (q) and the BOD (L_a). (BOD_5 should be used for weak or presettled waste-waters, BOD_u for strong or untreated waste-waters.) Field observations have shown that if the average water temperature of the coldest month of the year is known and cloud cover does not last for more than a few days at a time, a reasonably accurate design can be developed from this graph. A vertical line is drawn from the calculated loading factor to the selected temperature; the required volume in m^3 can then be read off on the left ordinate, and the required surface area on the right ordinate. Recommended depths are considered later (Table 19).

FIG. 13. GRAPH FOR CALCULATING THE REQUIRED VOLUME AND SURFACE AREA OF A FACULTATIVE POND *



* Pond depth ⩾ 1 m. Influent volume must be sufficient to maintain liquid depth.
T = average water temperature of coldest month (°C)

Light intensity, wind, total dissolved solids, and a host of other factors may influence the required pond volume, but where the waste-water is domestic sewage temperature is usually the most important single consideration (Suwannakarn & Gloyna, 1964). The basis of Fig. 13 is:

$$V = (3.5 \times 10^{-5}) N q L_a \theta^{(35 - T_m)} \qquad (13)$$

where

V = pond volume (m³)
N = number of people contributing waste
q = *per caput* waste contribution (litres/day)
θ = temperature reaction coefficient = 1.085
T_m = average water temperature of coldest month (°C)

The volume and surface area must be increased for wastes containing more than 500 mg/litre of sulfate ion (Gloyna & Espino, 1969). Similarly, a larger volume is required for some industrial wastes because they are not readily biodegradable, because they are toxic to the algae, or because they reduce the chlorophyll content (Huang & Gloyna, 1968; Thirumurthi & Gloyna, 1965).

If specific *per caput* contributions of BOD and waste volume are assumed, Fig. 13 may be used for other purposes. For example, a new graph can be drawn to show the permissible population contributions to a given pond under different temperature conditions. If the following assumptions are made:

Area = 1 ha
Depth = 1.5 m
$\mathrm{BOD}_u(L_a)$ = 300 mg/litre
Waste volume (q) = 100 litres *per caput* per day

the loading factor can be obtained from Fig. 13 and the population per ha can be calculated by solving for N (Table 18).

TABLE 18

SPECIMEN DATA FOR CALCULATING PERMISSIBLE WASTE DISCHARGES INTO FACULTATIVE PONDS, BASED ON FIG. 13

Loading factor (L.F.) = NqL_a	T_m (°C)	$N = \frac{L.F.}{qL_a}$
4×10^7	5	1.3×10^3
6×10^7	10	2×10^3
9×10^7	15	3×10^3
1.5×10^8	20	5×10^3
2×10^8	25	6.7×10^3
3×10^8	30	1×10^4
4.2×10^8	35	1.4×10^4

It is difficult to write a generalized equation that describes the influence of light intensities, but this factor must be considered. Surface area must be increased where prolonged cloudy weather is known to occur. The use of an anaerobic pretreatment unit might be advisable where complete cloud cover may persist for periods of two or more weeks.

Values for insolation are not included in the empirical design equation, but a correction factor for this may be applied, since insolation and even temperature are usually related to the degree of latitude. For probable values of insolation at various latitudes, the reader is referred to Table 27 (Annex 2).

One reason why no precise figures can be given for light and depth relationships is that wind and temperature gradients help to mix the contents of ponds. As a result of this mixing the algal mass, above the thermocline at least, is exposed to usable light.

The appropriate pond depth is determined by environmental conditions, by the type of waste to be treated, and by the general safety factors desired.

When the required volume has been estimated in accordance with Equation 13, the depth is selected with the aid of Table 19. If the pond is too shallow (less than 1 m), emergent vegetation may destroy it and unpleasant odours may develop during the hot season. Depths in excess of 1.5 m are for sludge storage or excess capacity during cold weather.

TABLE 19

RECOMMENDED DEPTHS OF FACULTATIVE PONDS IN RELATION TO ENVIRONMENTAL CONDITIONS AND TYPE OF WASTE

Recommended depth (m)	Environmental conditions and type of waste
1.0	Uniform warm temperature; presettled waste-water
1.0 - 1.5	Uniform warm temperature; untreated waste-water
1.5 - 2.0	*Moderate seasonal temperature fluctuations;* raw waste-water containing settleable solids
2.0 - 3.0	Wide seasonal temperature variations; large amounts of settleable grit or settleable solids

Specimen calculations of pond design using the Gloyna procedure are shown below. Alternative solutions are provided to show the influence of temperature.

Given

A locality where the waste-water consists mainly of domestic wastes

Influent BOD_5, 20°C	250 mg/litre
Influent BOD_u (L_a)	300 mg/litre (3×10^{-4} kg/litre)
Population contributing waste (N)	20 000
Contribution *per caput* (q)	150 litres/day
Flow (Nq)	3 000 m^3/day (3×10^6 litres/day)
Average water temperature of coldest month (T_m)	10°C
Temperature coefficient (θ)	1.085
Percolation rate	negligible
Evaporation	= rainfall
SO_4^{--}	less than 500 mg/litre
Industrial toxicity	negligible

Required

Calculations showing (*a*) organic load; (*b*) volume; (*c*) depth; (*d*) surface area; (*e*) detention time; and (*f*) surface loading.

Solution

(*a*) *Organic load*

$$= BOD_u \times Nq$$
$$= 3 \times 10^{-4} \times 3 \times 10^6$$
$$= 900 \text{ kg } BOD_u/\text{day}$$

(*b*) *Volume* (Equation 13)

$$= (3\cdot5 \times 10^{-5})\, NqL_a\theta^{(35-T_m)}$$
$$= 3.5 \times 10^{-5} \times 3 \times 10^{6} \times 300 \times 1.085^{(35-10)}$$
$$= 2.4 \times 10^{5}\ \text{m}^3\ (0.7 \times 10^{5}\ \text{m}^3)^{a}$$

(*c*) *Depth*

$$= 1.75\ \text{m}^{b}$$

(*d*) *Surface area*

$$= \frac{\text{volume}}{\text{depth}}$$
$$= \frac{2.4 \times 10^{5}}{1.75}$$
$$= 1.4 \times 10^{5}\ \text{m}^2 = 14\ \text{ha}\ (4.1 \times 10^{4}\ \text{m}^2 = 4.1\ \text{ha})^{a}$$

(*e*) *Detention time*

$$= \frac{\text{volume}}{\text{flow per day}}$$
$$= \frac{2.4 \times 10^{5}}{3000}$$
$$= 80\ \text{days}\ (24\ \text{days})^{a}$$

(*f*) *Surface loading*

$$= \frac{\text{organic load}}{\text{surface area}} = \frac{900}{14}$$
$$= 65\ \text{kg BOD}_u/\text{ha per day}\ (222\ \text{kg BOD}_u/\text{ha per day})^{a}$$

3. *South African procedure*

While developing criteria for the design of facultative ponds in southern and central Africa, Marais & Shaw (1961) noted that (*a*) with low wastewater flow and a large surface area *per caput*, the pond did not attain the required liquid depth on account of losses through seepage and evaporation; (*b*) the effluent BOD remained substantially constant irrespective of the seasonal variations in temperature; (*c*) BOD loadings per unit area could be increased above those frequently reported in the literature; and (*d*) anaerobic conditions developed in the summer, while during the winter the ponds appeared to operate satisfactorily.

In South Africa the maximum BOD load for a facultative pond was initially related to the depth:

$$L_p = \frac{1\,000}{(0.18\, d + 8)} \tag{14}$$

where

$$d = \text{depth (m)}$$

Marais & Shaw later reduced the dividend in this equation from 1000 to 750. On the basis of a study of the long-term behaviour of the ponds, Shaw, Meiring & van Eck (1962) reduced this value still further to 600, so that Equation 14 is now written as follows:

---

[a] The solutions in parentheses are for the case where T = 25°C. It should be noted that uniformly warm climates assist in reducing the size of the facultative pond. If the solids content is relatively high, anaerobic pretreatment is required. Provided that climatic conditions are uniform, that sunlight is uniform and intense, that evaporation and seepage are equal to rainfall, and that the solids-to-liquid ratio is low, the depth of the pond could be reduced to 1 m.

[b] Assuming specific climatic conditions and type of waste (see Table 19).

$$L_p = \frac{600}{(0.18\,d + 8)} \tag{15}$$

Field data indicated that the effluent BOD could be formulated in terms of Equation 16. By setting the value of K_T in Equation 9 at 0.17, it is possible to estimate the BOD in the effluent (L_p) if the initial BOD (L_o) and the detention time (R_T) are known. Similarly, it is possible to determine R_T if both L_p and L_o are known.

$$L_p = \frac{L_o}{0.17\,R_T + 1} \tag{16}$$

The effluent BOD_5 from a pond with a depth of 2 m and designed in accordance with Equations 15 and 16 will be approximately 60 mg/litre. If this level is not acceptable, additional treatment in secondary and tertiary ponds may be required. These extra ponds should be designed so as to provide a detention time of 7 days or more per pond. Field experience has shown that the effluent BOD_5 from the series-connected ponds should not exceed 20 mg/litre. On no account should the total area of series-connected ponds be averaged to obtain the acceptable BOD load for the first pond.

A specimen calculation of facultative pond design using the South African procedure is given below. This example may be particularly applicable for subtropical areas and domestic wastes containing a high solids-to-water ratio.

Given

A domestic waste-water:
$BOD_5(L_o)$ = 250 mg/litre (2.5×10^{-4} kg/litre)
$BOD_u(L_a)$ = 300 mg/litre (3×10^{-4} kg/litre)
Flow = 3 000 m^3/day (3×10^6 litres/day)

Required

Calculations showing (*a*) expected effluent BOD_5 concentration, (*b*) detention time, (*c*) surface area, and (*d*) surface loading.

Solution

(*a*) On the basis of Equation 15 and assuming a depth of 1.75 m, the effluent BOD_5 (L_p) is:

$$\frac{600}{0.18\,d + 8} = \frac{600}{(0.18)(1.75) + 8} = 72 \text{ mg/litre}$$

(*b*) On the basis of Equation 9, the detention time (R_T)

$$= \left(\frac{L_o}{L_p} - 1\right)\frac{1}{K_T} = \left(\frac{250}{72} - 1\right)\frac{1}{0.17} = 14.5 \text{ days}$$

(*c*) Surface area

$$= \frac{\text{volume}}{\text{depth}}$$

$$= \frac{\text{flow} \times \text{detention time}}{\text{depth}}$$

$$= \frac{3\,000 \times 14.5}{1.75} = 25\,000 \text{ m}^2$$

$$\simeq 2.5 \text{ ha}$$

(*d*) Surface loading

$$= \frac{\text{organic load}}{\text{area}}$$

$$= \frac{\text{BOD} \times \text{flow}}{\text{area}}$$

$$= \frac{2.5 \times 10^{-4} \times 3 \times 10^6}{2.5}$$

$$= 300 \text{ kg BOD}_5\text{/ha per day}$$

If these results are compared with those obtained by the empirical procedure; it can be seen that there is considerable uniformity. In both cases, the settleable solids must never be exposed to the air and sunshine. There must be adequate liquid present to fill the ponds and ensure satisfactory operation, otherwise alternative pretreatment is required.

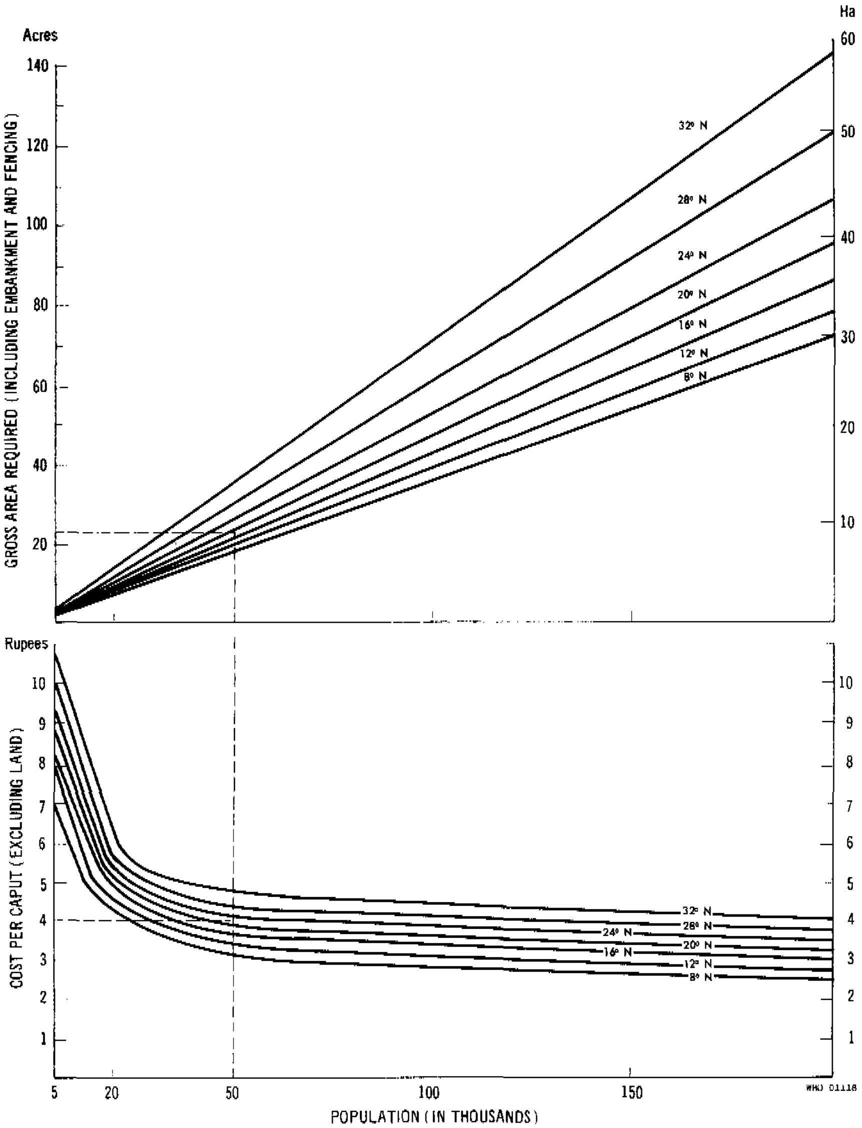
4. *Procedure based on suggestions by the Central Public Health Engineering Research Institute (CPHERI), India*

In tropical India, the design of facultative waste stabilization ponds for domestic waste-waters has been based on a loading of 336 kg BOD_5/ha per day and an operating depth of 1.22 m (Dave & Jain, 1967). A study of 8 ponds has shown a BOD reduction for this loading of 70–85 %. It is noteworthy that this organic loading is considered unattainable in many geographical areas, even though summer temperature conditions approach those found in India. In some cases, moreover, BOD removals in excess of 70–85 % are required. To increase the BOD removal to 90 % usually requires a significant increase in detention time. The average sludge build-up appears to be as little as 2–4 cm per year; this low value may be associated with the uniform and relatively high temperatures. Nevertheless, Indian practice suggests that it is desirable to have 2 ponds wherever possible to facilitate servicing, cleaning, etc.

CPHERI prefers the multi-cell pond system (parallel units) when the flow exceeds 2 250 000 litres/day. This use of small ponds contrasts with experiences in the USA and South America, where wind action on larger ponds has helped to maintain desirable conditions.

Another design has been suggested by Arceivala & Desai (personal communication, 1968). In Fig. 14, population is plotted against gross pond area and against cost for various latitudes. This graph takes into account such

*FIG. 14. REQUIRED AREA AND COST OF PONDS FOR DIFFERENT POPULATIONS AT VARIOUS LATITUDES, INDIA, 1969**



* Reproduced, in modified form, from Arceivala et al. (1969) by permission of the publishers.

* Exchange rate (1969): US $ 1.— = Rs 7.50.

Example: For a population of 50 000 located at 20° N, the chart shows the cost of pond construction to be Rs 4.— per caput, and the gross land requirement to be 23.10 acres. If the cost of land is Rs 8 000 per acre, the total pond cost will be:
Rs 4.— × 50 000 + 8 000 × 23.1 = Rs 384 800.

factors as oxygen production, conversion of solar energy, and temperature. It is understood that these two approaches to pond design provide reasonably similar results.

Calculations for the design of a facultative pond, using the procedure suggested by CPHERI, are shown below:

Given

Population = 10 000 persons
Water supply = 227 litres *per caput* per day
Waste-water flow = 1.82×10^6 litres/day (80 % of water supply)
Influent BOD_5 = 300 mg/litre
Suspended solids = 300 mg/litre
Width-to-length ratio = 1 : 2
Operational depth = 1.22 m
Organic loading per hectare = 336 kg BOD_5/ha per day

Required

Calculations showing (*a*) total organic load and (*b*) surface area.

Solution

(*a*) Total organic load

$$= \text{flow} \times \text{influent } BOD_5$$
$$= 1.82 \times 10^6 \times 300 \text{ mg/day}$$
$$= 546 \text{ kg/day}$$

(*b*) Surface area

$$= \frac{\text{total organic load}}{\text{organic load per ha}}$$
$$= \frac{546}{336} = 1.63 \text{ ha}$$

Shape and depth of facultative ponds

The length-to-width ratio of the ponds may vary from 1 : 1 to 2 : 1. Fluctuations in seasonal temperatures will make deeper ponds necessary. The findings of Svore (1968) and Oswald (1968) and observations of ponds in Texas, USA, show that ponds only 75 cm deep do not provide the best stabilization. Because of their shallowness, the ponds may attain high temperatures during the day and this may adversely affect algal growth. Moreover, the sludge layer in shallow ponds may rise or turn septic and cause bad odours. Oswald (1968) reported that odours occurred when the liquid depth was 60 cm, but not when the depth was increased to 2 m.

Sludge layer

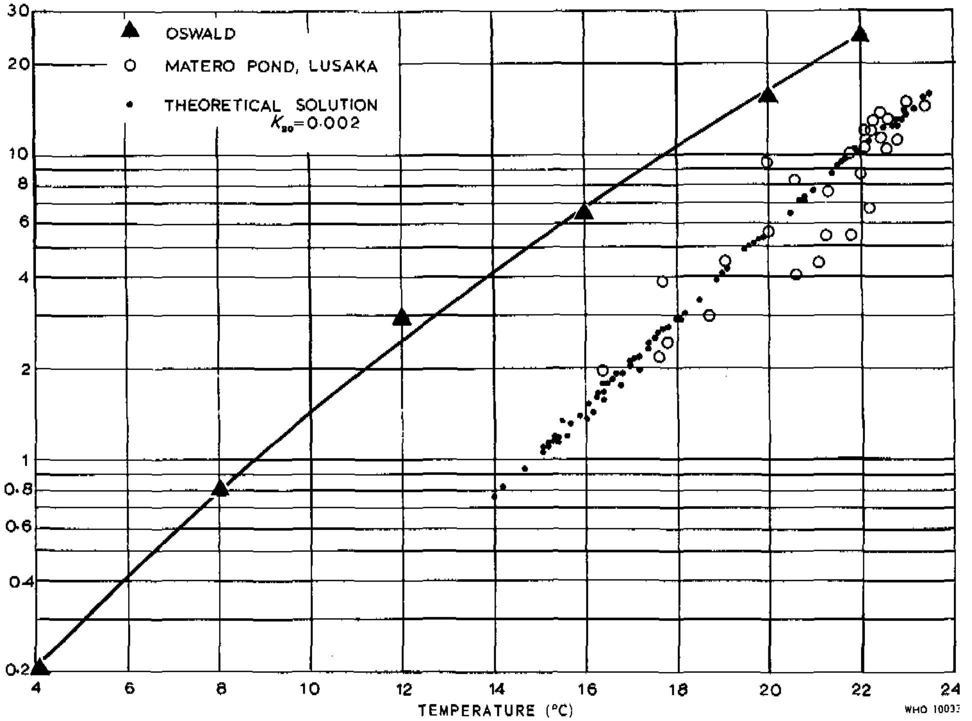
The rate of gas evolution from the sludge layer is a sensitive measure of the biological activity in the bottom layers of a facultative pond. In Fig. 15 the gas evolution from digesting sludge is plotted against temperature. There is approximately a fourfold increase in gas evolution for every 5°C rise in temperature between 4° and 22°C. The digestion rate can be estimated with the aid of Equation 17 (G. v. R. Marais—personal communication, 1968):

$$K_{s_{(T)}} = 0.002\,(1.35)^{-(20-T)} \tag{17}$$

where

$K_{s_{(T)}}$ = digestion rate (rate of evolution from the sludge layer)

FIG. 15. EXPERIMENTAL AND THEORETICAL RELATIONSHIPS BETWEEN TEMPERATURE AND GAS EVOLUTION PER DAY FROM SLUDGE LAYER IN A PRIMARY OXIDATION POND *



Reproduced from Marais (1966).

* It will be noted that the points representing the theoretical solution do not lie on a smooth curve. This is because two points at the same temperature may relate to different parts of the yearly temperature cycle; owing to the change in the volume of sludge in the time interval, the amounts of gas evolved are not identical.

The influence of the sludge layer on the pond can be described as follows. During the early life of a primary facultative pond receiving settleable solids, a portion of the observed reduction of BOD in the pond is due to the settlement of organic material as sludge. Anaerobic fermentation develops in the sludge, reducing its organic load but releasing fermentation products to the pond liquid. Under uniform conditions of deposition and temperature the volume of sludge will increase, releasing correspondingly greater quantities of fermentation products, until a state of equilibrium is reached in which the BOD added to the sludge by new deposits is equal to the BOD released as fermentation products.

During the cold season, little breakdown takes place in the sludge and the pond bottom becomes primarily a sludge storage area. During the hot season, the fermentation products from the sludge add to the BOD in the pond liquid. Eventually a condition is attained where there is an annual cyclic variation in sludge volume, high in the winter and low in the summer. During the summer the increased feedback of fermentation products from the sludge to the pond liquid may exceed the reoxygenation capacity of the pond and turn it anaerobic. This is particularly serious if the warmer temperatures coincide with cloud cover that reduces solar radiation.

In the tropics and subtropics, high BOD surface loading rates can be utilized. If water consumption *per caput* is low, however, the sludge will occupy a greater proportion of the pond volume and its influence may become a dominant factor in design.

Mixing

Several factors are usually responsible for the mixing patterns that develop in facultative ponds. Wind and heat are probably the most important. Mixing by wind energy depends on the surface area of the pond. The maximum effect of the wind can be obtained when there is an unobstructed wind path of 100–200 m. Thus, smaller ponds are more likely to remain stratified than larger ones.

The effect of mixing on pond behaviour is profound. Mixing during daylight hours distributes the temperature and oxygen uniformly throughout the body of the pond, thereby inducing aerobic conditions throughout the pond depth. Mixing also transports non-motile algae into the region where usable light energy is available. During long periods of stratification, the non-motile species gradually settle on the sludge layer, resulting in a marked decrease in the algal concentration in the liquid. Only motile forms survive in any number, and this may be why the motile *Euglena* is sometimes dominant. It has been observed in several countries that ponds may be green in the morning because the *Euglena* algae are near the surface, but turn grey as the day progresses because the algae move out of the intensely sunlit and heated top layers. Typically, the motile algae may form a dense layer a few centimetres thick above the thermocline. The thermocline is the layer of water separating an upper warmer zone from a lower colder zone.

Data on thermal mixing have been obtained in several localities (Fisher & Gloyna, 1963; Marais, 1966).

> Typically, during the morning, if there is any wind, there is a period of complete mixing in which the temperature is uniform throughout the pond but, owing to the absorption of radiation, [the temperature] gradually increases. At some time, usually

> during a short lull in the wind, stratification develops abruptly and a thermocline forms. The temperature above the thermocline increases to a maximum and then decreases, while below the thermocline the temperature rapidly falls to a value approximately that of the earth and thereafter remains practically constant. . . .
>
> In the afternoon and evening, a second period of mixing may be initiated as follows:
>
> (*a*) Above the thermocline, under quiescent wind conditions, the top layers lose their heat more rapidly than the bottom layers. The cooler top layers sink, inducing mixing, with the result that the temperature down to the thermocline remains approximately uniform but gradually decreases. The thermocline gradually sinks and, . . . with further cooling, mixing [may be] initiated and sustained throughout the pond.
>
> (*b*) Under windy conditions, usually during the period of decreasing temperatures, the energy imparted by the wind to the water above the thermocline at some stage overcomes the stratification forces, [causing the temperature to be uniform throughout].
>
> It is possible that during summer, when the wind speeds are low, cooling by radiation is not sufficient to equalize the temperatures throughout the pond and the thermocline persists.[1]

In some semi-tropical areas, therefore, the thermocline may oscillate daily, but on the average it is less pronounced during the summer than during the winter.

Anaerobic Ponds

Anaerobic pretreatment of untreated domestic waste-waters significantly affects the behaviour of the primary facultative waste stabilization pond. It reduces the BOD load on the facultative pond and changes the nature of the settleable solids in the sludge layer. The sludge in a facultative pond that is preceded by an anaerobic pond has a reduced fermentation potential, so that little sludge rises during the hot season. Furthermore, the sludge is of a granular consistency and tends to break up on rising to the surface. In contrast, extensive drifting sludge of a sticky consistency may form in facultative ponds receiving untreated domestic wastes.

The main disadvantage of anaerobic treatment is the possibility of uncontrolled odour development. Consequently, the proximity of anaerobic ponds to dwellings is a factor in design that must always be considered.

Anaerobic breakdown in septic tanks, aqua privies, and anaerobic ponds appears to be identical. The reduction of BOD is a function of the retention time of the solids (i.e., as compared to the detention time of the soluble fraction).

The establishment of stable fermentation conditions in an anaerobic unit may take a considerable time, depending on the temperature. In a subtropical area, with an average annual temperature of about 20°C, BOD measurements in 150 aqua privy tanks have shown that approximately 4 months must elapse before the tanks deliver a stable effluent (Marais, 1966). If the average temperature is lower, this may take considerably longer. An indication of the relationship between temperature and digestion is provided by Fig. 15.

---

[1] Marais (1966).

Design

Southern African practice

The results of experiments with septic and aqua privy tanks in the USA (Weibel et al., 1949) and in Zambia (Vincent et al., 1963) indicate that in tropical and subtropical regions the reduction in BOD can be approximated by Equation 18 (Vincent et al., 1963). An influent and pond temperature of 20°C is assumed.

$$L_p = \frac{L_o}{K_n \left(\frac{L_p}{L_o}\right)^n R + 1} \tag{18}$$

where

R = detention time for completely mixed system (days)
n = exponent, to be determined by experimentation (for Zambia, $n = 4.8$)
K_n = design cœfficient

Often no data on the influent BOD are available and estimates of the influent BOD contribution and waste flow *per caput* must be based on past experience. Table 20 provides a rough estimation of the quantity of body wastes produced daily. For design purposes, total excreta may be taken to be about 1 kg per person per day (Wagner & Lanoix, 1958). Vincent et al. (1963) reported a daily BOD_5 contribution *per caput* of 0.036 kg in low-cost, high-density housing areas of Zambia; Van Eck (1965) reported 0.03 kg from similar areas in Durban, South Africa. In the USA a figure of 0.073 kg is used, and in other countries the level is probably about 0.05 kg.

TABLE 20

ESTIMATES OF THE QUANTITY OF HUMAN WASTES PRODUCED DAILY IN VARIOUS PARTS OF THE WORLD

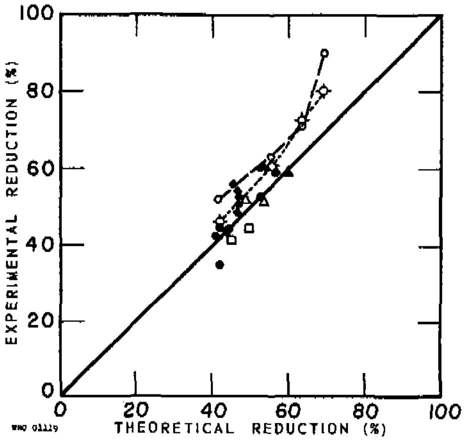
Geographical area	Wastes (in g per person per day)			Reference
	Human faeces (wet weight)	Urine	Urine and cleansing water	
World-wide summary	135-270	1000-1300		Gotaas, 1956
India	400		2300	
Tropics	280-530	600-1130		Macdonald, 1952
Philippines	665[a]			Wagner & Lanoix, 1958
Asia	200-400			Wagner & Lanoix, 1958
European and American countries	100-150			Wagner & Lanoix, 1958

[a] Including some urine.

It seems probable that domestic waste-waters in the developing areas have low *per caput* BOD values because very little garbage is disposed of through the sewer. An estimate of 0.05 kg BOD_5 *per caput* per day is probably on the high side but will serve as a safe estimate for developing countries. Effluent flow in these areas varies widely within a range of 20–150 litres per person per day. In developed industrial areas, the range may be many times greater than these levels. Usually an estimate can be based on the consumption of other communities in the area.

Correlation plots between observed BOD reduction and theoretical reduction calculated according to Equation 18 are shown in Fig. 16. Table 21 illustrates the relationship between theoretical BOD reduction and detention time under conditions typical of Zambia. Under such conditions there is little advantage in extending the detention time beyond one day. Normally 30–40 % BOD removal can be expected from sedimentation alone, although this settled organic matter would still have to undergo some form of biological breakdown.

FIG. 16. EXPERIMENTAL AND THEORETICAL REDUCTION OF BOD CONCENTRATION IN SEPTIC TANKS, AQUA PRIVIES AND ANAEROBIC PONDS



Reproduced, by permission, from Vincent et al. (1963), p. 160.

◆ ● Septic tank with single compartment
△ Septic tank with 4 compartments: effluent from 4th
▲ Septic tank with 2 compartments: effluent from 2nd
◇ ○ Anaerobic ponds (4 in series)
□ Aqua privy with single compartment

The rate of breakdown decreases sharply with reduction in temperature, and fermentation is negligible at temperatures below about 15°C, when the anaerobic pond serves principally as a sedimentation tank (Oswald, 1968) and becomes in effect a sanitary landfill. Van Eck (1965) found no reduction of BOD in anaerobic ponds during the winter months when the temperature was below 10°C, reductions of 40–60 % at 20°C, and reductions exceeding 80 % at temperatures of 25°C and above. Parker et al. (1950) also observed that ponds in Melbourne, Australia, with liquid detention times during the summer of about 1.2 days gave BOD reductions of 65–80 % but that during the winter months, although the detention time was increased to 5–7 days, the BOD reduction dropped to 45–60 %.

TABLE 21

THEORETICAL BOD REMOVAL IN ANAEROBIC PONDS *

Detention time (days)	BOD remaining (%)	BOD reduction (%)
0.12	80	20
0.40	70	30
0.71	65	35
1.3	60	40
2.4	55	45
4.7	50	50
9.4	45	55

* Data based on Equation 18, taking $n = 4.8$ and $K_n = 6$ at 22°C.

Specimen calculations for the design of an anaerobic pond in a subtropical area are shown below:

Given

Population = 8 000
Influent BOD_5 (L_o) = 400 mg/litre
Effluent BOD_5 (L_p) = 240 mg/litre
K_n = 6.0
n = 4.8
Waste production = 70 litres *per caput* per day

Required

Recommendations showing (*a*) detention time, (*b*) pond volume, (*c*) pond area, (*d*) number and arrangement of anaerobic ponds, and (*e*) additional treatment.

Solution

(*a*) Detention time, R, based on Equation 18

$$L_p = \frac{L_o}{K_n\left(\frac{L_p}{L_o}\right)^n R + 1}$$

$$\text{or } R = \left(\frac{L_o}{L_p} - 1\right)\left[\frac{1}{K_n\left(\frac{L_p}{L_o}\right)^n}\right]$$

$$= \left(\frac{400}{240} - 1\right)\frac{1}{6\left(\frac{240}{400}\right)^{4.8}} = 1.3 \text{ days}$$

(*b*) Volume = days × population × waste *per caput*
= 5 × 8 000 × 70 litres
= 2 800 m^3

(*c*) Area (assuming depth of 2 m) $= \frac{\text{volume}}{\text{depth}} = \frac{2\ 800}{2} = 1\ 400\ m^2$
= 0.14 ha

(*d*) Number and arrangement of ponds = 2 in parallel

(*e*) Additional treatment: Can be provided by a facultative waste stabilization pond or a facultative pond connected in series with a maturation pond

The following specimen calculations relate to the design of an anaerobic pond to treat the effluent from a low-cost, high-density housing area:

Given

Population = 8 000
Waste = 70 litres *per caput* per day
Soil = impermeable
Lowest monthly average temperature = 10°C

Required

Calculations showing (*a*) pond volume, (*b*) pond area, and (*c*) effluent quality.

Solution

On account of the low temperature, a detention time of 5 days is used.

(*a*) Volume

$$= \text{days} \times \text{population} \times \text{waste } \textit{per caput}$$
$$= 5 \times 8\,000 \times 70 \text{ litres}$$
$$= 2\,800 \text{ m}^3$$

(*b*) Area (assuming depth of 2.5 m)

$$= \frac{\text{volume}}{\text{depth}} = \frac{2\,800}{2.5} = 1\,120 \text{ m}^2$$
$$= 0.11 \text{ ha}$$

(*c*) Effluent quality

Assuming that influent BOD *per caput* per day = 0.05 kg,

$$\text{influent BOD per litre} = \frac{\text{BOD } \textit{per caput} \text{ per day}}{\text{wastes } \textit{per caput} \text{ per day}}$$
$$= \frac{0.05 \text{ kg}}{70 \text{ litres}}$$
$$= 714 \text{ mg per litre}$$

Assuming 60 % BOD removal in winter due to sedimentation and some biodegradation,

$$\text{BOD in effluent} = \frac{714 \times 60}{100} = 428 \text{ mg per litre}$$

Design guide-lines used in Israel

Designers in Israel systematically use anaerobic ponds for the pretreatment of domestic waste-waters (Amramy et al., 1962; Watson, 1962). More recently, studies have been completed on some deep ponds (Wachs & Berend, 1968).

The general guide-lines used by the Israel Ministry of Health are as follows (H. I. Shuval—personal communication, 1966):

Primary anaerobic pond: 125 kg BOD per 1 000 m^3 per day;
Secondary aerobic pond: 15–20 kg BOD per 1 000 m^3 per day.

Pond depths are usually 1.2–1.5 m.

Design used by the Melbourne Water Science Institute, Australia

The ratio between the area of facultative and anaerobic ponds appears to be a significant design factor in Australia (Parker, 1962), where facultative: anaerobic ratios between 10 : 1 and 5 : 1 are found useful. Ponds with a low ratio of 3 : 1 are sensitive to short-term changes in the BOD. The city of Melbourne uses a summer detention period of 1.25 days (1 120–2 240 kg BOD/ha per day) in the anaerobic ponds and 7.5 days in the facultative ponds. In winter the detention periods are increased almost fourfold. The BOD reduction is reported to be 65–80 % in summer and 45–65 % in winter. Detention in anaerobic ponds is not extended beyond 5 days because the ponds then act as facultative ponds. Sludge removal from the anaerobic ponds is necessary.

Type and location of anaerobic ponds

The type of anaerobic facility chosen depends upon the magnitude of the waste flow and the proximity of the ponds to dwellings. Open anaerobic ponds are indicated where the volume of waste-water is too great to be treated by septic tanks and where the pond system can be located at least 0.5–1 km from human habitation. In the tropics a liquid detention time of 1–5 days is recommended. Longer detention may cause the upper layers of the pond to become aerobic and reduce the obligate anaerobic conditions necessary for maximum efficiency. In temperate regions where the average monthly temperature during the winter is less than 10°C, the liquid detention time during the winter should be about 5 days, and the sludge retention time should be based on a digestion reaction rate at 15°C.

Grit banks

Where combined sewers for domestic wastes and storm run-off are used and where the wastes contain large amounts of sand and grit, it is necessary to remove these deposits periodically. To provide continuous treatment, therefore, anaerobic pond units should be built in duplicate. Where the installation consists of a number of parallel units, one additional pond must be provided so as to permit those in operation to be taken out of commission, one at a time, for desludging.

Inlets and outlets

Inlet and outlet designs for anaerobic ponds present problems. Combined sewer systems carrying both storm run-off and domestic wastes may carry a large amount of grit, so that an inlet at the bottom of the pond is not practical. Nevertheless, it is desirable to keep the inlet submerged because this will relieve the floatable solids problem: many solids that would otherwise float will stay submerged if introduced near the bottom of the pond or at least at mid-depth. For small anaerobic ponds less than 0.5 ha in area a single inlet is sufficient. Larger ponds require several inlets so as to spread the grit and sludge load.

Outlets may be of the surface type, but a scum board must be provided to prevent the discharge of drifting sludge.

Sludge mats

During the winter months when the fermentation rates are low, "mats" of sludge are likely to form on the surface of the pond; as a result, hydrogen sulfide odours may become a nuisance and fly-breeding in the drifting sludge is likely. During warm weather intense fermentation will break up the drifting sludge, the bottom sludge layers will decrease in thickness, and there will probably be no odours. Hydrogen sulfide production has been effectively stopped by recycling up to 40 % of the effluent from the secondary or tertiary ponds to the influent line (Van Eck, 1965). This solution has also been advocated by Abbott (1962) and Oswald (1968). Where the temperatures in the anaerobic ponds are above 20°C, the ponds normally remain free of floating sludge throughout the year. It has been noted that the BOD reduction increases as the sludge layer increases (Parker et al., 1950; Weibel et al., 1949).

Because open anaerobic ponds have a large exposed surface, heat loss is considerable and they are subject to large temperature fluctuations. In contrast, covered septic tanks tend to retain the temperature of the influent waste-water. Heat loss from the surface of open ponds can be reduced by increasing the depth and thereby increasing the detention time for the same surface area (Oswald, 1968; Wachs & Berend, 1968).

Enclosed anaerobic facilities

In some cases it is desirable to pretreat the wastes in enclosed anaerobic units such as septic tanks. These units are discussed below because of their close association with various waste stabilization pond systems. Details of design for enclosed systems have been provided by the African Housing Board, Lusaka (Vincent et al., 1963).

For communities of 1 000–10 000 people, where the anaerobic waste stabilization pond system is located near houses, anaerobic pretreatment may be accomplished more effectively in enclosed tanks. A pretreatment tank facility for a population of about 8 000 has been put into operation in the suburb of Chelston, Lusaka. There two tanks are operated in parallel so that one may be taken out of commission if the tanks need to be desludged manually. Desludging can also be carried out by vacuum tanker or, if sufficient fall is available, by the installation of a siphon. If a siphon is used, it is advisable to withdraw the sludge at frequent intervals to prevent the sludge and grit mixture from becoming densely compacted. Sludge accumulation at the draw-off point can be encouraged by constructing the tank with a conical or sloping bottom. In areas with high water tables the tank must be designed to resist the resulting upward pressures. The liquid detention time in these large tanks is approximately 18–24 hours. Sludge accumulation can be determined on the basis of approximately 0.03–0.05 m^3 *per caput* per year.

For smaller communities (up to 1 000 people) rectangular pretreatment tanks are frequently provided, as these are usually easier to construct than circular ones. Sludge and grit are periodically removed by vacuum tanker or manual labour. In very small communities where vacuum tankers are available, single-chamber tanks may be installed. It is also desirable in such communities to increase the unit volume *per caput* of the tank so as to lengthen the interval between desludging. The tank is then designed on the basis of sludge accumulation, as shown in the following example:

Given

Sludge accumulation	0.03 m³ per person per year
Population	200
Frequency of desludging	every 5 years

Required

Volume of tank

Solution

Desludging is normally necessary when the tank is half-filled with sludge, therefore the tank volume should be $200 \times 0.03 \times 5 \times 2 = 60$ m³.

Anaerobic pretreatment of domestic wastes in aqua privies serving individual houses before discharge to sewers and waste stabilization ponds has been widely applied in Zambia (Vincent et al., 1963), where sewer systems have been built in areas with very flat topography. To ensure velocities adequate for self-cleaning in sewers carrying raw waste and grit, the gradients must give a velocity at maximum flow of about 1 m per second. With level terrain, provision would normally have to be made for deep cuts or lift pumps. However, where anaerobic pretreatment is applied at each home or group of homes, the sand and grit are retained in the tank and the organic solids are digested. Consequently, sewers conveying this effluent can be laid at gradients giving a velocity of about 0.3 m per second at maximum flow. This means that the sewers can remain fairly close to the surface and cuts down the work of trench construction.

The sewer network for Bwacha suburb in Kabse, Zambia, has a maximum average slope over the housing area of 1 : 400 and a minimum gradient of 1 : 780. This gradient, which is over a small distance, provides a velocity of only 0.25 m per second. While this velocity is not recommended for normal design, the system has worked satisfactorily for 3 years.

When the main sewer carries anaerobically pretreated waste, no raw waste-water should be discharged into it even if the rate of flow in the sewer is sufficient for normal self-cleansing, because the mixing of oxygen in the raw waste with the anaerobic waste may cause corrosion. As a precautionary measure, sewers carrying pretreated effluents are sealed by heavy-duty manhole covers that prevent the public from gaining access to them.

Aerobic Ponds

Aerobic waste stabilization ponds have a place in the planning of low-cost waste-water treatment systems. The design of aerobic ponds may be based on three concepts: (*a*) minimum depth and maximum algae production; (*b*) equalization of BOD removal and pathogen control (maturation ponds; and (*c*) induced mechanical mixing or aeration, which may support aerobic bacterial activity without necessarily involving photosynthetic processes.

Minimum depth and maximum algal production

The high-rate aerobic waste stabilization ponds are still in the experimental phase of development. The purpose of these ponds is to convert as much carbon dioxide into algal cell material as possible. In this manner maximum protein and oxygen production can be maintained. Designs of such ponds are based on large area-to-volume ratios, and under these conditions large amounts of algae may be grown. The organic loading in a continuously mixed pond may be in excess of 56 g BOD_5/m^2 per day. However, a more likely range of values is 10–35 g BOD_u/m^2 per day, with a removal efficiency of about 70 %.

If aerobic ponds are designed to produce oxygen in quantities exceeding the influent BOD, the contents must be mixed for about 3 hours per day to keep the settled sludge in an aerobic condition (Oswald et al., 1957). Mixing at velocities of about 50 cm per second brings an additional supply of nutrients to the surface, where they can be utilized most efficiently with the available light. It is essential with these ponds to skim and separate the algae from the effluent, for otherwise the organic load supplied to a stream by the algal cells may present a problem.

In the design of aerobic ponds, it is important to make an accurate estimate of the density of the algal cells and to relate oxygen production to the light conversion efficiency and light intensity (Oswald, 1963). Equation 19 states that oxygen production is directly related to light intensity.

$$O = 0.22\, F I_L \qquad (19)$$

where

O = oxygen production (kg/ha per day)
F = light conversion efficiency (%); values may range from 0.5 to 6.0
I_L = light intensity (cal/cm^2 per day)[1]

The energy balance in an aerobic pond can be reduced to a fairly simple relationship. Experimentally, it has been found that the production of

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[1] For values, see Table 27 (p. 154).

algal cells is related to the product of the light conversion efficiency and the solar radiation:

$$Y_a = 0.125\, FI_L \tag{20}$$

where

Y_a = yield of algal cells (kg algae/ha per day)

The ratio between oxygen production and algal cell production is about 1.6 : 1. The highest percentage BOD removal occurs when the oxygenation factor, which is the ratio of the oxygen produced to the oxygen required, is about 1.6. The energy balance thus becomes:

$$\frac{HO'}{1.6} = FI_L\, R\frac{10}{d} \tag{21}$$

where

H = heat of combustion of cells = 6 cal/mg
O' = oxygen production (mg/cm^3)

At present, a pond of this type would be uneconomical, for it would require an unusual amount of attention. In the future, when there may be a market for plant protein, this type of pond may have some merit.

Maturation ponds

The main purpose of maturation ponds is to provide a high-quality effluent. Maturation ponds are not intended to relieve facultative waste stabilization ponds or conventional biological treatment plants that have proved too small or to reduce expenditure on operating costs and supervision.

The principal factor in the design of a maturation pond is detention time, but for efficient reduction of the faecal bacteria it is essential that the pond be arranged in series with the preceding pond (Van Eck, 1961). The detention time in a maturation pond, as well as the number of ponds, is determined primarily by the degree of bacterial purification required. Usually the maturation pond should provide 7–10 days' detention, and a liquid depth of 1 m. The rate of disappearance of faecal bacteria in aerobic ponds follows the relationship given in Chapter 5.

Induced mechanical mixing and aeration

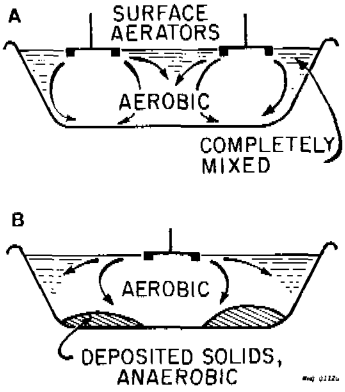
Low-cost mechanical aeration is a useful engineering alternative when waste loads increase, when space is limited, and when high-quality effluents are required. Aeration and mixing may be accomplished by (*a*) recirculation of oxygen-rich waters from the effluent of a facultative or maturation pond system to the influent area; (*b*) mechanically induced surface aeration; (*c*) diffused aeration using compressed air; or (*d*) combinations of the above techniques.

Recirculation of oxygen-rich pond waters helps to maintain an oxygen balance and eliminate odours in one or more series-connected ponds. Such recirculation of algae-rich waters and a form of stepped loading are employed extensively in South Africa and Sweden (Abbott, 1962).

A very successful and relatively economical aeration system is the mechanically aerated waste stabilization pond. Aerobic and facultative micro-organisms are supplied with oxygen by mechanical aerators (usually surface-type units), whereas in normal facultative ponds algae provide the oxygen. Inexpensive earthen ponds may be used.

Two types of pond using surface aerators are generally used (Fig. 17). Pond A is kept completely aerobic. All the solids are maintained in suspension, and the solids in the effluent are equal to the solids in the pond. Pond B is mostly aerobic, but the velocities and mixing characteristics permit much of the non-oxidized biological solids to settle to the bottom. These deposits must undergo anaerobic decomposition. In both cases, the effluent can be further treated in a facultative waste stabilization pond.

FIG. 17. TWO TYPES OF AERATED WASTE STABILIZATION POND



Surface aerators may be fixed in one position or used as floating units. Generally, 10–20 % of the total oxygen required results from oxygen transfer across the surface interface, and the rest is provided through mixing and entrapment of air. Mechanical aeration units of the surface type can be expected to supply about 1–1.8 kg oxygen/hph (horsepower-hour). The larger the pond, the lower the value.

The oxygen supplied by mechanical aeration is usually determined as follows:

$$O_s = O_m \frac{C_{sw} - C}{C_s} \times \alpha \times \theta^{(T-20)} F_A \tag{22}$$

where

O_s = oxygen supplied by mechanical aeration (kg oxygen/hph)
O_m = manufacturer's rating of aerator, usually 1.8–2.1 kg oxygen/hph
C_{sw} = O_2 saturation level in the pond at temperature T (mg/litre)
C = level of dissolved oxygen in pond (mg/litre)
C_s = O_2 saturation level of distilled water at 20°C (mg/litre)

α $= \dfrac{\text{overall transfer coefficient of the waste-water}}{\text{overall transfer coefficient of the tap water}}$
(typical values range between 0.6 and 1.1)
θ = temperature reaction coefficient = 1.02
F_A = correction factor for altitudes above about 1 200 m (use standard air density relationships)

Equation 23 can be used for calculating the oxygen requirements in an aerobic system with complete mixing. Such ponds need oxygen for the BOD exerted by the micro-organisms that are degrading the influent wastes and to degrade at least some of the accumulated biological volatile solids.

$$O_n = a'(L_o - L_p) + b'X_t \tag{23}$$

where

O_n = oxygen required (mg/litre)
a' = BOD removed and used to provide energy for growth, as fraction of BOD$_u$ or COD (varies from $\frac{1}{2}$ to $\frac{1}{1}$)
b' = endogenous respiration rate per day, as fraction of BOD$_u$ or COD ($\frac{\text{[illegible]}}{100}$ to $\frac{10}{100}$)
X_t = volatile suspended solids in mixed liquor (mg/litre)

In most ponds where only pretreatment or supplemental aeration is required, the quantity of biological solids can be maintained at a low level. Consequently, Equation 23 can be simplified as follows:

$$O_d = a''L_r \tag{24}$$

where

O_d = oxygen required (kg/day)
a'' = coefficient (values range from 0.7 to 1.4)
L_r = BOD removed (kg/day)

The following calculations illustrate the use of Equations 22 and 24 in the design of an aerobic pretreatment unit.

Given

Flow = 7 570 m^3/day
BOD$_5$ = 250 mg/litre
BOD$_u$ = 305 mg/litre
C = 1.5
C_s = 9.2
C_{sw} = 10.2
T = 15°C
a'' = 0.7
O_m = 1.6 kg O$_2$/gross hph at 20°C and when DO is zero
α = 0.90
K_T = 0.35 per day

Required

The size and number of mechanical aerators necessary to reduce the influent BOD by 50 %.[1]

Solution

(*a*) Organic load $= \text{BOD}_5 \times \text{flow}$
$= 250 \times 7\,570 \times 1\,000$ mg BOD/day
$= 1\,829.5$ kg BOD/day, say 1 900 kg/day

BOD to be removed $= 950$ kg/day

(*b*) Detention time (Equation 9) $- R_T$

$$= \frac{L_o - L_p}{K_T L_p}$$

$$= \frac{250 - 125}{0.35 \times 125} = 2.86 \text{ days}$$

(*c*) Volume = detention time (in days) × flow (per day)
$= 2.86 \times 7\,570 = 21\,600 \text{ m}^3$

(*d*) Surface area (assuming depth of 3 m)

$$= \frac{\text{volume}}{\text{depth}} = \frac{21\,600}{3} = 7\,200 \text{ m}^2$$

$= 0.72$ ha

(*e*) Oxygen requirement (Equation 24) $= O_d = a'' L_r = 0.7 \times 950$
$= 665 \text{ kg O}_2\text{/day} = 27.7 \text{ kg O}_2\text{/hour}$

(*f*) Mechanical aeration requirement (Equation 22)

$$= O_s = O_m \frac{C_{sw} - C}{C_s} \times \alpha \times \theta^{(T-20)}$$

$$= 1.6 \times \frac{10.2 - 1.5}{9.2} \times 0.90 \times 1.02^{(15-20)}$$

$= 1.23 \text{ kg O}_2\text{/hph}$

(*g*) Power requirements

$$= \frac{\text{oxygen requirement}}{\text{amount of oxygen per hph}}$$

$$= \frac{27.7}{1.23}$$

$= 22.5$ hp

One 25-hp aeration unit is therefore required (1 hp = 0.7457 kW).

Air diffusion systems consisting of blowers and aeration tubing have been used in some places to provide the oxygen required. Weighted tubing placed on the bottom of a pond helps to keep the pond mixed. In cold climates, the submerged tubing is not as likely to be damaged with ice as are some types of surface aerator; moreover, heat may be conserved through the use of submerged tubing.

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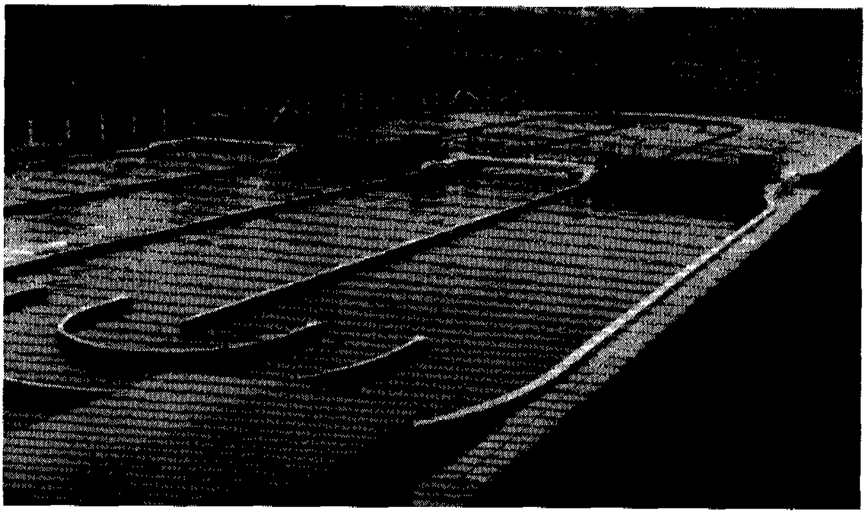
[1] If the effluent from the aerated pond requires further treatment, this may be provided by a facultative waste stabilization pond. It should be remembered that the remaining BOD will be the residual soluble BOD plus that BOD exerted by the biological cell material produced in the aerated lagoon. It is therefore necessary to use the total BOD (BOD$_u$) for the design of the facultative pond.

Another system employing prolonged aeration is the oxidation ditch, in which rotating brushes are used to agitate the surface of the liquid as it circulates. Although this system of aeration is usually used in activated sludge units, it can also be used for pretreatment in units followed by waste stabilization ponds. The oxidation ditch (Baars, 1962) provides full treatment of waste-water for small communities at the same cost *per caput* as treatment by the conventional activated sludge system for large communities. The mixed liquor in the ditch circulates at about 25–30 cm per second. Kessener brushes or rotors rotate at 75–125 rev/min and provide about 3 kg of oxygen per kilowatt. This system provides flexibility in that it permits intermittent fill-and-draw operations. Other uses for the oxidation ditch have been described by Pasveer (1962). A ditch used for a community of 7 000 in the Netherlands is shown in Fig. 18.

FIG. 18. OXIDATION DITCH FOR 7 000 INHABITANTS, SCHERPENZEEL, NETHERLANDS



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Summary and Design Recommendations

The designer of a waste stabilization pond system may find that many of the data he needs are not available. Consequently, engineering decisions must sometimes be made on the basis of experiences gained in other regions having comparable climatic conditions or on the basis of data obtained locally from experimental ponds. The type of waste-water, special environmental factors, local customs, and the specific treatment objectives should all be considered carefully by the engineer.

Experience gained through the operation of a local waste stabilization pond is of the utmost value. If properly operated for at least a year a single pond, even an experimental pond, can provide very useful design information.

The waste stabilization pond arrangements suitable for different types of waste may be summarized as follows:

Waste characteristics	*Suitable treatment processes* [a]
High solids content (i.e., night soil with minimum water added)	(*a*) A + F (*b*) A + F + M
Water-carried waste-water (domestic only)	(*a*) A + F + M (*b*) F (*c*) F + M
Water-carried waste-water (domestic and industrial)	(*a*) A + F + M (*b*) F (*c*) F + M (*d*) MA + F + M

Some procedures that have been found useful in the design of waste stabilization ponds under varying conditions are outlined below:

Case A. Water carriage waste system

1. Determine the influent flow by estimating the contributing population (N) and the *per caput* waste-water contribution (q). Flow = Nq litres/day. If data on quantities of domestic wastes are not available, Table 13 (p. 46) will provide a basis for initial estimates.

2. Estimate the concentration of BOD in the waste-water, and calculate the loading factor. In developing areas, a conservative estimate is 50 g BOD_5 per person per day, i.e., 1 kg BOD_5 = 20 population equivalents. BOD_5 may be assumed to be about 70 % of BOD_u. From these data it is possible to calculate the loading factor (L.F. = Nq BOD_u) and to locate a point on the abscissa of Fig. 13.

3. Evaluate temperature conditions; establish or estimate the mean temperature for the coldest month (needed for the design of facultative ponds); and establish or estimate the mean annual temperature (needed for estimating the breakdown rate and solids storage capacity of anaerobic ponds or the sludge storage volume of facultative ponds).

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[a] A = anaerobic waste stabilization pond
F = facultative waste stabilization pond
M = maturation pond
MA = mechanically aerated waste stabilization pond

4. Review the meteorological and hydrological characteristics of the local environment, i.e., estimate periods of protracted cloud cover, wind, rainfall, evaporation.

(*a*) The surface area of a facultative pond must include a safety factor if there are periods of extended cloud cover. Cloud cover for periods of a month or more may require a safety factor of as much as 25 %.

(*b*) Wind is important to the operation of a facultative pond. If there is no likelihood of wind, add 10–20 % to the surface area. Continuous strong wind will increase pond mixing and compensate for the decrease in photosynthetic oxygenation resulting from moderate cloud cover.

(*c*) Rainfall and evaporation may influence the choice of the number of ponds, depth, type of embankment, drainage, etc. Excessive evaporation in anaerobic and facultative ponds may expose deposited sludge and give rise to odours.

5. Survey the local terrain for soil characteristics and the location of ground-water and receiving streams or waterways.

6. Evaluate alternative combinations of ponds: (*a*) facultative only, (*b*) anaerobic plus facultative, and (*c*) anaerobic plus facultative plus maturation. The decision as to which arrangement to use will depend on the amount of water, the size of the plant, bacteriological requirements, and other factors. For communities with low water consumption (less than 50 litres *per caput* per day) and where evaporation rates are high, serious consideration should be given to using an anaerobic pond as a pretreatment unit, followed by a facultative pond, or to giving the facultative pond extra depth if it is to be used independently. The use of a maturation pond will depend upon the desired degree of biodegradation and coliform removal and on permissible evaporation.

Facultative ponds

(*a*) Estimate surface area, volume, and detention time (1) for the case where the total BOD_u enters the facultative pond and (2) for the case where 50 % of BOD_u is removed by preliminary treatment in an anaerobic pond and only the remaining 50 % enters the facultative pond.

(*b*) Calculate surface area and volume, using Fig. 13 or Equation 13.

(*c*) Use a design depth of 1 m in tropical climates. In temperate climates add 50 cm for sludge and grit storage; in cold climates add 1 m for variable liquid storage and for long-term sludge storage during cold periods when anaerobic decomposition virtually ceases.

Anaerobic ponds

Such ponds should be considered only if the sludge temperature is expected to be above 16°C.

(*a*) Estimate the digestion rate on the basis of Equation 17 and BOD_5 removal on the basis of data in Table 21. Adjust the estimated detention time according to the ratio between $K_{s(T)}$ and $K_{s(22)}$; as calculated by Equation 17, T is the mean annual temperature of the sludge layer.

(*b*) Determine volume and area. The depth should be 1.5–2 m (exclusive of grit storage).

(*c*) For large facilities or where the waste contains a large amount of grit, construct 2 ponds in parallel so that one may be cleaned periodically.

Maturation ponds

Design a maturation pond on the basis of 7 days' detention time and a depth of 1 m.

(7) Compare estimates of waste stabilization pond sizes, volumes, and depths with other suggested procedures. Suggest appropriate dimensions for the first pond system in a given locality.

(8) Prepare an operation and inspection schedule such as to provide data that can be used for improving the design of future waste-water treatment plants.

Case B. No local data available

Design a pilot facultative pond on the basis of Table 17, and observe operation for at least one year, or preferably longer. The pond should have a minimum depth of 1 m; it should be provided with inlet and outlet facilities as described in chapter 6, including measuring devices for influent and effluent flow. The data collection programme should include:

(1) BOD_5 measurements of influent and effluent (at least once a fortnight for a period of one year). Caution must be observed in measuring the BOD of water containing algae: the dark-bottle technique should be used to eliminate oxygenation due to photosynthesis by algae in the effluent. At least 5 of these BOD measurements should be made on composited 24-hour samples, i.e., samples taken every 4 hours over a period of 24 hours and weighted according to flow.

(2) Total residue of influent and effluent.

(3) Total volatile residue of influent and effluent.

(4) Settleable solids of influent and effluent.

(5) Influent and effluent flow measurements taken over a 24-hour period (each fortnight). A removable V-notch weir or a plastic (or fibreglass) Parshall flume is recommended.

(6) Fortnightly measurements of diurnal pH and dissolved oxygen. These measurements should be made over both a vertical profile (every 20 cm) and a horizontal profile (at 3 points).

(7) Depth measurements around the inlet, in order to assess the degree of solids build-up (once a year).

(8) Periodic observations and notes describing (*a*) odours; (*b*) colour of pond; (*c*) types of insect around the pond, particularly any increase in numbers of insects (e.g., flies, midges) that were not present prior to the construction of the pond; (*d*) water temperatures; (*e*) sludge temperatures; (*f*) suspended solids; (*g*) dissolved oxygen (of effluent); (*h*) changes in the apparent algal population and types of algae present; (*i*) presence and frequency of floating debris in the pond; (*j*) need for maintenance of embankments, etc.

Case C. Special case (night-soil)

Shaw (1962) suggests a loading of wastes from 2 740 persons per ha per day, with a depth of 60–150 cm if the climatic conditions are similar to those in Pretoria, South Africa. Concrete ramps or chutes should be provided to ensure that all residual night-soil can be washed or drained into the pond. Sludge banks should be dispersed. Anaerobic ponds should be filled with water before night-soil is added. Additional water may be required if evaporation and seepage rates exceed rainfall.

CHAPTER 5

Disease Transmission Control as a Factor in Pond Design

The purpose of waste-water treatment is normally not only to stabilize unsightly putrescible wastes but also to remove disease-causing agents. Designers of waste-water treatment plants must therefore consider the inclusion of special features that will reduce the possibility of contamination [1] of water supplies, agricultural produce, marine products, recreational areas, and the environment in general.

Many factors relevant to disease control affect design requirements. These include the recognition of potential pathogenic agents; the detection of specific pathogens; the life-cycle of pathogenic agents; the mode of transmission from one host to another; the survival of pathogenic organisms when subjected to various physical, chemical and biological treatment processes; die-away rates; and treatment plant operations as they affect disease transmission. The problem of disease transmission via waste-waters requires close attention; through the choice of appropriate design techniques, the engineer can do much to control the transmission of disease-causing agents.

Pathogenic Agents in Waste-waters

The principal human diseases that can be transmitted by water are listed below, together with their causative agents:

Etiological agent	*Disease*
Bacteria	
Vibrio cholerae	Cholera

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[1] The distinction between pollution and contamination is explained in the footnote on page 13.

Etiological agent	*Disease*
Escherichia coli (pathogenic serotypes)	Gastroenteritis
Salmonella typhi	Typhoid fever
Shigella dysenteriae, Sh. flexneri, Sh. bodyii, Sh. sonnei	Shigellosis (bacillary dysentery)
Protozoa	
Entamoeba histolytica [1]	Amoebic dysentery
Helminths	
Schistosoma haematobium *S. japonicum* *S. mansoni* blood flukes	Schistosomiasis (bilharziasis)
Fasciolopsis buski (intestinal flukes)	Fasciolopsiasis (non-dysenteric diarrhoea)
Viruses	
Unknown	Infectious hepatitis

In addition, there are a number of other diseases that may be transmitted by water under certain circumstances. These include bacterial diseases such as brucellosis, Salmonella food poisoning, and paratyphoid fever, virus diseases such as gastroenteritis due to various enteroviruses and poliomyelitis, a rickettsial disease (Q fever), and a number of helminthic diseases such as clonorchiasis (Chinese liver-fluke disease), parasitic or endemic hæmoptysis (lung-fluke disease), ascariasis, trichuriasis, and enterobiasis.

Indicator Organisms

Since the detection of individual pathogenic agents in either treated or untreated waste-waters is extremely time-consuming, if not impossible, substitute determinations are made to indicate the degree of contact that a sample of water has had with human wastes. However, none of the bacteriological, virological, or parasitological examinations of water at present available replace a thorough knowledge of the source and history of contamination (Mandahl-Barth, 1958; Pollitzer, 1959; Wagner & Lanoix, 1958; World Health Organization, 1960, 1963).

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[1] Other protozoa may exist concomitantly with *E. histolytica*.

At present the indicator micro-organism for which bacteriological analyses of water are most commonly performed is *Escherichia coli.* Found in the intestinal tract of man, this hardy micro-organism can be grown under laboratory conditions without too much difficulty.

When the analyses need to be simplified further, a more general count of coliform organisms can be made. The coliform group includes all aerobic and facultative anaerobic, Gram-negative, non-spore-forming, rod-shaped bacteria that ferment lactose, with formation of gas within a specified time and at a specified temperature (e.g., 48 hours at 35°C). Coliform organisms that normally inhabit the soil or warm-blooded animals will also act as general indicators of pollution. The sanitary condition of water can either be assessed in terms of the number of *E. coli* or, since all coliform organisms may be of faecal origin, be estimated in terms of the number of coliform organisms per sample volume. Newer techniques for the identification of *E. coli* and coliform organisms include the membrane filter method. To assist in this evaluation, bacteriological laboratories may also determine the total number of bacteria that will grow on an agar plate at body temperature (37°C). This plate count is expressed in terms of the number of bacteria per ml of the original sample.

The number of coliform organisms is estimated on the basis of positive determinations resulting from multiple-portion plantings using decimal dilutions or by the membrane filter method. It has been customary to express the results for the multiple tube method in terms of the "most probable number (MPN)" per 100 ml.

Permissible standards for drinking water, bathing water, irrigation water, etc., are frequently set on the basis of coliform counts. Obviously, the use of indicator organisms only reveals part of the picture. For example, if *E. coli* is detected there is a distinct possibility that some other intestinal organisms are also present. This type of indicator test is fairly valid for intestinal-type organisms such as *Salmonella typhi*, but it does not define the risk of finding other disease-causing entities, such as worms, other parasites, and viruses.

Some types of bacteriophage destroy bacteria occurring in human faeces, and may therefore be useful as indicators of human enterovirus survival (Kott & Gloyna, 1965; Vajdic, 1967).

Survival of indicator organisms

The removal of coliform bacteria from various biological treatment facilities has been reported in considerable detail. In general, removal rates are high, although it must be borne in mind, when accepting a coliform reduction value in percentage terms, that the sanitary condition of water is dependent on more than indicator organisms. With these limitations in mind, some case studies are presented below to provide a basis for the evaluation of plant performance.

The retention of sludges, detention of liquids, availability of nutrients, and numerous environmental factors are important for the survival of enteric micro-organisms. To obtain the most effective removal, therefore, facilities should be so designed as to minimize short-circuiting. Moreover, adequate contact time must be provided so that the pathogenic organisms are destroyed.

Table 22 shows the results obtained under controlled conditions in 5 small waste stabilization ponds, 0.4 ha in area and with operating depths ranging from 80 cm to 1.5 m. The raw waste-water was applied at organic loading rates of 22, 45, 67, 90, and 112 kg BOD_5/ha per day, and the flow rates varied from 600 to 1 050 m^3 per day. The temperature ranged from 2°C to 33°C. The MPN reached the highest levels in the ponds receiving the heavier loadings and operating with the shorter detention times. Coliform removal rates varied only in the third and fourth significant figures. No pond failed to remove less than 99.99 % of coliform bacteria during any month in which measurements were taken (Neel et al., 1961).

TABLE 22

SURVIVAL OF COLIFORM BACTERIA IN FIVE SMALL WASTE STABILIZATION PONDS, FAYETTE, MO., USA *

Pond	Loading (kg BOD_5/ha per day)	Coliform bacteria surviving in pond effluents (MPN[a])			Percentage of samples with MPN[a] equal to or greater than	
		Lowest value	Arithmetic mean	Highest value [b]	43 000	93 000
1	22	430	14 700	43 000	16	0
2	45	3 600	24 700	93 000	29.5	7.9
3	67	430	24 300	93 000	24.5	8.9
4	90	2 400	34 000	93 000	48	18
5	112	2 400	40 200	93 000	47	19.2

* Data from Neel et al. (1961), p. 631.

[a] Most probable number per 100 ml.

[b] Exclusive of short-circuiting (inadequate mixing).

Field studies have shown that large-scale maturation ponds (tertiary ponds) are particularly effective in reducing faecal *E. coli* (Drews, 1966). The success of maturation ponds as a buffer against environmental bacterial pollution has been clearly stated by Stander & Meiring (1962, pp. 6–7):

> Although a faecal *E. coli* count of nil per 100 ml cannot always be obtained in maturation ponds, *the degree of safety* (as indicated by faecal *E. coli* count) that *can* be obtained is comparable with that attainable in practice, where sand filtered effluent is chlorinated.

The London Metropolitan Water Board reports that tertiary pond treatment of effluents from conventional secondary sewage treatment plants reduced *E. coli* by 99.5 % (Windle Taylor, 1965). A detention time of 8 days is recommended. It is significant that certain pathogenic micro-organisms (including some of the Salmonella group, which cause typhoid fever, paratyphoid fever, and gastroenteritis) were regularly isolated from the secondary biological treatment plant effluent but were virtually eliminated from waste-water that had passed through the ponds. Likewise, enteroviruses were present in only two of the samples discharged from the third pond. Similar experiences have been recorded in Australia (Parker, 1962), where detention times of 30–40 days in multiple ponds were shown to reduce the number of coliform organisms to levels approaching those found in drinking-water. Of course, other factors besides coliform density must be evaluated before treated waste-waters and drinking waters can be compared. Studies in Auckland, New Zealand, showed that over 99 % of the coliform organisms were removed in waste stabilization ponds (New Zealand, Auckland Metropolitan Drainage Board, 1963).

In Israel, coliform removal rates of about 99 % have been reported in 2 ponds connected in series having a total detention time of 26 days (Wachs et al., 1961). The average BOD_5 reduction was 87.4 %, from an initial BOD of 283 mg/litre. Unsatisfactory coliform reduction was obtained with a detention time of only 3½ days (Hershkovitz & Feinmesser, 1962). At present detention in any waste stabilization pond for 5 days under aerobic (or facultative) conditions is accepted in Israel as the minimum treatment sufficient for waste-waters to be used for the irrigation of "restricted" crops.[1] After 20 days' detention in a facultative-aerobic pond system, the effluent may, after chlorination, be used for any crops and also for selected industrial purposes (Meron et al., 1965). Ponds designed to provide effluent that can be used for the irrigation of "restricted" crops generally have a 5-day detention period, with loadings between 196 and 246 kg BOD_5/ha per day. Under the climatic conditions prevailing in Israel, these ponds are capable of reducing the BOD_5 by 70–90 % and the coliform count by 90–99 % (Shuval, 1963). The health of the farm-workers who distribute these treated wastes does not seem to be impaired.

Fig. 19 shows the results obtained from two different pond systems. The Herzliya ponds consist of double cells, provide total detention of 26 days, and are operated as anaerobic-facultative systems. The Yagur units consist of an anaerobic pond followed by an aerobic-facultative pond with detention periods of 18 hours and 5 days respectively (Kott, 1961). The number of coliform bacteria in the influent averaged about 5.4×10^8 per 100 ml for the Herzliya ponds and about 3.5×10^8 per 100 ml for the Yagur ponds. The antagonistic environment of the anaerobic pond and the effect of the

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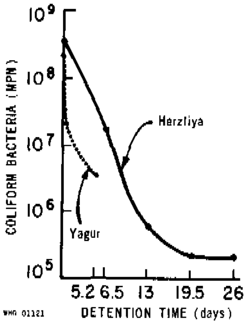
[1] The following list of restricted crops was published by the Israel Ministry of Health in 1954: "1. Industrial crops unfit for human food, such as sugar beets, cotton and fibres; 2. Pasture, provided the animals shall not be allowed on the field until the grass is entirely dry; 3. Grass grown for "dry" hay; 4. Vegetables which are consumed only after cooking, i.e., eggplant, potatoes, sweet potatoes, maize, and dry onions; 5. The following fruit trees: citrus, bananas, nut trees, date palms, avocados, and all sapling nursery stock; 6. Ornamental shrubs, plants, and flowers; 7. Plants grown for seed production only; 8. Sunflowers and carobs (St. John's bread), provided irrigation is in furrows only; and 9. Apple, pear, and plum trees, provided irrigation is stopped at least one month before harvesting."

longer detention time provided by the secondary pond are pronounced (Fig. 19). Salmonellae were found once in the influent of the Herzliya treatment plant, but never in the effluent.

The key to consistently high reduction of coliform organisms is series-type pond construction, which provides adequate liquid detention and solids retention. The primary advantage of series-type ponds is the prevention of inadequate mixing. The effects of such ponds have been described by Van Eck (1961) and Bolitho (1965).

FIG. 19. REDUCTION OF COLIFORM BACTERIA AT TWO TREATMENT PLANTS, ISRAEL



Reproduced, by permission, from Kott (1961), p. 7.

Other Bacteria

Man is the reservoir of most bacterial diseases that destroy or incapacitate him. The better-known human diseases caused by faecal contamination include typhoid fever, paratyphoid fever, cholera, and amoebic and bacillary dysentery. Fortunately most pathogenic bacteria are unable to reproduce outside the body, and under most circumstances they slowly die off when exposed to the antagonistic environment of a biological treatment plant, river, or other body of water. There is some indication, however, that the greater the pollution the less rapid is the die-off of pathogenic bacteria (Hynes, 1960).

It has been estimated that about 3 % of patients with typhoid fever excrete *Salmonella typhi* indefinitely, whereas convalescent food poisoning patients may excrete salmonellae for periods from a few weeks to a few months.

The literature indicates that the long detention times used in waste stabilization ponds result in a considerable reduction in *S. typhi*. In Java, Gillespie (1944) reported that raw sewage contained 1–41 typhoid organisms per ml, the settled sewage 1–6 per ml, and the pond effluent none.

Laboratory tests have indicated that *S. typhi* survival is dependent on the supply of nutrient (McGarry & Bouthillier, 1966). The greater surface loadings of ponds with short detention periods seem to aid the survival of typhoid bacilli, whereas facultative ponds, because of their longer detention periods and the lower concentration of nutrients, provide a more antagonistic environment. For example, the survival period under organic loadings of 4480 kg BOD_5/ha per day was found to be 11 days, as compared with 30 hours for loadings of 56 kg BOD_5/ha per day.

It has been shown in field studies that waste stabilization ponds operating in series can provide 99.5 % reduction of *S. typhi* (Coetzee & Fourie, 1965). The detention periods for the first and second pond respectively were 20 and 15 days. The average reduction of *S. typhi* and other organisms is compared in Table 23. There was no increase in the bacteria studied in these 2 ponds at any time, and the total reduction of *E. coli* was similar to that of other bacteria. However, *S. typhi* was found to be more resistant than *E. coli*.

TABLE 23

AVERAGE BACTERIAL REDUCTION IN STABILIZATION PONDS*

Bacteria	Organisms in crude sewage	Pond A		Pond B		Total reduction (%)
		Organisms in effluent	Reduction (%)	Organisms in effluent	Reduction (%)	
Escherichia coli (per ml, average of 25 determinations)	257 000	6 400	97.5	50	99.2	99.98
Salmonella typhi (per 100 ml, average of 20 determinations)	2.4	0.20	91.7	0.01	95	99.6
Pseudomonas aeruginosa (per 100 ml, average of 16 determinations)	26 045	307	98.82	62	79.7	99.69
Clostridium perfringens (per 100 ml, average of 16 determinations)	57 791	1 162	97.98	53	95.4	99.91

* Reproduced, in modified form, from Coetzee & Fourie (*1965*), p. *211*, by permission of the publishers.

In a comparative study, 4 maturation ponds were investigated. The detention time provided in each of these ponds was about 2½ days, providing a total detention time of 10 days. As shown in Table 24 the reduction of *S. typhi* was less than that of *E. coli* and *Pseudomonas aeruginosa*. The efficiency of bacterial removal in the maturation ponds was not as great as in facultative waste stabilization ponds. It would seem that the differences in detention time were partly responsible for the lower reduction rates. As with the facultative ponds, no increases in bacterial counts were observed in any of the maturation ponds. However, these results are in contrast to the phenomenon of "after-growth" noted in laboratory and stream studies, whereby the density of coliform organisms sometimes increases downstream from the point of discharge. Such after-growth is frequently observed below discharges that have been partially disinfected with chlorine. It may be that the amount of nutrients available in maturation ponds is so low as to preclude after-growth. The effects of temperature on bacterial survival must also be considered.

In carefully conducted laboratory studies involving series-connected waste stabilization ponds (Suwannakarn & Gloyna, 1964), the die-away rate for *E. coli* was greater than that for coliform organisms, which in turn was

TABLE 24

AVERAGE BACTERIAL REDUCTIONS IN MATURATION PONDS *

Bacteria	Pond A			Pond B		Pond C		Pond D		Total reduction (%)
	Organisms in influent	Organisms in effluent	Reduction (%)	Organisms in effluent	Reduction (%)	Organisms in effluent	Reduction (%)	Organisms in effluent	Reduction (%)	
Escherichia coli (per ml, average of 16 determinations)	8 220	1 561	81.0	754	51.7	403	46.6	107	73.4	98.7
Salmonella typhi (per 100 ml, average of 11 determinations)	5.8	4.1	29.3	2.8	31.7	1.1	60.7	0.8	27.2	86.2
Pseudomonas aeruginosa (per 100 ml, average of 10 determinations)	1 147	111	90.4	69	73.8	10	90	1	90	99.91

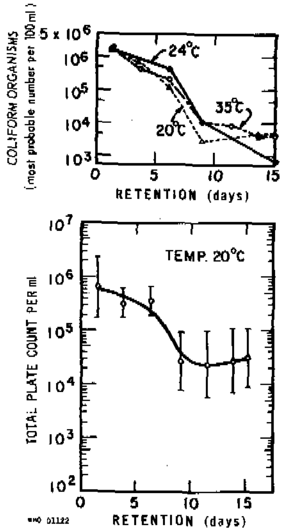
* Reproduced, in modified form, from Coetzee & Fourie (1965), p. 212, by permission of the publishers.

greater than that for the total bacterial population (Fig. 20). In a field study, coliform reduction rates varied from 85.9 % in the winter to 94.4 % in the autumn, while the reduction rates for faecal coliform organisms and faecal streptococci were 87.9 % and 97.0 % respectively (Geldreich et al., 1964).

In laboratory studies involving *Salmonella abortus equi* the die-away rate was found to be dependent on temperature, initial organism density, and availability of nutrients. The reduction was primarily due to competition for food (Hok, 1964). In a field experiment with the same organism, under summer temperature conditions (25°C), the number of organisms per ml was below the sensitivity of analyses within 72 hours.

Field studies in Nagpur, India, showed that species of salmonella were present in the influent, but not in the effluent, of facultative ponds (Lakshminarayana et al., 1964). At the same time, however, coliform organisms, *E. coli*, and streptococci were reduced by only 89–92 %, 81–91 % and 84–85 % respectively. Under the tropical conditions prevailing, it was possible to achieve 72 % reduction of BOD_5 with a loading of 450–560 kg/ha per day. The pond depth was 1.2 m, and the detention time only 2 days per pond. Aeration was applied below the pond surface for 1 hour in the morning and 2 hours in the evening.

FIG. 20. REDUCTION OF BACTERIA IN EXPERIMENTAL WASTE STABILIZATION PONDS



Data from Suwannakarn & Gloyna (1964), pp. 134 and 135.

Similarly, in a study involving 2 ponds linked in series, there were no appreciable differences in reduction rates between total bacteria, coliform organisms and enterococci (Parhad & Rao, 1964). Influent samples indicated the presence of *S. typhi*, but samples from both ponds gave negative results.

Studies conducted at the King Institute, Madras, India, indicate that the die-off rate of *Mycobacterium tuberculosis* is high. Domestic wastes mixed with sanatorium wastes were treated in a septic tank and then in a waste stabilization pond. Acid-fast bacteria were found in about 50 % of the samples of untreated waste and in 25 % of the samples of septic tank effluent, but not in samples taken from the pond and the pond effluent (T. Viraraghavan—personal communication, 1966).

Mortality Rate Estimates

Because of the variety of disease-causing agents and the multiplicity of environmental factors involved, it is not likely that a simple and universally usable procedure can be developed for predicting the number of specific pathogenic agents in a pond or effluent at any given time. However, there are a number of pointers that will assist in estimating the bacteriological quality of the treatment plant effluent. Some of the environmental factors that cause a decrease in bacterial concentration are listed below:

1. *Dilution and mixing.* Dilution is a significant factor in dispersing organisms and thereby reducing the probability that a human host will come into contact with a pathogen. Where mixing is assumed in a design, inadequate mixing must not be permitted.

2. *Aggregation.* In an aqueous environment, whether fresh or salt, bacteria have a tendency to form clusters and become attached to particular matter, which then tends to settle out. In waste treatment plants adsorption and sedimentation are favoured by the introduction of both organic and inorganic solids.

3. *Presence of toxic substances.* The presence of some industrial wastes, and dilution of the effluent in salt water, can rapidly reduce the coliform bacteria concentration. Some chemicals, such as silver and copper in concentrations of 0.0003 mg/kg and 0.001–0.01 mg/kg respectively, may have a toxic effect. The osmotic action of large concentrations of dissolved salts has an adverse effect on the survival of pathogenic organisms in sewage. Similarly, biological breakdown products, antibiotics, and metabolic waste products may be toxic to some micro-organisms.

4. *Predation.* Sewage bacteria in a waste treatment plant are part of the biological food chain, and large numbers of these organisms are consumed by protozoa and higher forms of animal life. Specific bacteriophage may also be concerned in destroying some of the faecal organisms.

5. *Sunlight.* Where detention periods are long and ponds are shallow, sunlight and surface aeration probably contribute to the reduction of bacteria. However, in all probability the ultraviolet rays in sunlight do not have an appreciable effect on bacterial survival.

6. *Temperature.* Bacteria in sewage seem to survive longer at cooler temperatures, at least until the temperature drops below 12°C. Predators multiply faster at higher temperatures and thereby reduce the number of bacteria more quickly.

7. *Availability of nutrients.* Many faecal bacteria require organic sources of carbon and nitrogen. There is some evidence that bacteria in sewage require at least 10–100 mg/litre of nutrient material if no suspended solids are available (ZoBell, 1946).

Chick's Law has sometimes been used to describe the rates of bacterial disappearance when removal is uniform (Equation 25).

$$\frac{N'_t}{N'_o} = 10^{-k'R} = e^{-k'_1R} \tag{25}$$

where

N'_t = bacterial population at detention time R (days)
N'_o = initial bacterial population
k' = rate constant, $\log_{10}$ (bacterial disappearance/day)
k'_1 = rate constant, $\log_e$ (bacterial disappearance/day)
R = time of exposure, i.e., detention time (days)

There must be no qualitative or quantitative changes in the breakdown scheme. The value of k' must not change as a result of an increase in the number of micro-organisms as the waste-water is transferred from one pond to another. However, a series of ponds operating under uniform conditions can develop different bacterial populations. In one study it was found that when k' was equal to 0.1 and R was 10 days, the removal of coliform organisms was 90 % (Meron et al., 1965). It should be noted that when R changes, k' also changes.

Since biological purification involves a variety of interacting factors, Equation 25 is not likely to be truly representative. Consequently, a general purification rate equation, applicable to stream purification, might be considered (Fair & Geyer, 1954). Furthermore, if the bacterial population is composed of both a resistant fraction and a less resistant fraction, it will be necessary to modify Equation 25 to Equation 26 (Phelps, 1944). This equation applies only to flows through ponds and rivers that have not been mixed.

$$\frac{N'_t}{N'_o} = p\,e^{-k'_1R} + (1 - p)\,e^{-k'_mR} \tag{26}$$

where

p = fraction: for less resistant fraction, $0 < p \leqslant 1$, with a die-away constant k'_1; for more resistant fraction, $1 > (1 - p) \geqslant 0$, with a die-away constant k'_m that is less than k'_1.

Equation 26 can be rewritten in terms of a modal number of bacteria and an all-inclusive non-uniformity coefficient (Equation 27):

$$\frac{(N'_o - N'_R)}{N'_o} = \frac{N'_t}{N'_o} = (1 + ck'_1R)^{-\frac{1}{c}} \tag{27}$$

where

$N'_R = N'_o - N'_t$
c = non-uniformity coefficient

When removal occurs at a uniform rate, $c = 0$ and Equation 27 reverts to Equation 25, but again it applies only to flow through ponds or rivers that have not been mixed.

Experimental results seem to indicate that Equation 27 can be modified still further to describe the concentration of faecal bacteria in a pond and a series of ponds (Marais & Shaw, 1961). Equation 28 can be written if the following assumptions are made:

(1) Mixing is complete and instantaneous.

(2) Breakdown occurs by a first-order reaction with the constant k.

(3) No reactions, except k, are temperature-dependent.

(4) Influent flow is equal to the effluent flow.

(5) Equilibrium conditions are established in each pond.

(6) BOD, organisms, flow, etc. can be expressed in terms of their daily mean value.

Under steady-state conditions, therefore, the effluent concentration from a single pond might be estimated as shown in Equation 28:

$$N'_t = \frac{N'_o}{K'R+1} \tag{28}$$

For succeeding ponds, new values of N'_o and K' are used to determine the remaining bacterial density.

Finally, the percentage reduction of faecal bacteria in ponds and rivers may be estimated according to Equations 29 and 30 respectively. These equations assume that K' has a value of 2 for reduction of *E. coli.*

$$\frac{N'_t}{N'_o}(\%) = \frac{100}{(2R_1+1)(2R_2+1)(---)} \tag{29}$$

$$\frac{N'_t}{N'_o}(\%) = 100\,e^{-2R} = \frac{100}{(2R+1)^{N_p}} \tag{30}$$

where

$\frac{N'_R}{N'_o}(\%)$ = bacterial removal (%)

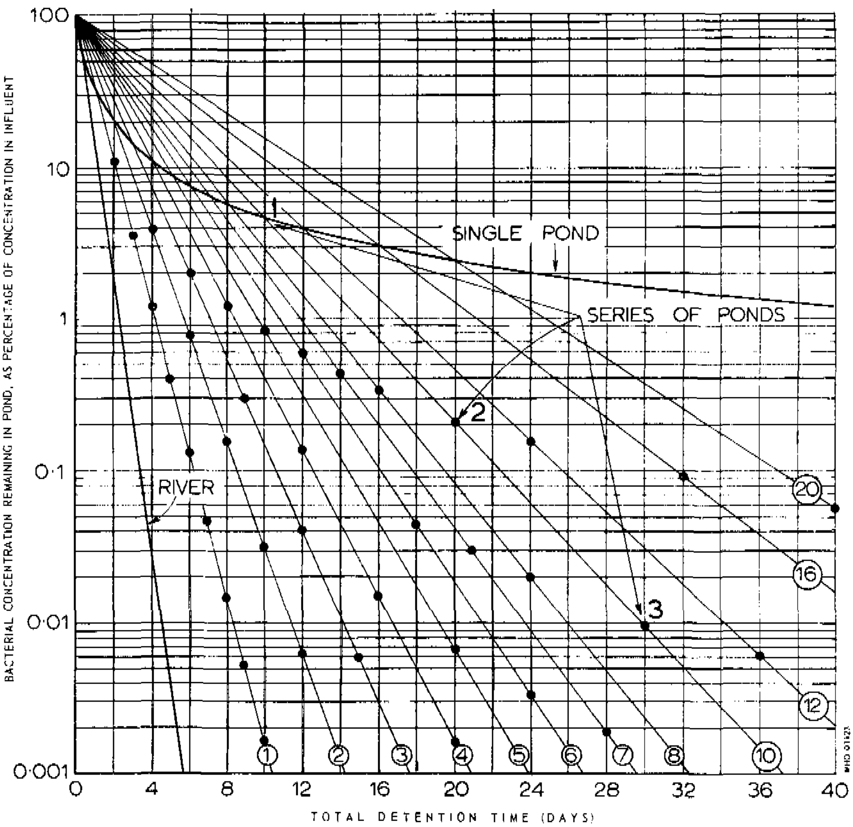
R = detention time per pond or in the river (days)

R_1, R_2, etc. = detention time in pond 1, 2, etc. (days)

N_p = number of ponds

The estimated percentage reduction of influent bacteria for various detention times is plotted in Fig. 21, which assumes a bacterial removal rate of $K' = 2$. When $K' = 2$, bacterial reduction in ponds in South Africa and Zambia shows a fair correlation with the theoretical values obtained by application of the above equations (Marais, 1966). The faecal bacteria in these ponds include *E. coli*, *Streptococcus faecalis*, and other coliform organisms.

FIG. 21. THEORETICAL CONCENTRATIONS OF FAECAL BACTERIA IN SINGLE POND, SERIES OF PONDS, AND RIVER, WHERE $K' = 2$*



After Marais (1966), p. 740.

* The slanting lines interrupted by black dots represent series of ponds. Each dot represents one pond, and the figures in circles show the detention time in each pond in the series. The ordinate indicates the bacterial concentration remaining at each stage. For example, the line with the circle containing the figure 10 represents 3 ponds connected in series, each having a detention time of 10 days. The bacterial concentration in the effluent of pond 2 (i.e., after 20 days' detention) is 0.2 % of the concentration in the influent of pond 1.

In another study it was shown that the degradation constant, K', is only 0.8 for *S. typhi* as compared with 2.0 for *E. coli* (Coetzee & Fourie, 1965; Marais, 1966). These differences are emphasized as a warning against generalizing too much when extrapolating from results obtained with indicator organisms to specific disease-causing agents. However, projections made on the basis of the equations and curves shown above can provide a useful indication of the degree of general bacterial pollution control that is required. Furthermore, base-line estimates can be useful at times for comparing winter and summer conditions, inadequate mixing within pond systems, and regional operations.

Viruses

Data concerning the survival of viruses in waste stabilization ponds are limited. These ponds, with their long detention times, reduce viral concentrations, but algae do not seem to be responsible for this. It is known that some human enteric viruses survive conventional waste-water treatment processes, including chlorination (Shuval et al., 1967). These agents include strains of poliovirus, coxsackievirus, echovirus, reovirus and infectious hepatitis virus (Berg, 1967).

The survival of an attenuated type 2 poliovirus and a reovirus was studied in a system of model waste stabilization ponds (South Africa, National Institute for Water Research, 1964). For this study, 4 series-connected ponds were used, with detention periods of 20, 10,5 and 5 days. Although the total detention time was 40 days, it was significant that the concentration of virus in the effluent reached its peak on the 24th day after inoculation. The virus appeared in the effluent of the first pond after 2 days. Environmental conditions remained uniform, there was no rain during the test period, and the rate of inflow was constant. The reovirus, which was already present in the waste-water, appeared to survive longer than the poliovirus.

In South Africa some work has also been done on the development of an indicator agent for certain viruses. *Serratia marcescens* phage was chosen for this work because it is not normally found in waste-water in significant concentrations and because it is cultivated easily, readily identified, and harmless to man. For comparative purposes, type 1 poliovirus was used in all tests except those involving waste stabilization pond water, for which the Mahoney strain of type 1 poliovirus was used (South Africa, National Institute for Water Research, 1965). Poliovirus appeared to be more resistant at 2°C, equally resistant at 20°C, and less resistant at 37°C and 56°C by comparison with *S. marcescens* phage. Even 20 cycles of freezing and thawing had no effect on either of the viruses, but direct exposure to sunlight affected both. Inactivation due to light was more efficient in the shallow layers of fluid. The phage was more affected by pH (up to a value of 6.4) and by detergents than by poliovirus. Algae seemed to play no significant role in the removal of either virus (South Africa, National Institute for Water Research, 1965).

Special attention is drawn to the unique water reclamation project of Santee, California (USA). This community deliberately planned to utilize its treated waste-water through impoundment in recreational lakes. A series of comprehensive studies of the project have been made (Askew, 1965; Merrell et al., 1965), and available reports discuss eutrophication, vector control, epidemiology, social acceptance and the ecology of the entire recreational area. An assessment has been made of the fate of viruses, as well as of total and faecal coliform organisms and of faecal streptococci, as the waste-water passes through the conventional secondary treatment process, the tertiary processes, and the recreational lakes.

The domestic waste treatment plant and water reclamation process consists of a primary clarifier, an activated sludge unit, a digester, a waste stabilization pond with an area of 6.5 ha (capacity 11 000 m^3), a chlorination unit, a spreading area of 1.25 ha, and 4 recreational lakes. The waste-water percolates into the gravel beds and emerges about 60 m below the spreading area.

No positive identifications of organisms of public health significance have been made in the first two lakes. Virological quality studies were conducted on a weekly basis at 7 locations from July 1962 to April 1963. Detailed analysis of 3 bacterial indicators was also made. It seems that any bacterial activity of public health significance in the lakes is more likely to be due to recontamination than to regrowth from organisms present in sewage.

Helminths

The control of schistosomiasis is particularly important. Because the helminth must pass through the egg and larval stage before it can be passed to a second host, attention must be concentrated on the survival of the egg, larva, and adult in various waste-water treatment processes. Infection with *Schistosoma mansoni*, *S. haematobium*, and *S. japonicum* is widespread, and it has been estimated that as many as 200 million people are infected with these organisms (Forsyth & Bradley, 1966). An important source of infection is water contaminated with larval forms (cercariae) derived from snails.

Laboratory tests have indicated that aerobic and facultative waste stabilization ponds are not likely to suppress the hatching of the eggs or to affect materially the survival of miracidia of *S. mansoni* or the vector snails; anaerobic ponds, however, will curtail the hatching and survival of the eggs (Kawata & Krusé, 1966); snail vectors transplanted into full-scale ponds all died within 10 weeks (Hodgson, 1964), but not apparently because of a shortage of food.

Larvae of *Ancylostoma duodenale*, ova of *Schistosoma mansoni* and *Enterobius vermicularis*, and cysts of *Giardia lamblia* were isolated from the untreated sewage of a small urban community in Southern Rhodesia, but no helminths were found in the effluents of the stabilization pond (Hodgson, 1964). Observations in South Africa have confirmed that no helminths, cysts, or ova are present in the effluent from a series-connected pond system (Meiring et al., 1968). Their removal is most likely due, at least in part, to sedimentation. The specific gravity of ova and cysts is about 1.1 (Liebmann, 1965).

Another important point is that the eggs of *Ascaris lumbricoides* and *Trichuris trichiura* may survive in places such as septic privies for as long as

2–3 months at temperatures of 20–22°C, and longer still at cooler temperatures (Snell, 1943). Retention requirements in septic tanks of 5–11 months have been reported elsewhere,[1] but the retention period may be reduced by redesigning the septic privy.[2] The use of unheated digesters to treat night-soil and the further treatment of the supernatant with a trickling filter destroyed only about 80 % of the viable *Ascaris* eggs.[3]

Another important helminth, *Wuchereria bancrofti*, is transmitted in nature by many species of mosquitos. The most important vectors are *Culex fatigans*, *C. pipiens*, *Aedes polynesiensis*, and several species of *Anopheles*. For the control of filariasis caused by *W. bancrofti*, it may be desirable to use some top-water minnow such as *Gambusia affinis* in the maturation pond. A minnow or similar fish might also survive in some less heavily loaded facultative ponds. Any small fish that feeds upon the mosquito larvae and pupae and can survive in the ponds would be satisfactory.

Protozoa

Studies in Israel have revealed cysts of *Entamoeba histolytica* in the pond influent but never in the effluent (Wachs et al., 1961). This is an area of study that should receive much more attention.

Summary

The design of treatment facilities based on the control of a specific pathogen has not yet reached a high degree of development. Much work and effort is still needed to solve the problems of pathogen transport through waste-water treatment plants, but more engineers and scientists are beginning to recognize the problem. The removal of BOD without regard to the destruction of disease-causing agents is inadequate. At least it is now recognized that pathogenic matter must be destroyed in the waste-water treatment plant or diluted to a level in the environment where the chances of disease being caused in a given population are negligible.

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[1] Sprent, J.F.A. (1963) *Environmental control*, Geneva, World Health Organization (unpublished document).

[2] Teodorović, B. (1968) *A modified septic (LRS) privy*, Geneva, World Health Organization (mimeographed document WHO/WD/68.3).

[3] Horasawa, I. (1963) *Night-soil disposal, with reference to parasitic control*, Geneva, World Health Organization (mimeographed document WHO/Helminth/64).

CHAPTER 6

Facilities Design

Essentially, facilities design is concerned with the location and layout of the plant, the interconnexion of the waste-water treatment system, the physical structures required to contain the process, complete construction drawings, and detailed specifications. The specifications will vary with the size of the facility, but in principle they must provide a precise description of what is shown in the construction drawings; of any special requirements provided for in the design; of the workmanship required; of the types of materials and equipment needed; of the manner in which construction operations are to be performed; of the nature and details of special testing procedures; of the method for measuring work accomplished; of the method of payment; of any special guarantees given to the owner in the event of defects developing; of procedures to be followed by the engineer and builder for clarifying drawings, equipment purchases, etc.; and of the process of final acceptance.

The reader should note, however, that while the information and recommendations contained in this chapter are useful for the general design and perhaps for the typical case, they must not be used without the advice of an engineer who is familiar with local conditions and has a good knowledge of waste treatment processes, waste characteristics, availability of local materials, special socio-economic requirements, and overall development plans for the region. The chapter is intended as a guide to engineers who may be called upon to design a waste stabilization pond for the first time, or to health authorities who wish to compare alternative designs in the light of ideas that have been considered or used at other places.

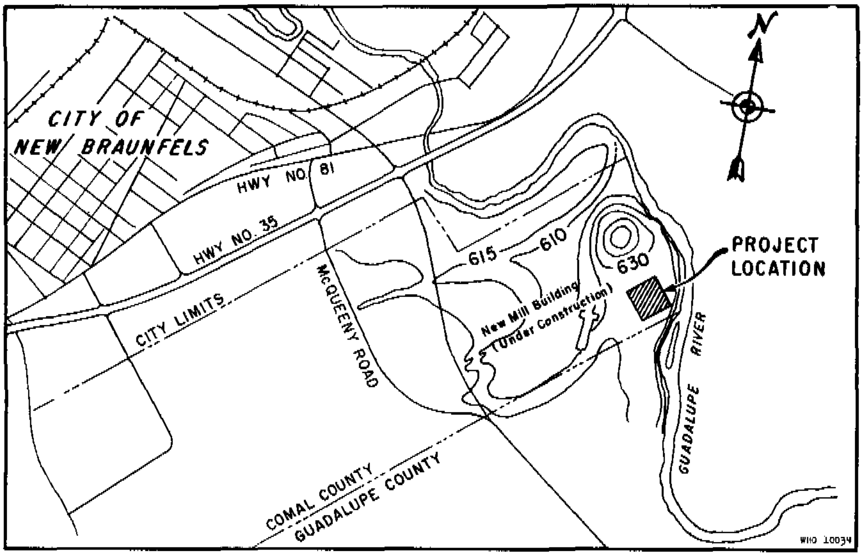
In addition to a discussion of the facilities and the selection of alternatives, examples of various types of waste stabilization pond are included. Sometimes an entire facility is shown without reference to overall merits, but merely to show how various units may be connected.

Plant Location and Layout

The location of a treatment facility may be fixed by some previous decision involving the location of an outfall sewer, by the availability of dilution water, or by some other local requirement. However, some important points involving location and layout must be considered in connexion with any proposed waste-water treatment facility. The overall economics, design, and operation are greatly affected by the nature of the site. These site-selection factors apply whether the development envisaged concerns a new sewerage system,[1] a modification of an existing waste-water treatment plant, or a new treatment plant that must be connected to an existing sewer system.

Site selection is particularly important when waste stabilization ponds are contemplated. The terrain must be such that the pond or ponds will meet the shape and size requirements of the process design. A topographic map of the site, showing contours at about 30-cm intervals, is desirable (Fig. 22). Nearby utilities, industries, houses, special structures, and topographical features should be shown on the map.

FIG. 22. CONTOUR MAP SHOWING SITE LOCATION



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Consideration must be given to nearby residential areas. The requirements concerning distance from residences are naturally dependent on local customs, climate, and the type of treatment process planned. The permissible distance from the pond to a residence is determined largely by the loading applied to the pond, the adequacy of design and the standard

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[1] A sewerage system is a system of sewers and appurtenances for the collection, transportation, pumping, and treatment of sewage and industrial wastes.

of maintenance provided. In some cases residences have been located within 100 m of well-designed and well-operated facultative ponds. Greater distances are needed for anaerobic systems. In India, it is considered good practice to locate facultative ponds at least 150 m from small residential colonies and 450 m from large residential areas (Dave & Jain, 1967). A correspondent in Israel has suggested a minimum distance of 1–2 km (H. I. Shuval—personal communication, 1966). In the USA a minimum distance of 0.5–1 km between a pond and a housing development is customary.

There must be an access road or path to the treatment plant. Where motor vehicles are customarily used, this access road should be surfaced to facilitate all-weather travel.

Special attention must be paid to drainage requirements. The pond system must be protected from general flooding, for ponds, inlet and outlet devices, and other features can be damaged or destroyed by floodwaters and accompanying debris. Floodwaters containing large amounts of sediments may, through deposition and erosion, completely destroy an inadequately protected waste stabilization pond. Of equal importance is the drainage of normal stormwater runoff. If rainwater cannot be drained, dikes may deteriorate, insects may breed in the puddles, excessive and unwanted vegetation may grow, and maintenance requirements will increase sharply.

The site must be large enough for any anticipated expansion. Since the size of waste stabilization ponds is fixed on the basis of present flow and load, it is very important that space be readily available for the addition of similar or different units as the need arises.

Provision must be made in the design for the inclusion of certain miscellaneous items, such as a service building, that may be called for under certain circumstances.

Pretreatment Units

Under some conditions, it may be desirable to provide grit chambers, bar racks (bar screens), and cutting devices. Grit removal is required where sanitary and storm waste-waters are collected in the same sewer and where grit is added intentionally to the sanitary sewer. Bar racks are necessary if pumping is required to lift the waste-water from the outfall sewer into the ponds, or if cutting devices or mechanically-cleaned grit chambers are used. Cutting devices may be used in larger facilities to reduce the size of the organic solids.

Grit is the name given to the heavy mineral material in waste-water, such as sand, silt, gravel and cinders. Since grit is roughly 2½ times as heavy as organic solids, it can be effectively removed if the velocity of the waste-water is adjusted so that the grit, but not the suspended organic matter, will settle.

The amount of grit present depends on many factors, including the type of sewer system, industrial contributions, the condition of the sewers, and local cleansing customs.

In some countries sand and silt are often used as aids in cleaning domestic utensils, and this creates high grit concentrations in the waste flow. Grit loads up to 0.17 m^3 per 1 000 m^3 of waste-water have been reported in South Africa (Meiring et al., 1968), with an average of about 0.05 m^3 per 1 000 m^3. If this grit is not removed before discharge to waste stabilization ponds, banks will soon build up around the discharge point and will eventually reach the pond surface. This hampers the mixing of the influent with the pond contents and may lead to unsightly conditions and unpleasant odours. Problems of sand deposition have also been reported in Peru and the USA. In the USA the amount of grit averages about 0.01–0.1 m^3 per 1 000 m^3 of waste-water, but local sewer problems may lead to much greater quantities. When such wastes are treated in ponds, grit removal is essential.

Power-operated grit removal facilities are expensive to install and maintain, and are usually beyond the technological and financial resources of small communities in developing countries. Unskilled labour is normally plentiful and inexpensive in such areas, however, and a grit channel cleaned by hand is simple but effective. The waste flow through such a channel should be kept at a constant velocity of about 0.3 m per second, and the length of the channel needs to be sufficient to retain the liquid for about one minute, i.e., 15–18 m. These conditions permit the grit, but not the organic material, to settle on the floor of the channel.

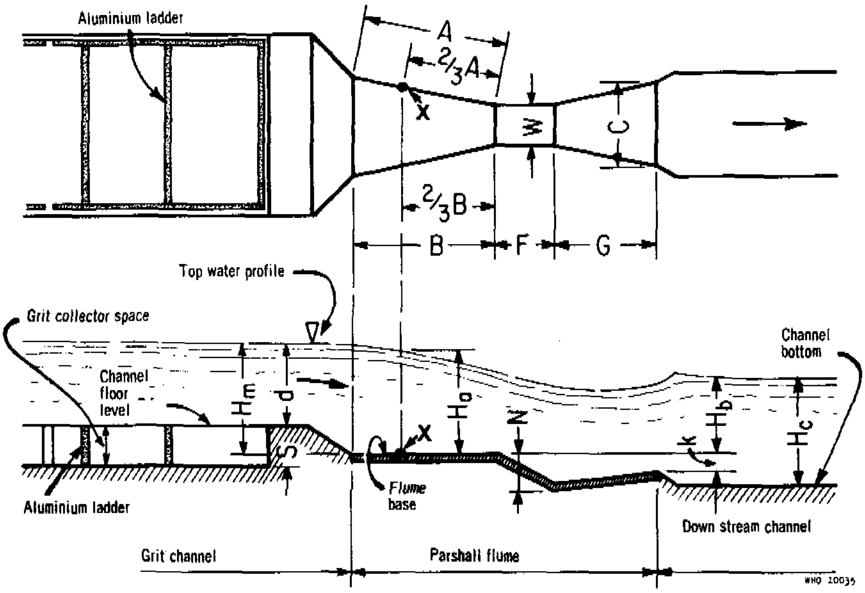
Parabola-shaped channels, proportional flow weirs and Parshall flumes can be used to maintain a constant velocity in the channel. Proportional flow weirs require free-fall conditions, and are not recommended where little head is available. Furthermore, in small installations the weir plates tend to become clogged with rags and other debris. The construction of parabolic grit channels requires careful supervision.

Velocity control by means of a Parshall flume has the following advantages: (*a*) the flume is self-cleaning; (*b*) up to 60 % of the head at maximum flow is recoverable; and (*c*) the grit channel is of rectangular cross-section. Flume sections can be precast and assembled on site, and completely formed plastic flumes are commercially available.

An approximately constant velocity is attained by setting the flume at a lower level than the floor of the grit channel, such that at two flow rates (usually the maximum and minimum expected) the velocity in the channel is equal to a preselected value. For flows between the maximum and minimum, the velocities are slightly lower than the preselected value.

The diagrams in Fig. 23 show a Parshall flume and grit channel in longitudinal cross-section and in plan. The base of the flume forms the reference level for all head measurements. The following equation relates the head ($\mathbf{H_a}$)[1] with the flow rate (Q).

FIG. 23. PARSHALL FLUME AND GRIT CHAMBER



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$$Q = 2.27\,\mathbf{W}\,(\mathbf{H_a})^{3/2} \tag{31}$$

where

$\mathbf{W}$ = width of throat (m)
$\mathbf{H_a}$ = depth of liquid above base of flume at point **X** (m)
Q = flow (m³/s)

The head of liquid upstream of the flume ($\mathbf{H_m}$) is given by the equation:

$$\mathbf{H_m} = 1.1\,\mathbf{H_a} \tag{32}$$

Hence,

$$Q = 2.27\,\mathbf{W}\left[\frac{\mathbf{H_m}}{1.1}\right]^{3/2} \tag{33}$$

The relationship between $\mathbf{H_m}$ and Q for different flume throat sizes is given in Fig. 24.

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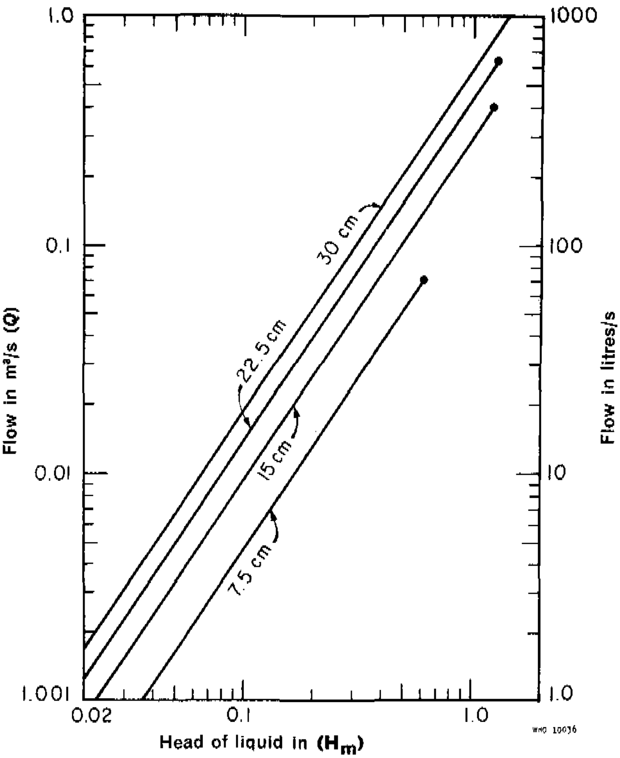
[1] Symbols in bold type refer to flume dimensions as shown in Fig. 23.

FIG. 24. RELATIONSHIP OF FLOW AND HEAD IN A PARSHALL FLUME



• Maximum discharge for flume throat of width indicated.

The dimensions of the flume for different throat sizes are listed in Table 25. The throat sizes only have been rounded to the metric units most nearly corresponding to 3, 6, 9 and 12 inches; other dimensions must be closely adhered to, otherwise the depth/flow relationship may not conform to Equation 31.

TABLE 25

DIMENSIONS OF PARSHALL FLUME FOR DIFFERENT THROAT SIZES *

	Dimensions in cm						
Throat width (W)	A	B	C	d	F	k	N
7.5	46.68	45.73	17.78	25.88	15.24	2.54	5.72
15	62.09	60.97	39.70	39.70	30.49	7.62	11.43
22.5	87.97	86.38	38.11	57.48	30.49	7.62	11.43
30	137.19	134.34	60.97	84.48	60.97	7.62	22.86

* Data from Seelye (1960).

At any intermediate flow value—e.g., 0.1 m^3/s—the velocity in the grit channel can be calculated by reference to Fig. 24:

$$H_m = 0.49 \text{ m}$$

$$d = H_m - S = 0.69 - 0.195 = 0.295 \text{ m}$$

$$v = \frac{Q}{d \times W} = \frac{0.1}{0.295 \times 1.25} = 0.27 \text{ m/s}$$

The velocity of flow (v) in the grit channel is slightly lower than 0.3 m/s between the maximum and minimum flows.

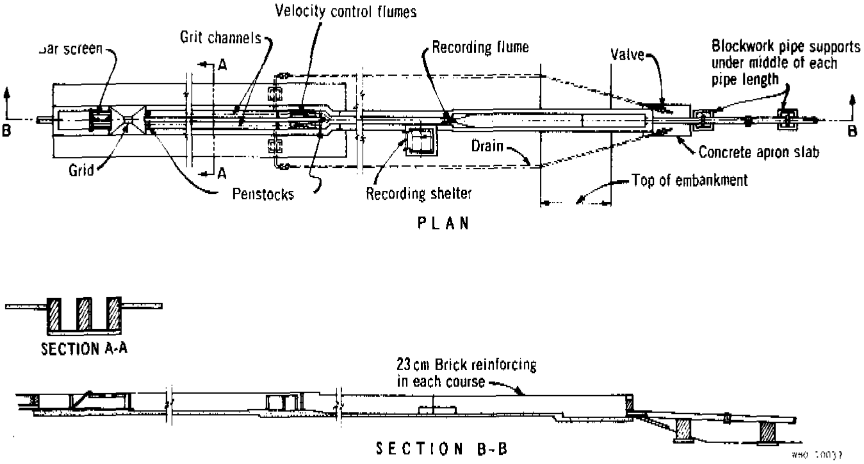
The storage space required for grit depends on the grit concentration in the waste-water and the frequency of cleaning. Storage space is provided below the floor of the grit channel. A series of aluminium "ladders" are placed in the floor of the storage area, the top rung being level with the grit channel floor so that the flow in the grit channel will not be unduly disturbed (Fig. 23).

Normally two channels are provided, so that one can be cleaned while the other is in operation. Gates are placed at the ends of each channel to isolate it and to allow drainage before cleaning. The discharge pipes must be at least 5 cm in diameter, with valves at the entrance and discharge points to prevent frogs from entering and blocking the pipes.

If the flume is to operate in the manner set out above it must not be drowned, i.e., the level of the liquid in the channel downstream of the flume must be such that H_b does not exceed 0.7 H_a (0.6 H_a for a 15-cm flume). For any flow, H_a can be determined, hence the upper level of H_b is known. As draining commences the level of the liquid above the point where H_b is measured is the same as that in the downstream channel. H_b therefore gives the maximum level of the liquid in the downstream channel for that particular flow. If H_c is known, the level of the channel floor is established; otherwise, H_c can be determined easily if the flow is controlled further downstream by, for example, a measuring flume. This calculation is repeated for a number of flows between minimum and maximum, and the lowest channel level is used for the design. A drawing of a small grit channel installation in Zambia is shown in Fig. 25. The difference in level (S) between the grit channel floor and the flume base is determined by Equation 34 (Babbit & Baumann, 1958).

$$Q_a = \frac{1.1\,(Q_{max}/2.27\,W)^{\frac{2}{3}} - S}{1.1\,(Q_{min}/2.27\,W)^{\frac{2}{3}} - S} \qquad (34)$$

FIG. 25. GRIT CHANNELS WITH BAR SCREEN AND VELOCITY RECORDING FLUME FOR A SMALL-SCALE INSTALLATION, ZAMBIA



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where

Q_{max} = maximum flow (m³/s)
Q_{min} = minimum flow (m³/s)
$Q_a = \dfrac{Q_{max}}{Q_{min}}$

Solving for S,

$$S = \frac{(Q_a^{\frac{1}{3}} - 1)}{(Q_a - 1)} \times 1.1 \left(\frac{Q_{max}}{2.27\,W}\right)^{\frac{2}{3}} \tag{35}$$

$$= Q_r \times H_{m\,max} \tag{36}$$

where

$$Q_r = \frac{Q_a^{\frac{1}{3}} - 1}{Q_a - 1} \tag{37}$$

Note that Q_r is independent of the throat width. The relationship between Q_r and Q_a is given in Fig. 26.

An example of grit channel design, using a Parshall flume for flow control, is given below:

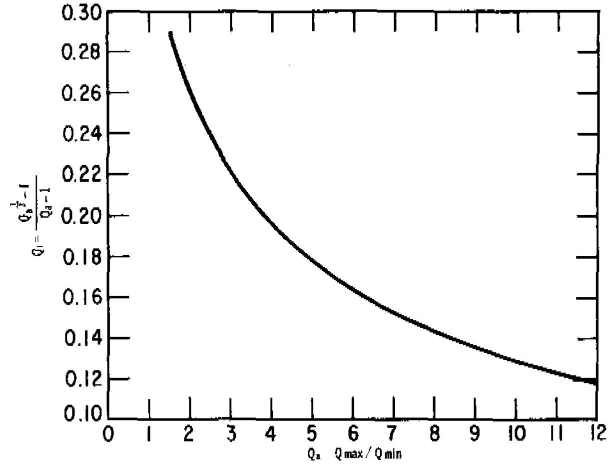
Given

Range of flow (Q_{min}–Q_{max}) = 0.0656–0.351 m³/s
Velocity at maximum and minimum flow (v) = 0.3 m/s

Required

Dimensions for a grit channel.

FIG. 26. RELATIONSHIP BETWEEN Q_a AND Q_r IN A PARSHALL FLUME



Solution

(1) Select throat width (**W**) = 15 cm

(2) $Q_a = \frac{Q_{max}}{Q_{min}}$ = 5.35

(3) From Fig. 26 (Equation 37), Q_r = 0.1725

From Fig. 24 (Equation 33), $\mathbf{H}_{\mathbf{m}\ max}$ = 1.13 m

From Equation 36, **S** = 1.13 × 0.1725 = 0.195 m

(4) Liquid depth in channel at maximum flow ($\mathbf{d}_{max}$) $= \mathbf{H}_{\mathbf{m}\ max} - \mathbf{S}$

$= 1.13 - 0.195 = 0.935$ m

Width of channel $= \frac{Q_{max}}{(\mathbf{d}_{max} \times v)}$

$= \frac{0.351}{0.935 \times 0.3} = 1.25$ m

If insufficient head is available the next larger throat size is selected, i.e., 22.5 cm. This does not influence the value of **S**, but the width of the grit channel increases as the liquid depth in the channel decreases. The channel floor can be set at such a level that the influent pipe to the grit channel is flooded at maximum flow; any solids settling in the pipe are scoured during periods of low flow.

Flow-Measuring Devices

It is desirable to provide a flow-measurement device wherever possible. Without influent flow measurements it is impossible to determine the loading of a pond system, and without effluent flow measurements it is impossible to determine the percolation and evaporation losses. If only one flow-measuring device can be provided, it is preferable to determine the volume of the influent.

The designer of low-cost waste-water treatment systems must choose a flow-measuring device that is essentially self-cleaning. The Venturi flume, Parshall flume, weirs, and adaptations of these are the least expensive and most serviceable devices for measuring open-channel flow.

The Parshall flume is more practical than conventional weirs or Venturi flumes because it is self-cleansing and can operate at lower gradients (Parshall, 1936). It can be built of wood, concrete, sheet metal, or plastic. For small-sized flumes, concrete or plastic components may be precast and assembled on the site. Moulds are available for the construction of plastic flume liners. In the case of free flow, the measurement of discharge in the Parshall flume depends upon a single head or depth only; in this respect it is similar to a weir or a Venturi flume.

For small flows, a staff gauge set flush with the inside face of the flume at the proper distance back from the crest is satisfactory to determine the upper head for free flow, although stilling wells or depth gauges located outside the structure give better results.

FIG. 27. ADAPTABLE V-NOTCH WEIR

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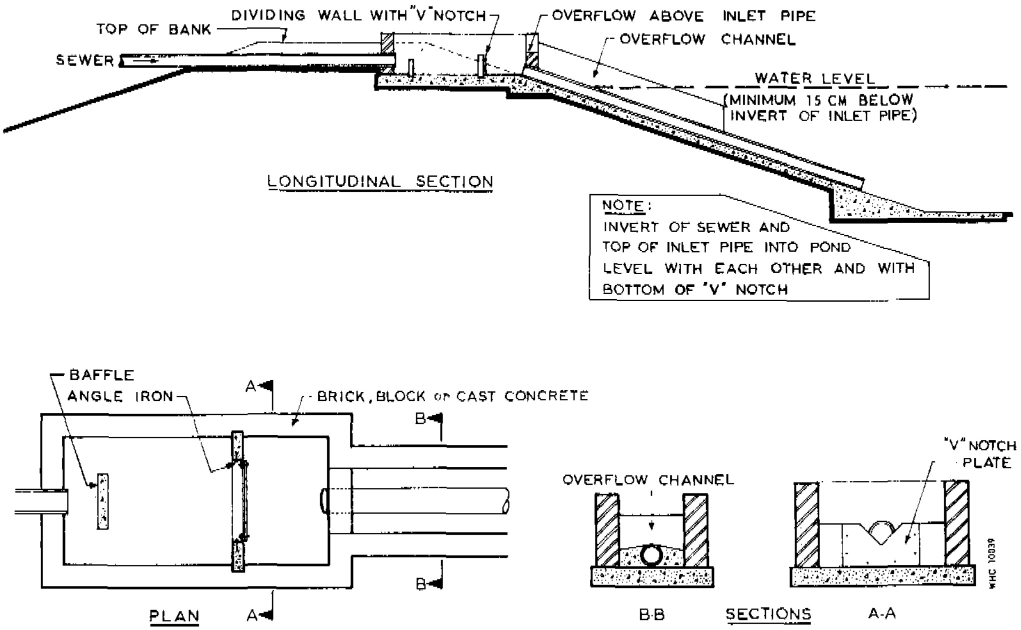
Weirs, either proportional or notched, are simple and useful flow measuring devices. Because of high solids build-up and head loss, however, the V-notched weir should not be used in the influent unless appropriate measures are taken to clean the channel. Fig 27 shows a V-notch weir that can be used in circular sections.

A weir design for inlets carrying pretreated waste-waters is shown in Fig. 28. Although a weir is used, it is reported that grit and settleable organic solids do not present a major problem. Designs of this type have been found practicable in South Africa.

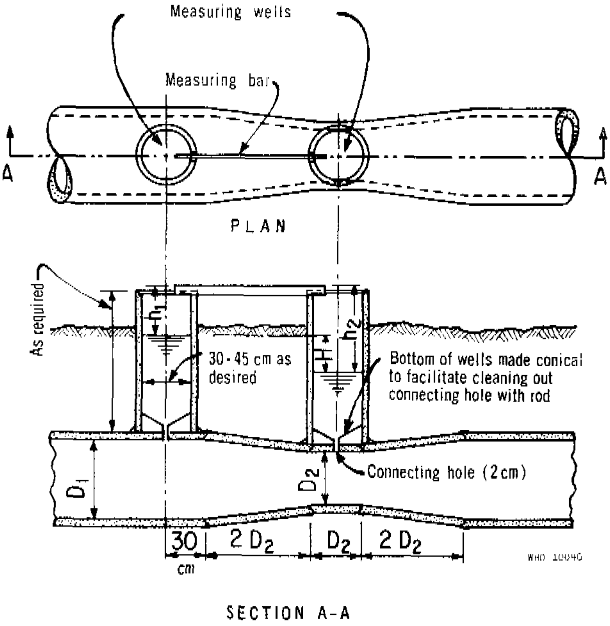
A type of Venturi meter that is relatively inexpensive to construct is shown in Fig. 29. Where a high degree of accuracy and a minimum head loss are not essential, the Venturi meter can be constructed entirely of precast concrete sections. The overall length and cost of the structure can be reduced by making the tapered inlet and outlet sections symmetrical, which makes the outlet section somewhat more abrupt than in the conventional

FIG. 28. INLET FLOW-MEASURING STRUCTURE FOR PRETREATED EFFLUENTS



From Marais (1966), p. 759.

FIG. 29. CONCRETE VENTURI METER



SECTION A-A

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Venturi meter. The throat must be constructed with great care, for the percentage error in flow will be about twice the percentage error in throat diameter. The interior surface, as in any flow-measuring device, must be smooth. Obviously, great care must be taken to keep the measuring wells and connecting holes clear of obstructions. The flow through this modified Venturi tube is given in Equation 38:

$$Q = \frac{c_o A 2 g d_H}{1 - r^2} \tag{38}$$

or

$$Q = s d_H \tag{38a}$$

where

c_o = coefficient of discharge (estimate 1.50)
A = area of throat (m^2)
g = acceleration of gravity (m/s^2)
d_H = differential head (m)
r = ratio of throat diameter to pipe diameter $= \frac{D_2}{D_1}$
s = constant for a particular meter

Piping Arrangements

The most important considerations in designing inlets, outlets, transfer systems, and manholes are size, location, and flexibility of use. The wastewater transfer system must have adequate capacity to carry the maximum waste flow and some storm flow, and must be able to maintain sufficient velocity during periods of minimum flow to transport settleable solids. The hydraulic system may have to operate under extreme ranges of climatic conditions. It is highly desirable to provide service fixtures so that only a minimum amount of maintenance is necessary. For example, the use of concrete or metal collars around pipes buried in dikes will help to control seepage.

The location of each unit must be considered as an individual problem. It should be remembered that the transfer device with the lowest construction costs may prove the most expensive to maintain.

Hydraulics of pond systems

The hydraulic characteristics of the usual waterborne domestic wastes are the same as those of water. Unless they form part of a force main system, however, sewers rarely flow full; furthermore, domestic wastes such as night-soil and settled sludges exhibit plastic characteristics. A partially filled sewer can be considered an open channel. For any circular sewer, the cross-sectional area of the sewage, the velocity of flow, and the discharge vary with the depth of the waste-water in the pipe. Coefficients used to calculate sludge transfer are different from those used to calculate water transport.

Inlets (influent)

Inlets should be submerged, and multiple inlets should be provided for all but the smallest of facilities. Near the point of discharge, the inlet pipe may be buried or mounted on well-anchored supports. Surcharging of the sewer upstream from the inlet manhole should not be permitted. In all probability, horizontal inlets are preferable for gravity flow because of head requirements. However, when the waste-water is pumped or when sufficient head is available, the inlet should discharge vertically upward, except when the waste-water contains large amounts of grit; in this case, the inlet should be on elevated pedestals, but the wastes must still be discharged below the water surface, for this discourages flotation of some materials.

In all cases inlet pipes must be protected from destructive forces such as settlement, ice and erosion. Firm bases must be provided for the pipe and the discharge section. A small concrete pad is usually required below the discharge pipe to provide rigidity, particularly in the case of pipes discharging vertically upward.

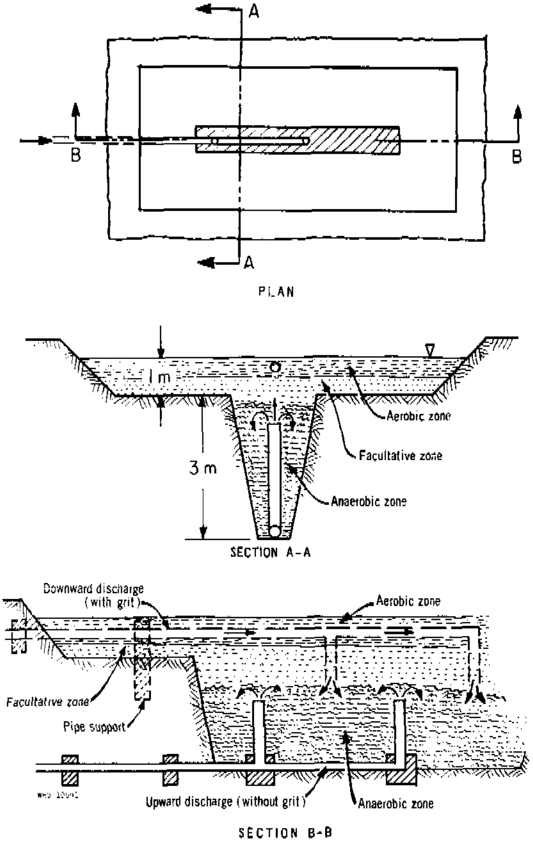
In deep anaerobic pretreatment ponds, the inlet pipe should be located at the centre of the pond. At the point of discharge the influent line should extend upward a metre or two or as described above so as to reduce the possibility of clogging. Additional sand or "digester" capacity can be provided by excavating a little deeper around the inlet.

Another method of providing an inlet to the anaerobic zone of a facultative pond system is shown in Fig. 30, where a number of deeper zones ensure the anaerobic decomposition of settleable solids. Moreover, a large surface area ensures enough photosynthetic action to satisfy both the initial soluble BOD and the BOD released into solution as a result of anaerobic action.

A surface discharge near the embankment invites unpleasant odours and other nuisances. The inlet of a rectangular pond should be located at a point about one-third of the distance along a line drawn from the upstream edge of the pond to the outlet. The development of exposed sludge banks must be avoided. In the case of deeper ponds receiving pretreated wastes, the inlet pipe should extend upward for about the last 50 cm of its length.

A special type of inlet has been devised in India, whereby the waste-water is brought to the surface for measurement and then discharged by gravity through an inlet pipe (Fig. 31). In this case, the bell mouth is turned downward. Such ponds are used when the maximum flow is about 2.25×10^6 litres per day. If no grit removal device is used, two ponds are recommended, to facilitate cleaning and servicing. Proper erosion protection pads must be installed for shallow ponds.

FIG. 30. DISCHARGE INTO AN EXPERIMENTAL FACULTATIVE POND

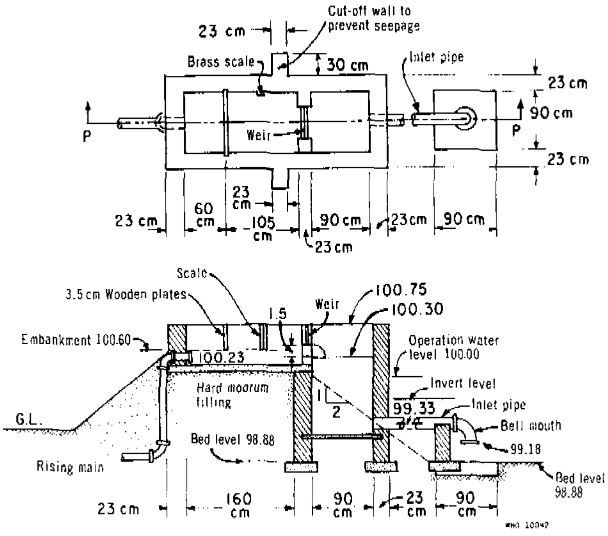


Interpond piping

All influent and effluent piping, including pipes linking series-connected ponds, should be located so as to minimize short-circuiting. In the case of small, simple pond systems, pipes through the embankment are satisfactory provided that adequate precautions are taken to prevent erosion. Erosion control can be accomplished fairly successfully if the connecting pipe discharges horizontally near the pond bottom. To permit efficient maintenance and flexibility in operation, this inter-connecting piping should be valved. Provision should also be made to permit depth control, diversion, or drainage.

A more elaborate interpond connexion is shown in Fig. 32. In southern Africa, it has been observed that during warmer weather it is desirable to draw the effluent from the surface, but that during cold weather a deep subsurface draw-off is preferable (Marais, 1966). A depth of 25 cm was found to be a suitable compromise for year-round operation of a fixed effluent collection system.

FIG. 31. INLET ARRANGEMENT USED IN INDIA



SECTION P-P

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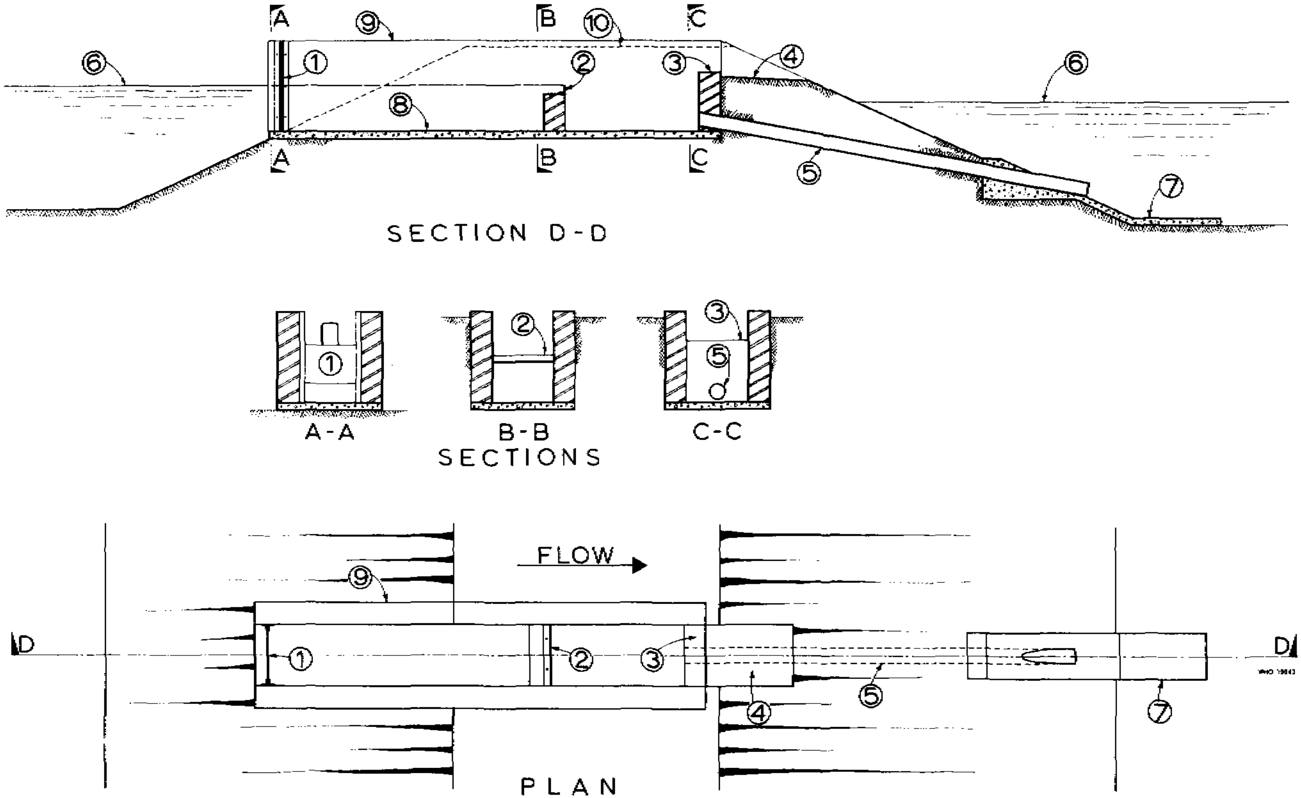
A connexion with a valve box is shown in Fig. 33. This design, which provides a useful means for cutting off the flow between ponds, must be used where a fixed water level is maintained. Some other provision must be made for drainage of the ponds.

Outlets (effluent)

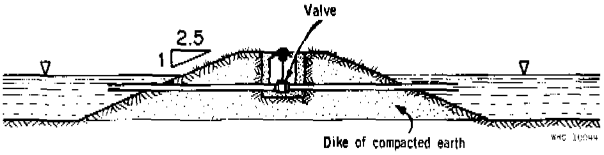
"Outlet design" as used in this section encompasses the hydraulics of the effluent system and the design of the outfall. The outlet is different from interpond connexions because the effluent may discharge into dry, intermittent, or flowing waterways. Provision must be made in the design to ensure proper plant operation under all conditions of stream flow and discharge. Stream flow should never be permitted to enter the ponds. The design of the outfall system is especially important because attention must be given to level control in the pond and quality control of the effluent. Usually, the outlet either is submerged in a stream or provides a free fall whereby the water is propelled away from the pipe before it comes into contact with the ground; the discharge velocity may be controlled for some reason.

FIG. 32. DETAILS OF INTERPOND CONNEXION



After Marais (1966), p. 755.
1. Adjustable scum-board. 2. Overflow weir. 3. Emergency overflow. 4. Bank cut away. 5. Pipe discharging 30 cm above bottom. 6. Top fluid level. 7. Concrete apron. 8. Reinforced concrete floor. 9. Brickwork. 10. Top of embankment.

FIG. 33. INTERPOND CONNEXION WITH VALVE BOX



After F. Saenz (personal communication, 1965).

Effluent systems must be designed so that erosion of the embankment and backflow will not occur. Erosion control may be accomplished by limiting the discharge velocity or by placing a pad below the outlet. The designer must be aware of the frequency of floods and expected high-water levels outside the pond.

A drain pipe is desirable for small ponds and essential for larger ponds. This pipe must be either securely plugged or valved for complete closure. An alternative is to install the outlet pipe in sections and drain the pond by removing various sections of the outlet pipe.

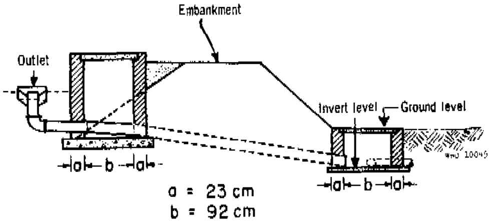
The inlet to the outfall line should be constructed so that effluent may be removed at the depth where it is of the best quality; in almost all cases this will be near the surface. Moreover, all outlets must be baffled to prevent floating material from being discharged. Only in unusual circumstances will it be necessary to provide more than one outfall per pond. Of course, some designs allow the interchange of ponds or changes in the sequence of operation, and in this case several outfalls are required.

In addition to topographic features and the prevention of short-circuiting, the design must take into account the prevailing wind direction. The wind may cause a considerable degree of mixing and turnover. In cases where the surface waters were saturated with dissolved oxygen, were low in BOD, and showed no trace of hydrogen sulfides, a wind blowing from the outfall towards the inlet has been known to bring bottom waters of low quality to the surface near the outfall. Such mixing may create an unacceptable effluent. Where wind can cause difficulties the overflow collector must be located a short distance from the bank.

The simplest and least expensive forms of effluent collector are a pipe placed in the dike at the desired elevation or a concrete spillway constructed as an integral part of the dike, but both have the drawback that the water level in the pond cannot be varied. The advantage of the concrete spillway is that it can be used as a flow-measuring device.

Most outfall collectors are of the overflow-weir type. Broadcrested weirs constructed of boards or concrete slabs can be used effectively to control the water level.

FIG. 34. INDIAN OUTLET DESIGN

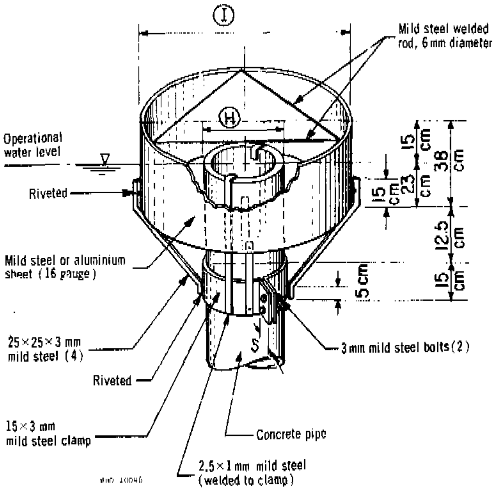


Reproduced, in modified form, from Mehta (1966), p. 8, by permission of the publishers.

In an outlet design shown in Fig. 34 provision has been made for cleaning the pond, changing the water level, and if desired, installing a flow-measuring device. This is a useful design, but requires a large amount of labour and concrete. Details of the scum ring are shown in Fig. 35.

Circular and rectangular box weirs are suitable collection devices for smaller ponds. The circular weir shown in Fig. 36 has the advantage of taking high quality water from the surface. Stacked pipe will permit the depth to be changed. A screen helps to keep floating matter out of the effluents, and a wooden box that can be raised or lowered around the screen will permit water to be withdrawn from any depth.

FIG. 35. DETAILS OF SCUM RING FOR INDIAN OUTLET



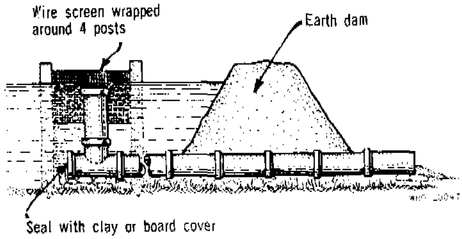
Reproduced, by permission, from Dave & Jain (1967), p. 140.
I = 3 H.
H = 1.5 × flow rate in cm/s.
The flow rate is assumed to be 90 cm/s.

Details of a more elaborate overflow structure are shown in Fig. 37. A noteworthy feature is the riser, which can be disconnected to aid in water level control, including drainage. The baffle plate shown in the sectional drawing should be effective in keeping scum and other debris out of the effluent pipe. Another feature that operators will appreciate is the sturdy walkway and handrail. The purpose of the concrete stop placed around the pipe near the centre of the embankment is to prevent water from seeping along the outside of the pipe between the pond and the receiving body of water.

An unusual outflow device has been designed for a 2-pond system with a combined area of 1.0 ha that serves a meat packing plant in Costa Rica (F. Saenz—personal communication, 1965). The outfall, which consists of two pipes with a valve on one of them, is fixed so that water can be taken from either 90 cm or 1.5 m from the pond bottom. However, this facility may cost more than the unit shown in Fig. 36.

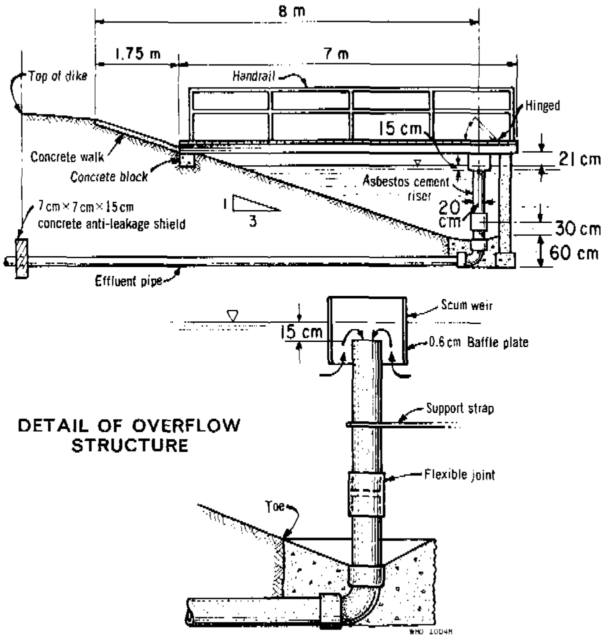
FIG. 36. CIRCULAR COLLECTION WEIR

SECTION THROUGH DAM



After Clay Sewer Pipe Association (1965), p. 102.

FIG. 37. MULTI-PURPOSE OUTFALL COLLECTOR



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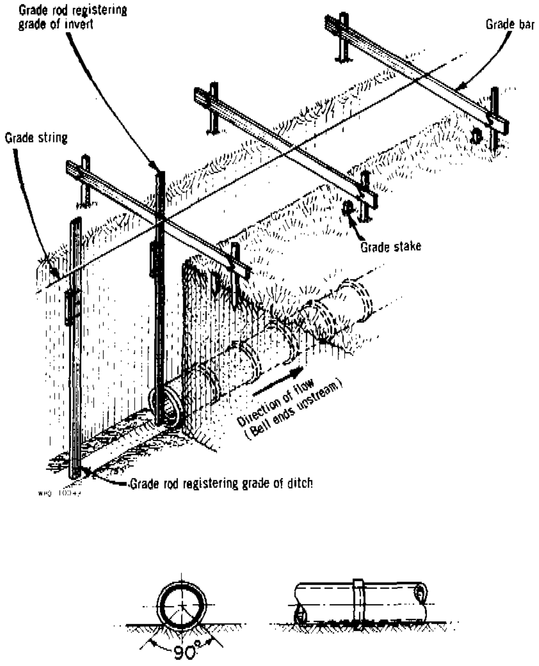
Transfer devices

Manholes or transfer wells may be required in a complex of ponds that must offer a maximum degree of operational flexibility. If return flow has to be pumped, wet wells may be necessary. Some engineers recommend that manholes or clean-outs be located where pipes pass through the embankment (Great Lakes—Upper Mississippi River Board, State Sanitary Engineers, 1960). The manholes may be constructed of precast concrete piping, poured concrete, concrete blocks, or bricks. If dense concrete is used, it is usually not necessary to plaster the inside wall, but the wall must be reinforced with steel. If bricks are used, the inside and outside walls of the manholes must be plastered. The bottom is normally of concrete, extending about 15 cm outside the walls, and a smooth channel runs along it from the entrance pipe to the exit pipe. Frostproof overflow manholes or valve boxes are recommended where freezing conditions are likely to occur. Force mains carrying waste-waters to the treatment plant should be valved at the inlet to the treatment plant, but in most cases the waste-water is carried to the plant by gravity.

Gravity-flow sewer lines are generally constructed from clay or concrete pipe. Fig. 38 shows a common method of laying clay pipe to line and grade.

FIG. 38. LAYING PIPE TO LINE AND GRADE



After Walton & Chase (1964), p. 87.

The grade rods are used to establish both vertical and horizontal levels. The lowest quarter of the pipe surface should be in firm contact with undisturbed earth or a granular bedding such as sand and small gravel. Extra excavations should be made for the bell of the pipe, but should be no larger than necessary. Where expansive clays are encountered considerable care must be taken, and the immediate surrounds of the pipe should be well prepared with sand and gravel to provide some stability. Where sewer pipe is laid across a very wet and fluid section of earth it may require anchoring to prevent it from floating. The best way to reduce unwanted inflow of groundwater or outflow of waste-water is to prepare the trench properly, select suitable pipe, and inspect the laying of the pipe with the utmost care.

Earthwork

Earthwork includes clearing and grubbing, excavation of the ponds, construction of the embankments, protection of the bank, sealing of the pond bottom, grassing, fencing, and excavation work connected with the construction of the access road, drainage ditches, and associated facilities such as pumping stations.

The entire pond and embankment area should be cleared of trees, stumps, brushwood, grass, rocks, and any other material not suitable either for filling or for pond operations. The material collected should be removed from the site or burned on the site and the residue removed.

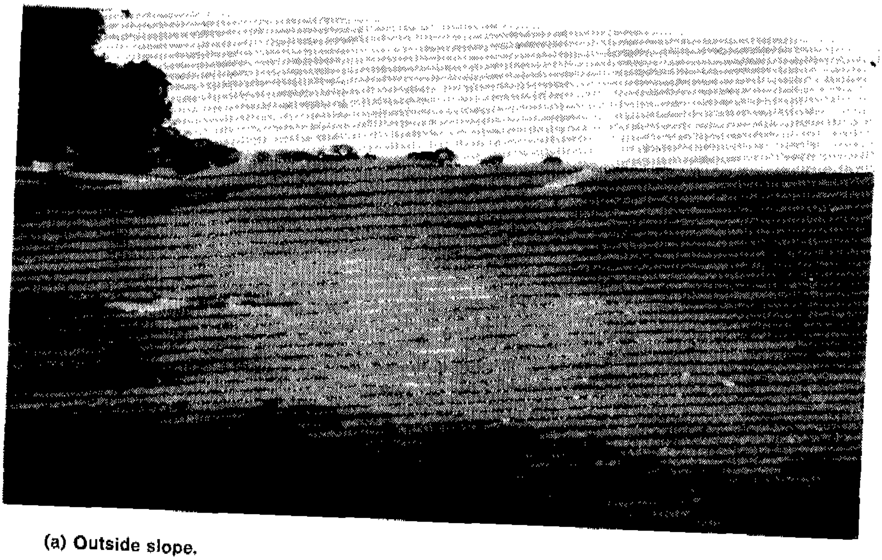
All ditches should be finished to the required elevation and cleaned up in a workmanlike manner to present a neat appearance. The excavated material should be used as filling in the construction of the pond embankment or access road.

The access road should have an all-weather surface. Local gravel or rock are satisfactory materials for this. Where a considerable number of vehicles may use the road or where trucks are employed for hauling night-soil, draining septic tanks, etc., a substantial and well-drained road surface is required.

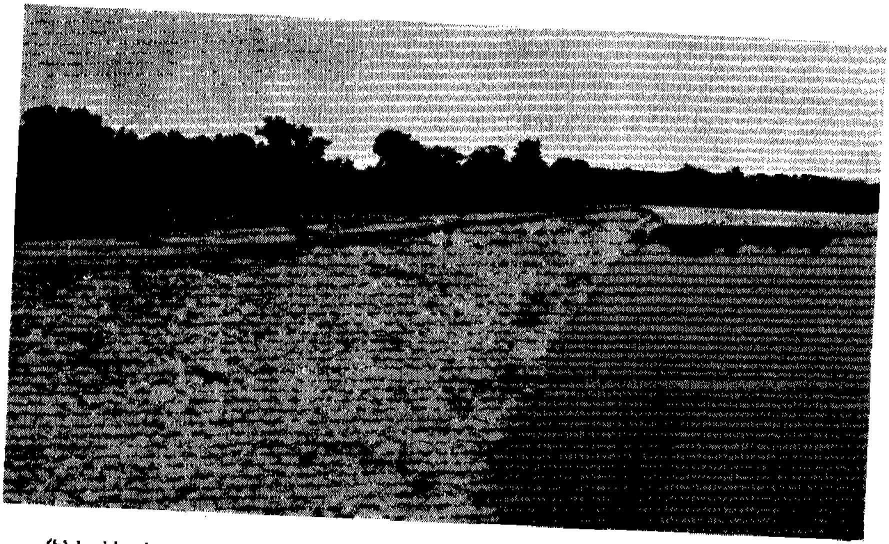
The dikes (embankments) for large ponds should be constructed very carefully. There must be close supervision of the selection and placement of filling material, the measurement of moisture content, and compaction. Material containing organic matter, such as grass, must not be used in constructing the core of the embankment.

All top-soil must be removed and a suitable foundation prepared before filling material is placed in position. The most impervious material available must be placed in the centre of the embankment. All "soft" spots must be excavated to a depth such that the base will support the load imposed by the embankment. Successive loads of fill must be placed so as to produce the best practicable distribution of material. Each successive

FIG. 39. A WELL-PROTECTED EMBANKMENT



(a) Outside slope.



(b) Inside slope.

Photographs kindly supplied by Mr Albert H. Ullrich, Director, Department of Water and Wastewater Treatment, Austin, Texas, USA.

layer of filling material should have the optimum practicable moisture content required for compaction purposes. If possible, the moisture should be added when the filling material is applied, for otherwise supplementary wetting of the earth fill may be necessary. Uniformity of moisture content may be obtained by raking or harrowing.

Where it is possible, every effort should be made to seal the bottom of the pond. Particular note should be taken of points where trees were formerly located, and all roots must be removed to a depth of at least 30 cm below the surface before compaction is commenced. If heavy compaction equipment is available, bottom consolidation is readily accomplished. Should soft spots develop during rolling operations, clay should be spread over them to a depth of 15 cm. Where heavy equipment is not available and the soil is permeable, it may be necessary to place a clay layer over the entire pond bottom.

The method used for protecting the embankments will depend on the extent of protection desired and the type of material used. The outside of the embankment should be protected from erosion caused by surface run-off and wind action. Usually grasses will provide adequate protection; if not, some form of rock or gravel protection must be incorporated into the plans. Fig. 39 shows examples of well-protected inside and outside slopes. It should be noted that the outside slope is free from weeds and trees. Appropriately graded rock, flat stone, precast concrete slabs, or some other suitable liner must be placed on the inside edge as a protection against wave action. Gravel can be used as a substitute for crushed rock, but small gravel has a tendency to move down a slope through wave action. Gravel has been used in conjunction with crushed rock or concrete mats to prevent erosion.

If the rock does not extend for the full height of the inside face, a shelf must be constructed just below the low-water mark to act as a support for the rock (Fig. 40). The placement requirements, type of rock, and

FIG. 40. SECTION OF PARTIALLY-PROTECTED EMBANKMENT

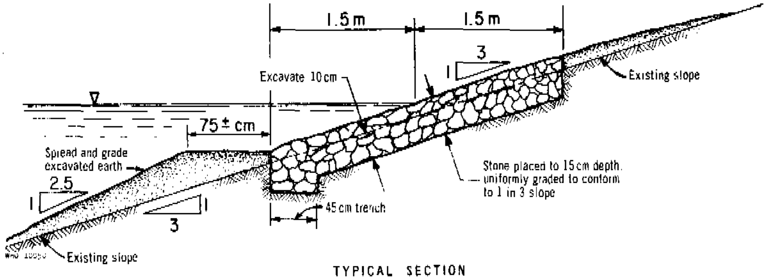


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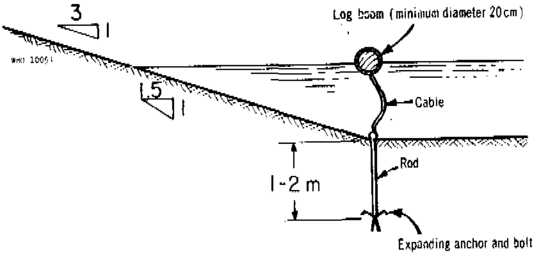
FIG. 41. USE OF LOGS FOR BANK PROTECTION



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slope details will depend upon local conditions. The availability of materials, size of the embankment and manpower costs must ultimately determine what erosion control measures are taken. The cost of rock can be reduced substantially by this partial facing technique, but where labour costs are an important consideration it is usually cheaper to face the entire inner bank with rock.

Some engineers have used logs as a means of controlling bank erosion. One method is to fix logs horizontally at the water level on to vertical log supports. This provides a considerable amount of protection, but not as much as crushed rock. A novel means of protection by attaching a log to a flexible support is shown in Fig. 41.

CHAPTER 7

Responsible Operation

This chapter summarizes the various steps involved in the development of a waste-water treatment facility and in the operation of the plant, and briefly develops a guide for sound management. Many of the points have been discussed in detail in earlier chapters. Clearly, not all of the material presented here will be applicable to every location. Due allowance must be made for local customs, the type of governmental control, economic conditions, and the availability of operating technicians.

Initial Considerations

The first step in the development of a waste-water treatment plan is to consult the local and central governmental authorities. Often this may involve only a single health agency or water board, but in other cases there may be several governmental agencies concerned with water supply and pollution abatement.

After the effluent quality and the type of waste stabilization pond system have been determined, the engineering designs and cost estimates can be prepared. Typical questions that arise at this stage include: Will the local authority finance the sewerage system? In view of the anticipated revenues, will the waste-water disposal services be developed by a private company? Is the community or industry concerned willing to finance the treatment project?

The owner of a sewerage system, whether private or governmental, is responsible for supervision and satisfactory operation, and should be able, through either direct or indirect revenues, to meet the system's operation and maintenance costs. If capital needs to be attracted, sufficient revenues must be obtained within a reasonable period to pay for construction, operation and eventual replacement.

No single regular source of funds is commonly used for the financing of municipal or industrial waste-water treatment units. The annual costs, including operation, maintenance, interest, and depreciation, may be met by income derived from a hidden levy on metered water consumption, a standard charge for each sewer connexion, a pickup charge for each household, a property improvement tax, special assessments, or some other form of taxation.

Responsibility of Management

Although the operation of a waste stabilization pond system is simple compared with that of a highly mechanized and instrumented activated sludge treatment plant, it is not so simple that it can be neglected for very long. The waste stabilization pond must operate continuously under all types of adverse conditions. Regular maintenance is necessary because the waste-water treatment plant can rapidly become a liability if it is not properly handled.

A waste treatment plant is an investment that must expand with the community or industry it serves. The authority that owns it, having accepted the responsibility for providing a more healthful environment for the community through the operation of a waste-water treatment plant, should specify maintenance practices and provide the funds to carry out effective programmes of waste treatment. This usually means providing funds for repairs, for salaries that will attract competent and keen operators, and possibly for modernization and expansion of the treatment facility as the need arises.

All operators of waste treatment plants must be taught to appreciate that they are in charge of a substantial community or industrial plant investment, and also that they are members of a very important community health team with the responsibility of providing a more healthful environment.

Maintenance must be incorporated into all levels of planning, design, construction, management, and operation. Beginning with the planning of the facility, management must establish an effective operation and maintenance programme by formulating and adopting an organizational chart that clearly defines a chain of authority and responsibility. Employees must be selected and trained; tools and materials must be provided. Finally, operators must be given regular instruction in maintenance; each operator must be thoroughly acquainted with the planned or "preventive" maintenance that must be routinely performed and with any "curative" maintenance that may be required in an emergency. Literature on maintenance and operational requirements is available (Mahlie, 1964).

Organization of Personnel and Plant

It has been found that people accept responsibility and work most effectively when there are well-defined lines of communication. A well-organized staff is necessary for the success and effectiveness of the plant.

Because the waste-water treatment plant must be well-maintained to prevent odours and other nuisances, each employee should be taught not only his own immediate duties but also the rudiments of the entire process and its significance for health. Employees should be encouraged to keep the plant, and themselves, as neat as possible, and should be shown how proper maintenance and operation can help to cut down the work as well as reduce nuisances. At more elaborate facilities, an instruction manual covering both operation and maintenance should be made available.

Personnel management may not be an important problem when a small pond in a developing country is being considered, but these same countries may have growing metropolitan areas and industries that will in due course require larger and more complex facilities.

Design Checks

Many problems can be overcome by close co-operation between the design engineer, experienced local operators, and the health authorities. Each locality will have some special requirements, but some points regarding design that are of general applicability are discussed below.

Present load

When facilities are designed, allowance should be made for the future growth of the community or industry, and the master plan should include adequate space for expansion. For the most effective utilization of waste stabilization ponds, however, it is best if only the portion that is currently needed is constructed. Ponds that are not used deteriorate rapidly and become a maintenance problem, and in any case there is no need to tie up funds in facilities that are not yet required. Adaptability is a distinct advantage of the waste stabilization pond process.

Grading

The site should be graded for proper drainage. The dikes must be graded uniformly before grass is planted. It is impossible to cut weeds effectively either by hand or by machine on unevenly graded embankments. Surface water run-off and sediments must be diverted and must not be permitted to enter the pond.

Roadways

Roads and walkways must be located so as to provide ready access to all inlets, outlets, transfer stations, tool sheds, and the banks of all ponds. In large facilities, any laboratory or office buildings must be reasonably close to the treatment plant. All structures must be interconnected with walkways.

Embankments and dikes

Embankments must be constructed of impervious materials. As a general guide, embankment slopes should probably not be steeper than 1 in 3 or flatter than 1 in 6. The top of the embankment should be high enough to prevent overflow during heavy rain and wind storms—normally about 1 m above the water level.

Provision must be made to protect the embankment from erosion. Embankments should be covered with top-soil and seeded with grass immediately after construction. Additional protection, in the form of crushed rock, gravel, soil cement, precast concrete slabs, etc., should be used to prevent erosion of that part of the embankment exposed to wave action. Where extensive mosquito infestation may occur, concrete slabs are preferable to crushed rock since mosquitos may breed in the protected areas around the rock.

Dikes should be rounded at the corners to minimize the accumulation of floating materials. Any sharp corner or inlet will create a nuisance of this kind and require additional maintenance.

Depth

The minimum liquid depth in the ponds should be 1 m. Where cold seasonal conditions prevail, it is desirable to use deeper facultative ponds 2–3 m deep. Similarly, anaerobic pretreatment and mechanically assisted ponds require depths of 2–4 m.

Pipes, valves, and overflows

All pipes and valves should be located so as to permit maximum flexibility in plant operation. All interconnecting piping and overflows should be of suitable material and of ample size. Overflow lines should be vented if siphoning is likely to develop. Effluent lines should discharge on to anchored concrete slabs to prevent erosion.

Where freezing may occur, the use of frostproof overflow manholes or valve boxes for controlling liquid levels in the ponds is recommended. Manholes and clean-outs are desirable where pipes pass through the embankment.

Selection of materials

Careful consideration must be given to metallic materials used in construction, for waste-water treatment plants provide an extremely corrosive environment. Copper, brass, and cadmium-coated materials are sensitive to ammonia and hydrogen sulfide and consequently should not be used. Aluminium, stainless steel, and vitrified clay are relatively resistant to corrosion. Transportable plastic pipe is ideally suited to some situations, but is relatively costly and may be structurally weak.

Insect control

Suggested methods of minimizing mosquito breeding through the design and control of construction features include: (*a*) designing the pond so as to permit complete water-level control and drainage, (*b*) clearing vegetation from the bottom before filling, (*c*) levelling the bottom, (*d*) designing the pond to hold water by artificially sealing the bottom with chemicals, clay, or other materials, and (*e*) proper maintenance of the embankment slopes.

Fences

It is desirable to erect some type of fence around the waste stabilization ponds. Signs describing the facility should also be displayed.

Tool and Material Requirements

A certain number of hand tools are required, and the operator to whom they are supplied should be responsible for their proper use and maintenance. Hand tools that might be most useful include lawn mowers, rakes, axes, files, oil-stones for sharpening tools, spades, forks, wheelbarrows, saws, claw hammers, measuring tape, pliers, screwdrivers, wire-cutting pliers, metal shears, pipe cutters, pipe seamers, pipe wrenches, pipe vices, grinding wheels, and scrubbing brushes.

For some facilities it may be desirable to provide a portable petrol-driven pump, light and heavy duty hoses, a portable insect spray applicator, a small rowing boat, grease guns, suitable grease and oils, a supply of paints, and a stock of certain spare parts that are not readily obtainable.

Operation and Maintenance Checks

Operational control

Provision should be made for determining the volume and BOD concentration of the waste-water. For large facilities, routine flow and analytical measurements should be made.

Flow measurements are needed to determine the most efficient mode of operation and to provide data for future additions to the treatment plant. The installation of a simple Parshall flume in the influent and a V-notch weir in the effluent would be satisfactory, although valuable data can be obtained with only one flow-measuring unit. Periodic flow measurements should be taken and recorded. Daily and seasonal variations in flow should be noted, along with the characteristics and performance of the ponds. A simple note-keeping and data-recording system should be initiated.

Periodic measurements indicating the influent and effluent concentrations of BOD and coliform organisms are desirable. These tests can be carried out by inspectors sent from a central laboratory.

Additional measurements that might make for better control and more efficient operation include: (*a*) diurnal pH and oxygen fluctuations in the ponds, (*b*) pH of the influent and effluent, (*c*) total solids, suspended solids, and volatile solids in both influent and effluent, (*d*) total organic nitrogen, ammonia, nitrates, and phosphates in the influent and effluent, (*e*) oxidation-reduction potential in the various ponds, particularly in the anaerobic pre-treatment unit, (*f*) detailed chemical analyses of either the influent or the domestic water supply, (*g*) chemical oxygen demand, particularly if industrial wastes are involved, (*h*) ultimate BOD, (*i*) sulfate ion content of influent waters, and (*j*) the biochemical degradation rate K_T for various temperatures. Obviously, not all of these measurements are necessary for the operation of small or medium-sized facilities, but the list illustrates what might be desirable for the best management of larger and complex facilities, including research establishments.

Control of insects

The degree of mosquito infestation in ponds is in direct proportion to the extent of emergent vegetation. Both *Culex* and *Anopheles* larvae have been found in poorly operated ponds. If water is available, the ponds should be filled immediately to operational levels to discourage the growth of vegetation. Undesirable vegetation in the ponds or on the dikes must be eliminated periodically by cutting or by using a suitable herbicide. The cut plants must not be permitted to float on the pond, for this will provide shelter for mosquitos. Similarly, water-loving trees such as willows and poplars should not be planted around the ponds.

Larvicidal measures should be undertaken if significant mosquito breeding takes place. The following larvicides have been used effectively in some ponds: a thin layer of kerosene or diesel oil, 2 % DDT and oil, lindane dust (3 % gamma isomer), and 2 % malathion. Caution should be exercised when using these substances, for excessive use may produce harmful effects on receiving watercourses.

The introduction of top-feeding minnows (*Gambusia*) to secondary or tertiary ponds is another effective way of controlling certain larvae. Other fish, such as *Tilapia mossambica* and *T. melanopleura,* and the guppy (*Poecilia reticulata*) have also been used successfully for the control of mosquitos in maturation ponds and some underloaded facultative ponds.

The heavy scum layers that may form on anaerobic ponds can be conducive to fly breeding. Both the common housefly, *Musca domestica*, and the filter fly, *Psychoda*, common in trickling filters, can breed in or around anaerobic pretreatment ponds and at the scum-laden edges of poorly operated facultative and aerobic ponds. One method of fly control is to break up the scum by frequent wetting. Fly-traps with poisoned bait can also be used around anaerobic pretreatment units.

Around anaerobic ponds, particularly those in which animal wastes are treated, flies can become a serious problem. The larvae of the common housefly, *Musca domestica*, and the biting stablefly, *Stomoxys calcitrans*, develop in most forms of organic sludge. Fortunately, the ponds themselves are not as attractive to the adults of these species, nor are there likely to be live maggots in well-managed anaerobic ponds, although *Musca* larvae and pupae may be contained in poultry manure and some adults may emerge if the manure accidentally floats to the surface (Hart & Turner, 1965).

Midges have occasionally created a nuisance. The larval and pupal stages are passed in mud that is rich in organic matter, and the adults frequently emerge in swarms. In New Zealand, the indigenous midge *Chironomus zealandicus* has caused considerable trouble when ponds were not loaded to the capacities for which they were designed. However, the midge population can be controlled by increasing the organic load on the ponds. Chemical control around the perimeters of the pond has also proved initially effective, although it has often been followed by the development of insecticide resistance.

In the USA, probably as many as 20 species of aquatic midges frequent various types of ponds. The biting midge (family Heleidae) and the pest midge (family Tendipedidae) are the most important. Gnats are sometimes found around spreading basins and in effluent channels.

Dike maintenance

Embankments should be inspected regularly for erosion due to wind, wave action, surface run-off, or burrowing animals. Any necessary repairs to the embankment must be made immediately after the damage occurs. The dikes should be seeded with grasses, fertilized and mown. Long-rooted plants such as alfalfa should not be used because they may impair the water-retaining capacity of the dike.

Control of odours

Odours may arise from a number of situations. Frequently they are associated with the decay of mats of algae that have been blown to a bank or corner. *Chlamydomonas*, for example, can grow rapidly, spread over pond surfaces, reduce the penetration of light to the remainder of the pond, and with the assistance of the wind accumulate in the corners where it decomposes and produces vile odours. In other instances, particularly during periods of high water temperatures in shallow ponds, sludge mats rise from the bottom. These masses of organic debris usually accumulate in corners, and if it is not disturbed the entire mass may become covered with blue-green algae. Usually the bacterial activity is intense and the odours are overpowering.

The solution to the mat problem is immediate dispersal. Agitation of the surface will usually cause the floating mass to break up and settle to the bottom. A jet of water from a garden or fire hose will normally create enough turbulence to achieve this (Fig. 42 a). Another remedy employed by some is to use an outboard motor or an engine-powered paddle wheel to agitate the surface (Fig. 42 b). Such devices have the advantage of flexibility, in that they can be mounted on rafts and moved from place to place.

Recognition of characteristic colours

In all probability, an odour or a change in colour is a warning of a major change in the performance of a pond system, and the operator should be alert to recognize such signs. Frequently, odours accompany illicit waste releases that cause a rapid change in the pond biology.

Normally a maturation pond has a characteristic green colour; the facultative pond most frequently appears green or brownish green, but may occasionally look quite pink or exhibit some other variation in colour; the anaerobic pond looks greyish black.

When the characteristic green colour of a pond begins to change or disappear, the operator should look for things that may be causing this. Changes in the volume, organic load, temperature, light, turbidity, etc., may cause changes in the existing algal pattern. A colour change from green to black, accompanied by floating mats of material from the bottom of the pond, usually indicates rapid fermentation of the bottom sediments, frequently as a result of changes in pond temperature or in the character of the waste-water.

Occasionally ponds receiving either domestic or industrial wastes develop a pink colour. This is sometimes occasioned by the development of coloured micro-organisms, particularly in facultative ponds during the summer and autumn if the sulfide or sulfate concentration is high. Chief among the types that have been noted are *Chromatium*, *Thiospirillum*, and *Thiopedia*, but other small rod-shaped and spiral forms have also been detected. The larger micro-organisms such as *Chromatium* and *Thiospirillum* are mostly restricted to ponds receiving typical municipal wastes, while *Thiopedia rosea* and smaller forms are likely to be present in ponds receiving industrial wastes. Generally these micro-organisms are associated with waters that contain excess hydrogen sulfide.

The presence of such coloured micro-organisms seems to indicate some prior overloading, stratification, or operational deficiency. Invariably, a pond receiving industrial wastes that contain relatively large amounts of BOD and hydrogen sulfide or sulfates will support periodic blooms of these micro-organisms.

FIG. 42. DISPERSAL OF FLOATING MATS OF ALGAE

(a) by means of a jet.

(b) by means of a paddle wheel.
Photographs kindly supplied by Mr Albert H. Ullrich, Director, Department of Water and Wastewater Treatment, Austin, Texas, USA.

Sulfides in a pond can be oxidized by colourless sulfur bacteria and by coloured photosynthetic bacteria. The top layer of the pond, where both dissolved oxygen and hydrogen sulfide can occur, offer the best environmental conditions for the growth of colourless sulfur bacteria, but such bacteria are not often found in stabilization ponds, or occur only in rather small numbers. The oxidation of sulfides by colourless sulfur bacteria is therefore negligible compared with that by photosynthetic bacteria and by dissolved molecular oxygen.

Photosynthetic sulfur bacteria are very often found in large numbers in stabilization ponds, particularly during the summer and autumn, and impart to the pond a characteristic brown or red colour. The presence of such bacteria has been reported in many different types of pond, including sewage ponds (Parker, 1962), refinery waste ponds (Cooper, 1963) with reported concentrations of 1.4×10^7 *Chromatium* cells/ml, rendering waste ponds (Cooper, 1963) with reported concentrations of 6.17×10^7 *Thiopedia* cells/ml, and a series of ponds in Pullman, Washington, USA, receiving the effluent of a research installation (Green, 1966).

In all of these cases, the appearance of photosynthetic sulfur bacteria was accompanied by a reduction in hydrogen sulfide odours; for this reason these bacteria are sometimes referred to in the literature as "biological deodorizers".

In a comprehensive study of the activities of photosynthetic sulfur bacteria in waste stabilization ponds, Green (1966) found that the brown colour sometimes imparted to the pond occurs when the concentration of sulfide in the pond is high and the photosynthetic bacteria store it as sulfur granules in the cell; when the sulfide concentration decreases and the sulfur granules are oxidized by the cell, the colour imparted by photosynthetic sulfur bacteria is a vivid red. Because of their photosynthetic activity, these bacteria do not help to reduce the BOD of the wastes and may even tend to increase it (Green, 1966). It should be remembered that this photosynthetic activity does not produce any molecular oxygen. When photosynthetic sulfur bacteria appear in a pond, they are initially found in the middle zone (zone 2, Fig. 43), where anaerobic conditions exist and both light and hydrogen sulfide are available. The cell concentration thus tends to increase, and if the environmental conditions are favourable the cells will rise to the top layer of the pond where better illumination is found.

The presence of photosynthetic sulfur bacteria has also been reported in several lakes, with similar effects. Detailed studies have been made on Lake Mendota, Madison, Wisconsin, USA (Ruttner, 1953).

Control of trees

Trees will interfere with the performance of the pond. Large trees will impede the natural wind action and may reduce the light intensity at the surface of the pond. Moreover, leaves falling into the pond will interfere with photosynthetic processes, add to the BOD load, and possibly create insect control problems. Wherever possible, large trees within 40–60 m of the pond embankment should be removed.

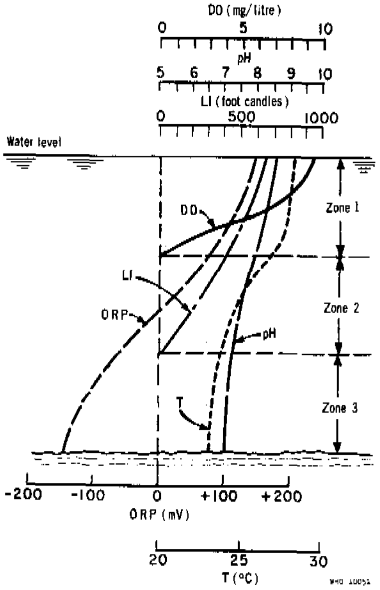
Recognition of mixing influences

The hydrodynamic and physical shape of a pond has a distinct effect on its performance. For example, surface aeration and photosynthetic oxygenation can be enhanced by both thermal mixing and wind action. Both forms of mixing are beneficial.

When the temperature is falling, mixing can occur without wind action. During periods of rising temperatures, however, wind action is required, and then stratification usually occurs. Dye tests have verified these observations.

Wave action is determined largely by the fetch, i.e., the continuous extent of exposed water. A fetch of about 300 m will usually ensure circulation and mixing to a depth of 1 m.

FIG. 43. TYPICAL STRATIFICATION CURVES IN FACULTATIVE WASTE STABILIZATION PONDS



Data from Espino de la O & Gloyna (1968), p. 14.
DO = Dissolved oxygen
LI = Light intensity
ORP = Oxidation reduction potential
T = Temperature

Start-up of Ponds

The general plan of operation must include a programme for filling the pond and initiating stabilized operational conditions. Clearly a pond will not immediately accept the full loads for which it is designed, and an adjustment period equivalent to several detention periods is necessary. The time taken to reach equilibrium will depend on the type of waste, type of pond, and mode of decomposition.

Facultative ponds

A facultative pond cannot immediately receive its full BOD or hydraulic load. Firstly, algal growth cannot become established as rapidly as the bacterial population; secondly, the pond may have to seal itself; and thirdly, the full population for which the pond is planned may not be connected immediately to the sewerage collection system. Normally the ponds are filled gradually, but ideally it is better to fill them with water and some digested sludge before introducing the waste-water. If a pond is filled gradually with untreated waste-water, it is desirable to place small dikes about 50 cm high across the pond. This temporary diking will assist in sealing the bottom rapidly, while maintaining sufficient depth of water to control weeds.

Another suggested procedure is to fill the facultative pond and provide an adaptation period (S. V. Ganapati—personal communication, 1968). In this method the pond is filled as rapidly as possible with waste-water to a depth of about 1 m and then left undisturbed for a period of 10–20 days, or until the pond turns greenish or bluish-green. This adaptation period, the length of which will depend on the water temperature, represents two phases: the bacterial phase followed by the algal phase. During the algal phase *Chlorella*, *Scenedesmus*, *Chlamydomonas*, *Euglena*, *Oscillatoria* or combinations of these will usually develop. There is normally no need for inoculation. When the ponds develop a true greenish colour, the inlets and outlets are opened to accept the design loads.

Anaerobic ponds

Anaerobic ponds will operate most effectively at the start if some well-digested sludge is placed in them. This sludge will provide the necessary seed organisms and an initial buffering capacity. However, if it is not possible to add digested sludge as seed, it will be necessary to develop an environment conducive to methane fermentation. Assuming that the anaerobic pond will eventually have a depth of 2–3 m, settleable solids will be subjected to an anaerobic environment. After a certain time, acid fermentation is followed by the desired alkaline fermentation phase. Two things can be done to assist in odour control during the acid phase, while the pond is undergoing adaptation to methane fermentation: (*a*) under certain conditions, particularly where some industrial wastes are included, lime may be added to bring the pH up to about 6.5–7.0; and (*b*) some oxygen-rich waters from the facultative or maturation ponds can be distributed over the top of the anaerobic pond.

Where domestic waste-waters contain a high amount of solids, such as night-soil, it may be necessary to add some water initially. This will facilitate better digestion, particularly if the evaporation rates are high.

Annex 1

LIVING ORGANISMS AND OTHER INFLUENCING FACTORS IN WASTE STABILIZATION PONDS

Waste stabilization ponds are the habitat of an enormous variety of living things. All the plants, animals, and fungi found in these ponds reproduce their kind to the extent that food is available. They are a heterogeneous population, competing for the same food or living parasitically off one another. The net result is that waste material is partially converted to cell substance; however, this conversion cannot take place without energy, and to provide this energy the biochemical system must break down an amount of food substrate over and above that required for cell reproduction. In this way the micro-organisms take high-energy materials such as partially digested domestic waste products and produce a low-energy end product such as carbon dioxide.

To examine the biological population of a pond minutely is impossible; attention is therefore concentrated on the requirements of the bacterial and algal communities that may develop in various types of waste stabilization pond, the factors influencing the growth rate of the organisms, and the inter-community relationships that cause a pond system to function.

Classification

The classification of living things is exceedingly difficult, and frequently confusing to the uninitiated, especially as the same organism may be classified differently by scientists in different disciplines. Classifications may be extended to considerable detail. For example, if a plant has bacterial characteristics its usual classification will be based on form, stain reaction, and biochemical character. Obviously, additional characterization tests will provide further differentiation. However, international congresses have established a system of biological nomenclature. It is desirable that the designers and supervisors of biological waste-water treatment plants should have at least a superficial understanding of taxonomy, the process of classifying and naming biological organisms. The full classification of an organism contains a large number of elements—kingdom, class, order, family, etc.—but the conventional scientific name consists of only two parts—the genus and the species. These names are printed in italics or underlined. The generic name is always placed first and capitalized, e.g., *Phacus pyrum.*

Bacteria, which are members of the class Schizomycetes, can be divided into two groups according to the type of substrate used for food. The larger of these groups comprises the heterotrophic bacteria, which require organic material as a basis for metabolism. The second group are called autotrophic bacteria, and can synthesize basic food from inorganic substrates. This latter group may be responsive to light action (photosynthetic) or chemical action (chemosynthetic).

Over 15 000 species of algae have been catalogued, but the species that are of importance to the waste-water engineer are relatively few (Palmer & Tarzwell, 1955) and can be broken down into four major groups (Table 26):

TABLE 26

COMPARISON OF FOUR MAJOR GROUPS OF ALGAE *

Characteristic	Group of algae			
	Blue-green	Pigmented flagellates	Green	Diatoms
Colour	Blue-green (brown)	Green or brown	Green	Brown (light green)
Location of pigment	Throughout cell	In plastids [a]	In plastids [a]	In plastids [a]
Starch	Absent	Present or absent	Present	Absent
Slimy coating	Present	Absent in most	Absent in most	Absent in most
Nucleus	Absent	Present	Present	Present
Flagellum	Absent	Present	Absent	Absent
Cell wall	Inseparable from slimy coating	Thin or absent	Semi-rigid, smooth or with spines	Very rigid with regular markings
"Eye" spot	Absent	Present	Absent	Absent

* After Jackson (1965).

[a] A *plastid* is a morphological unit consisting of a single cell; also, any of certain small specialized masses of protoplasm (e.g., chloroplasts, chromoplasts) occurring in some cells.

(1) *Blue-green algae*, which are simple forms lacking a nuclear membrane. The pigments are dissolved in the cellular fluid. These algae are frequently associated with odours and other nuisances.

(2) Non-motile green algae, which may be unicellular or multicellular. The unicellular organisms are usually small, whereas the multicellular ones may form great floating mats. The green forms are most predominant in aerobic and facultative waste stabilization ponds.

(3) Pigmented flagellates, which possess nuclei, chloroplasts, flagellae, and a red eye spot. This group may include various motile green forms.

(4) Diatoms, which are usually golden brown in colour, may be motile, and have a characteristic silicon dioxide (SiO_2) structural framework. These organisms are very common, and occasionally colonial forms resembling hair-like filaments are seen.

Animal organisms, which lack chlorophyll, ingest and digest other organisms. Typical of the animals found in waste treatment ponds are protozoa, rotifers, crustaceans, and annelids. As a general rule they are scavengers that tend to clear the excess bacteria and sometimes algae.

The protozoa are single-celled, motile organisms. In general, they metabolize solid food and have a more complex digestive system than the microscopic plants. The literature on the classification of protozoa is sometimes confusing, particularly as to whether the motile, single-cell, photosynthetic organisms are algae or protozoa. The protozoologist claims them as protozoa, while the phycologist claims them as algae. Neither all protozoa nor all algae can be classified simply as plants or animals because there is too much overlap in habits, structures, and physiology. Examples of such "intermediate" forms are *Chlamydomonas*, *Volvox*, and *Eudorina* (Needham & Needham, 1964).

Rotifers are found only in waters of low organic content, for they require an environment containing several mg/litre of dissolved oxygen. The diet of a rotifer consists primarily of bacteria and algae, but may also include small organic particles.

Another group of animals significant to waste-water treatment are the microscopic crustaceans, notably *Daphnia* and *Cyclops*. Like the rotifers, these organisms are aerobes. Since they feed on bacteria and algae, it is possible that, if cultivated in the last of a series of waste stabilization ponds, they would help to reduce the algal content of the effluent.

Some other animals found in waste stabilization ponds are associated more closely with the bottom muds and biological slimes. Various types of worms and larvae are found in the different types of ponds. Slender cylindrical nematode worms, which can be found in the sludges and slimes associated with an anaerobic environment, digest solid organic material not readily broken down by other organisms. *Tubifex* worms are more common in the organically rich sediments of waste stabilization ponds. Since *Tubifex* worms and midge larvae are both red, errors in identification may occur. The midge larvae are found in slightly polluted waters, but never in anaerobic pretreatment ponds.

Pond Ecology and Population Dynamics

A waste stabilization pond represents one form of biological community. As an environmental entity it acts as a transitional stage between polluted waste-water and a clean waterway. Obviously there must be some basic principles controlling the adaptation of certain predominant biological forms to alterations in the food supply.

In every type of pond that supports life there is an orderly process of community change (ecological succession) until a relatively stable community is evolved. The pond ecosystem (the community and its non-living environment) consists of abiotic substances, producer organisms, consumer organisms, and decomposer organisms. The abiotic substances are necessities of life such as water, carbon, oxygen, calcium, nitrogen, phosphorus, and amino and humic acids. In waste stabilization ponds large amounts of nutrients are available, whereas in a natural, unpolluted lake the nutrients must be released from the solid rock or other materials in the environment. The rate of release of nutrients is perhaps the most important regulating factor in an ecosystem, because competition for food is central to population dynamics. For illustrative purposes, the subject of population dynamics is discussed below in terms of the pond community as a whole, bacteria, protozoa, and algae, although it will be realized that the ecological system covers many organisms other than these.

The pond community

The size of the biological organisms present in a community controls the biomass (total dry weight of organisms present at one time) that can be supported by a steady supply of food. For example, on a weight basis the metabolic rate of algae and bacteria is much greater than that of larger plants such as trees. This can be illustrated more clearly by a rate of growth coefficient. Daily growth rates (K_g/day) for individual types of organism may vary widely, e.g., algae 0.20–2.0, protozoa 1–4, bacteria 2–60 (Bogan, 1961). Equation 39 states that the number of organisms present at a given time in the future will be dependent on the initial concentration and the growth rate.

$$N'_t = N'_o e^{(K_g t)} \tag{39}$$

where:

N'_t = number of organisms after time t
K_g = growth rate coefficient (per day)

The rate of metabolism of a community of organisms is important for the biological treatment of waste-waters. Normally this rate is estimated by measuring the oxygen consumption (respiration) or oxygen production (photosynthesis) or both, for which each species has its own basic requirements.

Organisms have a minimum and maximum limit of tolerance for environmental factors, and some have a wide range of tolerance for one factor and a narrow range for another. Any factor that approaches or exceeds the limits of tolerance will limit the distribution of an organism or group of organisms (Odum & Odum, 1959). This principle of limiting factors is particularly important when the effects of pollution on aquatic life are considered. The basis for the control or regulation of biological conditions can be stated in terms of Leibig's and Shelford's laws:

Leibig's Law of the Minimum. The determining factor in the limitation of the distribution of a species is that factor which is present in minimum amount. Such factors may include light, nutrients, temperature, time relationships, etc.

Shelford's Law of Tolerance. Those organisms that have wide ranges of tolerance for all factors are likely to be the most widely distributed.

Bacteria

The bacteria are the smallest organisms of interest in biological waste-water treatment; consequently, their metabolic rate is high, and under optimum environmental conditions they will invariably predominate over the true fungi and protozoa. The bacteria as a group generally have the same relative size, so that one bacterial form does not have a surface area-to-mass advantage over another. Their major competitive asset consists in a unique metabolic ability. For example, *Pseudomonas* is widely distributed in waste-water treatment plants because it can adapt itself to a variety of environmental conditions and organic substrates. Similarly, *Alcaligenes* and *Flavobacterium* will predominate whenever the protein content is relatively high, as may be the case with domestic waste-water or with waste-waters that contain the cellular releases of dead bacteria.

Specific foods or organic substrates will stimulate certain bacteria to multiply. Similarly, the residual products left from this selective growth of one group of bacteria will most certainly cause other groups to multiply. For example, if a particular waste-water containing a wide variety of seed organisms is placed in a bottle, a heterogeneous population will multiply. When the food supplies are depleted, some of these organisms will die and decompose. When the cell begins to deteriorate, proteins and other cellular products are released, and this new food source then gives rise to a secondary predominance. The cycle continues until the high energy-yielding foodstuffs have been significantly reduced.

Protozoa

Whereas specific organic food supplies stimulate certain bacteria, both bacteria and algae may stimulate the growth of protozoa. As a general rule, the protozoa follow a more definite pattern of succession (the replacement of one predominant species by another) than bacteria.

Generally, the Mastigophora flagellates prefer freshly polluted waters. The phytoflagellates have a difficult time competing for soluble substrates with bacteria, but the zooflagellates fare better because the bacteria are their food supply. The free-swimming Ciliata, however, are more efficient in collecting bacteria than the zooflagellates, while the stalked Ciliata use less energy than the swimmers and can survive on a smaller bacteria supply. Finally, the rotifers along with other higher animals scavenge the dead remains of bacteria, solid organic debris, and other non-soluble materials.

Algae

In general the distribution and the variations in abundance of phytoplankton in waterways are not well understood (Fogg, 1963). The seasonal succession of algae in waters and on the surface of the bottom sediment is a question that has long attracted the attention of scientists. Winter, spring, summer and autumnal annuals have been observed in nature. There are also perennials, whose vegetative cycle may be continuous from year to year, and ephemerals which may appear in numbers for a short time at any season.

It is recognized that changes in temperature, vitamins, dissolved salts, light intensity, dissolved gases, trace amounts of selected nutrients, geometry of the container, etc., have a marked effect upon the time at which algal blooms occur. However, the interaction of these various factors is extremely complex and not well understood. The production of blooms occurs as a result of a rapid overgrowth of one algal species or a succession of different algal genera.

The Role of Algae in Waste Stabilization Ponds

The life processes of algae have a powerful influence on the biology and chemistry of waste stabilization ponds and of the waterways that receive the algae-laden effluents. In aerobic or facultative ponds, photosynthesis is necessary to maintain a detectable dissolved oxygen content. The normal transfer of oxygen across the surface either by diffusion or by natural mixing is inadequate. In addition the algal systems consume carbon dioxide, thereby elevating the pH during daylight hours. Algae use ammonia, and so eliminate an oxygen demand normally inherent in the bacterial nitrification of ammonia to nitrite and nitrate. They are also involved in the reduction of other plant nutrients. Photosynthesis and respiration predominate alternately, and this results in conservation of energy. Finally, the aerobic bacteria rely upon dissolved oxygen, which is supplied in part by algae and surface reaeration.

Algae may vary in size from organisms measuring a few microns to long, segmented plants. Some marine algae grow to a length of 100 m or more.

As a rule the small, unicellular green algae are the most significant in maintaining a desired oxygen level in waste stabilization ponds.

Fig. 44 shows some algal forms that can be found in polluted waters. Typical of the green algae in waste stabilization ponds are *Chlorella*, *Scenedesmus*, *Chlamydomonas*, and *Euglena*. Common blue-green algae are *Oscillatoria* and *Anabaena*. In the operation of a pond *Chlamydomonas* and *Euglena* are frequently the first phytoplankton to appear, and tend to predominate during cooler weather. *Euglena* shows a high degree of adaptability to various pond conditions and may be present during all seasons and under most climatological conditions. Probably next in adaptability are *Chlamydomonas*, *Micractinium*, *Ankistrodesmus*, *Scenedesmus*, and *Chlorella*. At any season of the year, odours may occur if patches of benthic algae, such as *Phormidium*, become detached from the bottom and accumulate at the surface.

Some of the photosynthetic organisms characteristic of waste stabilization ponds are listed below (Clare et al., 1961):

Green algae

Volvocales: *Chlamydomonas*, *Chlorogonium*, *Pascheriella*, *Pandorina*, *Carteria*

Chlorococcales: *Chlorella*, *Golenkinia*, *Micractinium*, *Ankistrodesmus*, *Scenedesmus*, *Actinastrum*, *Coelastrum*, *Oocystis*, *Tetraedron*

Euglenophyta: *Euglena*, *Phacus*

Diatoms

Nitzschia

Blue-green algae

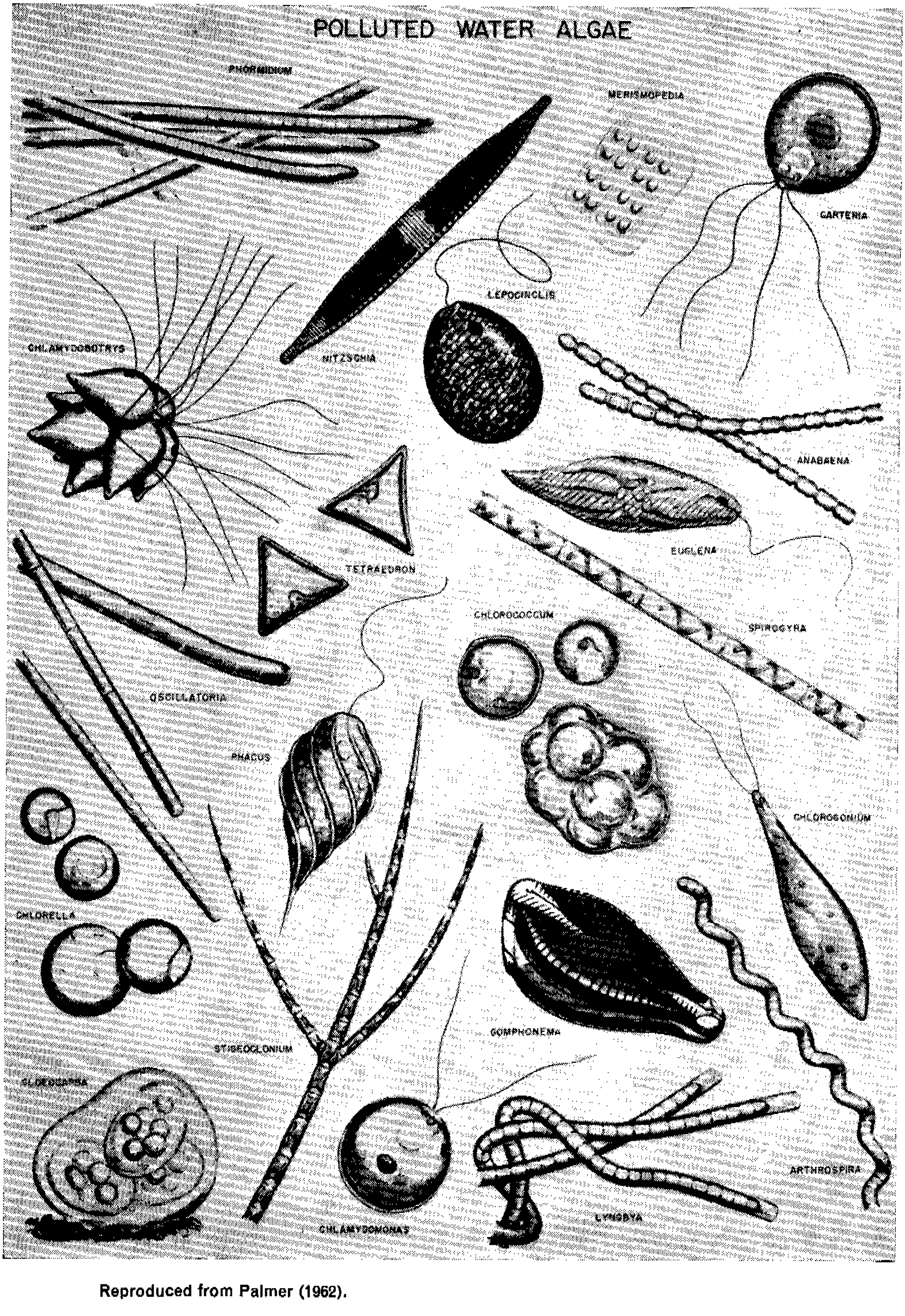
Oscillatoria, *Anabaena*

Unicellular algae, in particular, react rapidly to changes in the environment. The gross productivity (photosynthesis) and production rates beneath a given surface area of pond depend upon the density, species composition, age, and physiological stage of algae, as well as a variety of environmental factors. It is therefore important to examine critically those environmental factors which affect the growth rate, i.e., illumination (incident light and light attenuation in water), temperature, and nutrient conditions.

Illumination

Fortunately, it is not necessary to keep a pond aerobic for its entire depth. It is difficult to ensure full light penetration, because the utilization of light-energy in waste stabilization ponds is complicated by many factors. The magnitude of diurnal oxygenation is probably influenced more by

FIG. 44. ALGAE FOUND IN POLLUTED WATERS



Reproduced from Palmer (1962).

variations in algal populations than by changes in light intensity. The algal-bacterial masses are usually not homogeneous, so that design calculations based on light penetration alone may be misleading.

Assuming homogeneous mixing, there is a pond depth at which the light intensity just equals the light saturation intensity for a particular alga (Bush, 1953). At low light intensities the efficiency of light utilization by algae is almost linear and photosynthesis is primarily controlled by light intensity. At higher intensities there is a saturation plateau where increased light does not increase photosynthesis. Thus, in accordance with established rules, light that penetrates to greater depths is utilized with increasing efficiency. Equations 40 and 41 depict the light transmission and light saturation relationships.

$$I = I_o e^{-c'N_a d} \tag{40}$$

$$F_a = \frac{I_s}{I_o}(\ln\frac{I_o}{I_s} + 1) \tag{41}$$

where

I = light intensity after passage through media (erg/cm^2/s)
I_o = original light intensity (erg/cm^2/s)
c' = light absorption coefficient (cm^2/mg)
N_a = concentration of algal cells (mg/cm^3)
d = pond depth (cm)
F_a = fraction of light that is utilized by the algae
I_s = light saturation intensity (erg/cm^2/s)

Some probable average values of visible solar energy on a horizontal surface are given in Table 27. It is noteworthy that the effect of latitude is more marked in winter than in summer.

The relationship between the minimum light utilization level and the distance of penetration can be demonstrated. For example, in the presence of a homogeneous culture of *Chlorella pyrenoidosa*, with a photosynthesis-respiration compensation level for growth (the level where photosynthesis essentially stops) of 24 ft-c (1 000 erg/cm^2/s), and a saturation point at about 600 ft-c (25 000 erg/cm^2/s), the depth of the pond must be limited to about 35 cm (Phillips & Myers, 1954). By contrast, the optimum saturation level for *Euglena* may be in the neighbourhood of 2 000 foot-candles (83 000 erg/cm^2/s).

The heat of combustion for sewage-grown algae is about 6 cal/mg. Thus, by calculating the visible solar energy, estimating the fraction of solar energy converted to cell material, measuring the heat combustion of the cells, and establishing a detention period, the overall efficiency of photosynthetic conversion of solar energy to algal energy can be determined. Usual efficiencies range from 2 % to 9 %, with 5 % a common figure.

TABLE 27

PROBABLE VALUES OF VISIBLE SOLAR ENERGY AS A FUNCTION OF LATITUDE AND MONTH IN THE NORTHERN HEMISPHERE *

Latitude	Probable maximum and minimum values of light intensity (I_{Lmax} and I_{Lmin}) in cal per cm² per day											
	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
0°	255 210	266 219	271 206	266 188	249 182	236 103	238 137	252 167	269 207	265 203	256 202	253 195
10°	223 179	244 184	264 193	271 183	270 192	262 129	265 158	266 176	266 196	248 181	228 176	225 162
20°	183 134	213 140	246 168	271 170	284 194	284 148	282 172	272 177	252 176	224 150	190 138	182 120
30°	136 76	176 96	218 134	261 151	290 184	296 163	289 178	271 166	231 147	192 113	148 90	126 70
40°	80 30	130 53	181 95	241 125	286 162	298 173	288 172	258 147	203 112	152 72	95 42	66 24
50°	28 10	70 19	141 58	210 97	271 144	297 176	280 155	236 125	166 73	100 40	40 15	26 7
60°	7 2	32 4	107 33	176 79	249 132	294 174	268 144	205 100	126 38	43 26	10 3	5 1

* Reproduced, in modified form, from Oswald (1964b), p. 8, by permission of the publishers.

Average value of $I_L = I_{Lmin} + s' (I_{Lmax} - I_{Lmin})$, where $s' = \frac{\text{total hours of sunshine}}{\text{total possible hours of sunshine}}$.

Yield of algal cell material, in kg/ha per day = $Y_a = 0.15\,FI_L$.

Yield of oxygen, in kg O_2/ha per day = $0.25\,FI_L$.

It has been suggested that organic loadings to waste stabilization ponds might even be set on the basis of 1.12 kg BOD_5 per ha per day for every 2 langleys[1] of visible solar energy (Hopkins & Hopkins, 1964). As a rough guide, therefore, the loading in terms of kg BOD per ha per day should not exceed one-half the minimum daily langley level for the month with least solar radiation, excluding periods of ice cover.

Another empirical method for evaluating the light conversion on the basis of BOD and temperature is shown in equations 42 and 43 (Oswald, 1964a).

$$F = 1.3\,(\ln L_o - 3.5) \qquad (42)$$

$$F = 3.48\,(\ln T - 1.35) \qquad (43)$$

where

F = efficiency of light energy (insolation) utilization (%)

Another empirically-derived equation showing the relationship between the heat of combustion in algal cells and the stabilization of organic matter is given below (Arceivala, 1964).

$$H = \frac{B}{7.89} + 0.4 \qquad (44)$$

where

H = heat of combustion of algal cells (cal/mg)

The degree of reduction, B, is expressed in terms of carbon, hydrogen, and oxygen, i.e.,

$$B = \frac{100\,(2.66\,P_C + 7.94\,P_H\,P_O)}{398.9}$$

where P_C, P_H, and P_O are the percentages of carbon, hydrogen, and oxygen respectively.

Temperature

Temperature is highly important in the design of waste stabilization ponds. It affects photosynthetic oxygen production as well as other biological reactions. Optimum oxygen production (for some species) is obtained at about 20°C, and limiting lower and upper values appear to be about 4°C and 37°C respectively, although it is known that higher temperatures can be tolerated and some algae have been observed growing quite well under a cover of clear ice.

Biological reactions, within limits, tend to follow well-known chemical rate relationships. In a simplified system, the time-temperature relationship can be conveniently expressed as shown in Equation 45. As the temperature

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[1] A langley is a unit of solar radiation equivalent to 1 cal/cm^2 of irradiated surface.

increases up to a given limit, the total retention time needed to achieve the same BOD reduction decreases. Bottom conditions are normally improved by increases in temperature, but conditions at the top may not be improved because undesirable flora frequently develop at water temperatures exceeding 37°C. Coefficients describing the removal rates can also be used effectively to determine either the temperature coefficient or the associated detention times.

$$\frac{R_T}{R_o} = \theta^{(T_o - T)} = \frac{K'_{T_o}}{K'_T} \tag{45}$$

where

R_o = detention time for some specified temperature, T_o, for which θ or the rate coefficients are known (days)

θ = temperature reaction coefficient

K'_T = removal rate coefficient for temperature T (per day)

K'_{T_0} = removal rate coefficient for temperature T_o (per day)

Some of the beneficial green algae do not appear to function efficiently at temperatures higher than about 37°C. Furthermore, it is likely that ponds with higher water temperatures will be a little more sensitive to shock or sudden increases in the quantity of organic material and will be subject to less efficient BOD removal rates. It must also be recognized that bacterial activity becomes more intense at higher temperatures, whereby the dissolved oxygen is used at a higher rate.

Nutrients

Most species of algae use only free carbon dioxide in photosynthesis (Neilsen, 1955), but there is some indication that a few algae use bicarbonate ion (Osterlind, 1948). Even with the carbon dioxide concentration of 0.03 % usually found in air, an optimum rate of photosynthesis can be maintained (Davis et al., 1953). For every six moles of carbon dioxide reduced, about six moles of oxygen and one mole of sugar are produced.

Domestic waste-waters contain all the nutrients required to maintain a bacterial and algal community. Of course, in the design of a pond the environmental factors such as temperature, light, and concentration of foodstuffs and biota must also be considered. Industrial waste-waters, on the other hand, may not contain sufficient nutrients; frequently they are deficient in nitrogen or phosphorus, or both. The chemical requirements of various micro-organisms are shown in Table 28.

When the food supply is sufficient for optimum bacterial growth it is normally adequate for supporting the algal population as well. The required BOD-phosphorus-nitrogen ratio is about 100 : 5 : 1. In a pond where seepage is minimal and detention periods are long, however, there may be considerable reuse of both nitrogen and phosphorus by bacteria, algae, and other aquatic organisms.

TABLE 28

CHEMICAL REQUIREMENTS OF CERTAIN MICRO-ORGANISMS *

Element	Algae	Fungi	Bacteria	Protozoa
C, H, O	+	+	+	+
N, P, S	+	+	+	+
Mg	+	+	?	+
Ca	+	?	?	+
Co	+	?	?	?
Cu	?	+	?	?
Fe	+	+	+	+
Me	+	+	+	+
K	+	?	?	+
Zn	?	+	?	?
B	?	?	+	0
Ga	0	?	0	0
Mo	+	+	+	0
Si	?	0	0	0
Na	?	0	+	?
Va	?	0	0	0
Sr	?	0	0	0
Rb	?			

* After Krauss (1961), p. 45.

\+ Demonstrated to be essential.

? Uncertain whether essential, although a requirement has been demonstrated in some species.

Micronutrients are also of importance for growth and may, along with other factors, set up conditions that will support algal blooms, i.e., excessive growth of one species. Opportunist algae, such as *Chlamydomonas*, may undergo massive and useless proliferation. Other algae frequently associated with blooms are the Chlorococcales such as *Chlorella*, *Scenedesmus*, and *Golenkinia*, or the Volvocales such as *Chlorogonium* and *Chlamydomonas*. Algae of the order Euglenoidina are also triggered periodically into excessive growth. No one knows for sure what provokes a bloom or when a bloom will occur, but undoubtedly algal blooms can contribute to the production of unpleasant odours.

Since these blooms seem to depend mainly upon the availability of inorganic substances, metal solubilizers, and vitamins, it is reasonable to assume that imbalances in the ecological system may occur, particularly in new ponds where the evaporation and seepage losses are high. It is unlikely that there will be any deficiency in vitamins in domestic wastewaters, but there may be differences in concentration. All algal forms cannot use every nitrogen and phosphorus compound found in wastewaters, and this may lead to certain imbalances; some organic nitrogen combinations and some polyphosphates, for example, are not suitable nutrients for algae unless they are first acted on by bacteria.

Fate of algae

Living algae are not normally subject to bacterial attack, but little is known about phages or related viruses that may infect algae. Microfungi are known to be algal parasites. Like all organic materials, dead algae are subject to biological degradation.

In ponds receiving relatively small loads, the algae may be consumed by plankton such as *Daphnia* or *Cyclops*, or, by themselves or in conjunction with some debris, settle slowly to the bottom. Once the cells become part of the benthos, the material may be consumed by *Chironomus* larvae or undergo bacterial decomposition. The *Chironomus* larvae flourish in rich organic sediments when a small amount of oxygen is available.

Unfortunately, because of low settling velocities, unicellular algae do not normally settle to any appreciable extent, and unless scavenged from water they are discharged into receiving streams with the pond effluent.

Where anaerobiosis exists, the algal debris can undergo fermentation, but this is a slow process. The best digestion of algal cells is obtained at 50°C, but even at this temperature the retention period may be as long as 30 days. For temperatures of less than 35°C the rate of cell (volatile matter) destruction is low (Golueke et al., 1957), indicating a slow biological degradation rate.

Dead algal cells exert higher BOD values than live cells (Fitzgerald, 1961). The BOD of algal cells killed by freeze-drying is reported to be nearly as great as that of cells killed by autoclaving, but the presence of *Chlorella* in concentrations as high as 4×10^6 cells per ml in a BOD bottle does not appreciably affect the 5-day BOD.

Sulfur Transformations in Waste Stabilization Ponds

Various types of sulfur transformation frequently occur in waste stabilization ponds. Moreover, the amount of SO_4^{2-} and S^{2-} ions present may influence the design of facultative ponds. Two types of micro-organism play an important role in these sulfur transformations: (1) reducing bacteria that utilize sulfate as an inorganic hydrogen acceptor and produce large amounts of sulfide, and (2) bacteria that are able to oxidize sulfur compounds.

Desulfovibrio is the most characteristic genus of the reducing bacteria. According to ZoBell and Rittenberg (1948), the optimum oxidation-reduction potential (ORP) for growth of sulfate-reducing bacteria is between —100 and —300 mV at pH 7.0. No growth has been observed at potentials of more than +27 mV (Aleshina, 1938). Studies of sulfate-reducing bacteria in synthetic substrate and in a polluted estuary show that sulfate reduction is inhibited by the presence of significant amounts (about 0.16 mg/litre) of dissolved oxygen (Wheatland, 1954). These environmental factors, DO concentration and ORP, can limit the reduction of sulfate by bacteria to the lower layers of the pond. The rate of sulfate reduction is also dependent upon water temperature; below 15°C the rate drops sharply (Wheatland, 1954); above this temperature the rate of sulfide formation approximately doubles for each increase of 10°C.

Two markedly different types of micro-organism, which can be characterized according to the hydrogen acceptor used, are able to oxidize sulfur compounds. The first group is formed by the colourless sulfur bacteria, which use molecular oxygen as a hydrogen acceptor. These bacteria are strict aerobes that require sulfide. Since sulfide is promptly oxidized by dissolved molecular oxygen, very special environmental situations must exist for this group of bacteria to grow. Examples of such situations are the surfaces of sewers and certain sulfur springs (Lackey et al., 1965). In a waste stabilization pond, the best environmental conditions for the growth of colourless sulfur bacteria would be in the top layer of the pond, but such bacteria are not found in stabilization ponds to any significant extent.

The second group of micro-organisms able to oxidize sulfur compounds comprises the photosynthetic sulfur bacteria. These bacteria are autotrophic anaerobes that require both light and sulfide for growth and use CO_2 as the hydrogen acceptor. Included in this group are the green Chlorobacteriaceae and the purple Thiorhodaceae. The oxidation of sulfide takes place in two main steps:

$$(1) \qquad CO_2 + 2H_2S \xrightarrow{\text{light}} (CH_2\,O) + H_2O + 2S$$

$$(2) \qquad 3CO_2 + 2S + 5H_2O \xrightarrow{\text{light}} 3(CH_2O) + 2H_2SO_4$$

An important feature of this photosynthetic reaction is that no molecular oxygen is formed.

Large numbers of photosynthetic sulfur bacteria are often found in waste stabilization ponds, particularly during the summer and autumn months when sulfate or sulfide concentrations are high, usually in excess of 500 mg/litre. These bacteria impart to the pond a characteristic red or brown colour, and are usually accompanied by a reduction in hydrogen sulfide odour (Green, 1966). They do not appreciably reduce the BOD of the waste and may even tend to increase it. The photosynthetic sulfur bacteria first appear in a pond at mid-depth, where anaerobic conditions exist and sulfides are present. (The greater depth presents no problem since these bacteria can use longer wavelength light than algae.) The cell concentration tends to increase, and the cells rise to the upper layers, where with better illumination the bacteria create turbid conditions that help to crowd out algae.

Annex 2

LIST OF SYMBOLS AND ABBREVIATIONS

A	Area of throat (m^2) (Venturi meter)
a'	BOD removed and used to provide energy for growth, as a fraction of BOD_u or COD
a''	Coefficient, based on a'
B	Degree of reduction
b'	Endogenous respiration rate per day, as fraction of BOD or COD
C	Level of dissolved oxygen in pond (mg/litre)
c	Non-uniformity coefficient (Equation 27, page 103)
c'	Light absorption coefficient (cm^2/mg)
c_o	Coefficient of discharge (Equation 38, page 120)
C_s	O_2 saturation level of distilled water at 20°C (mg/litre)
C_{sw}	O_2 saturation level in the pond at temperature T (mg/litre)
d	Depth (m)
$\mathbf{d}$	*See* diagram, p. 113
d_H	Differential head (m)
d_{max}	Liquid depth in channel at maximum flow
F	Light conversion efficiency (%)
F_A	Correction factor for altitudes above 1200 m
F_a	Fraction of light used by algae
g	Acceleration of gravity
H	Heat of combustion of cells (cal/mg)
$\mathbf{H_a}$	*See* diagram, p. 113
$\mathbf{H_b}$	
$\mathbf{H_c}$	
$\mathbf{H_m}$	
I	Light intensity after passage through media (erg/cm^2/s)

I_L	Light intensity (cal/cm^2/day)
I_o	Original light intensity (erg/cm^2/s)
I_s	Light saturation intensity (erg/cm^2/s)
k	Reaction constant, $\log_{10}$ (breakdown/day)
K'	Bacterial disappearance (removal) rate
k_1	Reaction constant, $\log_e$ (breadkdown/day)
k'	Rate constant, $\log_{10}$ (bacterial disappearance/day)
k'_1	Rate constant, $\log_e$ (bacterial disappearance/day)
K_g	Growth rate coefficient (per day)
K_n	Design coefficient
$K_{s(T)}$	Digestion rate (rate of evolution from the sludge layer)
K_T	Breakdown rate at temperature T
K'_T	Removal rate coefficient for temperature T (per day)
K'_{T_0}	Removal rate coefficient for temperature T_0 (per day)
L_a	Initial (influent) BOD$_u$ (mg/litre)
L_o	Influent BOD$_5$ (mg/litre)
L_p	Effluent BOD$_5$ (mg/litre)
L_r	BOD removed (kg/day)
L_s	BOD satisfied after t days (mg/litre)
L_t	BOD not yet developed after t days (mg/litre)
N	Number of persons contributing waste
n	Exponent, to be determined by experimentation (Equation 18, page 76)
N_a	Concentration of algal cells (mg/cm^3)
N'_o	Initial population (organisms, bacteria)
N_p	Number of ponds
N'_R	$N'_o - N'_t$
N'_t	Population at time t (organisms, bacteria)
O	Oxygen production (kg/ha/day)
O'	Oxygen production (mg/cm^3)
O_d	Oxygen required (kg/day)
O_m	Manufacturer's rating of aerator
O_n	Oxygen required (mg/litre)

O_s	Oxygen supplied by mechanical aeration (kg/hph)
p	Fraction of bacterial population (Equation 26, page 103)
Q	Flow (m^3/s)
q	*Per caput* waste contribution (litres/day)
Q_a	$\frac{Q_{max}}{Q_{min}}$
Q_{max}	Maximum flow (m^3/s)
Q_{min}	Minimum flow (m^3/s)
Q_r	$\frac{Q_a^{\frac{1}{3}} - 1}{Q_a - 1}$
R	Detention time (days)
r	Ratio of throat diameter to pipe diameter (Venturi meter)
R_1, R_2, ...	Detention time in pond 1, 2, ... (days)
R_o	Detention time for specified temperature, T_o, for which θ or rate coefficients are known (days)
R_T	Detention time at temperature T
S	*See* diagram, p. 113
s	Constant for a particular meter (Equation 38a, page 120)
s'	$\frac{\text{Total sunshine}}{\text{Total possible sunshine}}$ (hours)
T	Temperature, pond operating temperature (°C)
t	Time
T_m	Average water temperature of coldest month (°C)
V	Pond volume (m^3)
v	Velocity of flow
W	*See* diagram, p. 113
X_t	Volatile suspended solids in mixed liquor (mg/litre)
Y_a	Yield of algal cells (kg algae/ha/day)
α	$\frac{\text{Overall transfer coefficient of the waste-water}}{\text{Overall transfer coefficient of the tap water}}$
θ	Temperature reaction coefficient

Annex 3

CONVERSION FACTORS: METRIC, BRITISH, AND US UNITS

Length

1 millimetre (mm) = 0.0394 in
10 mm = 1 centimetre (cm) = 0.394 in
100 cm = 1 metre (m) = 1.093 yd = 3.28 ft = 39.37 in
1000 m = 1 kilometre (km) = 0.6214 mile = 1093 yd
1 inch (in) = 2.54 cm
12 in = 1 foot (ft) = 0.3048 m = 30.48 cm

Area

1 square centimetre (cm^2) = 0.155 in^2
10 000 cm^2 = 1 square metre (m^2) = 1.196 yd^2 = 10.76 ft^2
10 000 m^2 = 1 hectare (ha) = 2.47 acres
4 840 yd^2 = 1 acre = 0.405 ha = 4047 m^2

Volume

1 cubic centimetre (cm^3) = 0.06102 in^3
1×10^6 cm^3 = 1 cubic metre (m^3) = 1.307 yd^3 = 35.32 ft^3

Liquid capacity

1000 ml = 1 litre = 0.26 USgal = 0.22 UKgal
1 US gallon (USgal) = 0.83 UKgal = 3.79 litres
1 UK gallon (UKgal) = 1.2 USgal = 4.55 litres

Weight

1000 mg = 1 gram (g) = 0.0352 oz
1000 g = 1 kilogram (kg) = 2.2 lb = 35.27 oz
1 ounce (oz) = 28.35 g
16 oz = 1 pound (lb) = 453.6 g

Power

1 horsepower (hp) = 0.7457 kW
1 kilowatt (kW) = 1.341 hp
1 horsepower hour (hph) = 2.685 megajoule (MJ)

Miscellaneous

1 lb BOD/acre per day = 1.12 kg BOD/ha per day

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