

**Design Manual for
Waste Stabilization Ponds
in India**



**Design
Manual for
WASTE
STABILIZATION
PONDS
in
India**

Duncan Mara

*Professor of Civil Engineering
University of Leeds
and Chairman
Lagoon Technology International
Leeds, England*

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Preface

Waste stabilization ponds are an extremely appropriate method of wastewater treatment in India, and I hope that this Manual will serve to promote modern pond design in the country. Of course design by itself is not enough: operation and maintenance are crucial, but fortunately with ponds this is simple and does not require skilled labour. Guidance is also given on pond monitoring and evaluation, and this can lead to improved design – there is no substitute for local data. Sometimes, because of more rigorous legislation or neglect, pond systems need upgrading or rehabilitation, and this is also discussed.

In many developing countries, and India is no exception, wastewater is generally too valuable to waste, and the reuse of pond effluents for crop irrigation or for fish culture is very important in the provision of high quality food. In arid zones, the use of wastewater storage and treatment reservoirs is advantageous as it permits the whole year's wastewater to be used for irrigation, thus enabling the irrigation of a much larger area and the consequent production of much more food.

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Duncan Mara
Leeds, England
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1

Introduction

1.1 THE NEED FOR WASTEWATER TREATMENT

Wastewater needs to be adequately treated prior to disposal or reuse in order to:

- (a) protect receiving waters from gross faecal contamination as they are often used as a source of untreated drinking water by downstream communities (or, in the case of coastal waters, used for shellfisheries);
- (b) protect receiving waters from deleterious oxygen depletion and ecological damage; and
- (c) produce microbiologically safe effluents for agricultural and aquacultural reuse (for example, crop irrigation and fishpond fertilisation).

As sewerage, both conventional and unconventional (the latter comprising simplified sewerage and settled sewerage (see Mara, 1996) which are more suitable for low-income communities), becomes more common in India, so too will the need for appropriate and sustainable wastewater treatment systems. Such systems need to be low cost, easy to operate and maintain, and very efficient in removing both organic matter (BOD) and the wide range of excreted pathogens present in wastewaters.

1.2 ADVANTAGES OF WASTE STABILIZATION PONDS

Waste stabilization ponds (WSP) are shallow man-made basins into which wastewater flows and from which, after a retention time of several days (rather than several hours in conventional

treatment processes), a well-treated effluent is discharged. WSP systems comprise a series of ponds – anaerobic, facultative and several maturation. The different functions and modes of operation of these three different types of pond are described in Section 3 of this Manual. The advantages of WSP systems, which can be summarised as *simplicity*, *low cost* and *high efficiency*, are as follows:

Simplicity

WSP are simple to construct: earthmoving is the principal activity; other civil works are minimal – preliminary treatment, inlets and outlets, pond embankment protection and, if necessary, pond lining (further details are given in Section 5). They are also simple to operate and maintain: routine tasks comprise cutting the embankment grass, removing scum and any floating vegetation from the pond surface, keeping the inlets and outlets clear, and repairing any damage to the embankments (further details are given in Section 6). Only unskilled, but carefully supervised, labour is needed for pond O&M.

Low cost

Because of their simplicity, WSP are much cheaper than other wastewater treatment processes. There is no need for expensive, electromechanical equipment (which requires regular skilled maintenance), nor for a high annual consumption of electrical energy. The latter point is well illustrated by the following data from the United States (where one third of all wastewater treatment plants are WSP systems) for a flow of 1 million US gallons per day (3780 m³/d) (Middlebrooks *et al.*, 1982):

Treatment process	Energy consumption (kWh/yr)
Activated sludge	10,000,000
Aerated lagoons	8,000,000
Biodiscs	1,200,000
Waste stabilization ponds	nil

Thus the energy costs of activated sludge systems and aerated lagoons are very high. In Chennai, for example, total O&M costs, including energy costs, at the 23 Mld activated sludge plant at Nesapakkam are Rs 0.17 per m³ of wastewater treated, equivalent to an annual cost of Rs 14 lakhs. With aerated lagoons it is not

uncommon for the aerators to be permanently switched off as the energy costs are so high. The result is that the aerated lagoon then functions as an anaerobic pond. Provided this is recognised and the resulting anaerobic pond is not overloaded and regularly desludged (see Section 6.4), BOD removal efficiency can be as high as in the aerated lagoon but without, of course, the associated energy costs of the latter. A good example of an aerated lagoon operating satisfactorily as an anaerobic pond is at the Villivakkam wastewater treatment plant in Chennai.

The cost advantages of WSP were analysed in detail by Arthur (1983) in a World Bank Technical Paper. Arthur compared four treatment processes – trickling filters, aerated lagoons, oxidation ditches and WSP, all designed to produce the same quality of final effluent. Summary details are given in Box 1 on pages 4–6. The most important conclusion from Arthur’s work is that WSP systems were the cheapest treatment process at land costs of US\$ 50,000-150,000 (1983 \$) per hectare, depending on the discount rate (opportunity cost of capital; range: 5-15 percent). These figures are much higher than most land costs likely to be encountered, and so land costs are unlikely to be a factor operating against the selection of WSP for wastewater treatment, although land availability may be. Arthur’s economic methodology, which included both capital and O&M costs, is strongly recommended for use at the feasibility stage of all wastewater treatment projects in which a choice between different treatment processes has to be made. This should include, if necessary, the extra cost of conveying the wastewater to an area of low-cost land.

Tripathi *et al.* (1996) compared the costs of waste stabilization ponds, aerated lagoons, oxidation ditches and activated sludge for the treatment of domestic wastewater in India. The economic methodology used was broadly similar in principle to that used by Arthur (1983), but the WSP design procedure adopted (solar radiation principle for facultative ponds; 5 days retention for maturation ponds) is not now generally recommended (see Section 4). Activated sludge systems were found to be the most expensive option and WSP were the least cost system, although as expected the cost of WSP was highly dependent on the cost of land.

High efficiency

BOD removals >90 percent are readily obtained in a series of well-designed ponds. The removal of suspended solids is less, due

BOX 1**WASTEWATER TREATMENT COSTS**

A recent World Bank report (Arthur, 1983) gives a detailed economic comparison of waste stabilization ponds, aerated lagoons, oxidation ditches and biological filters. The data for this cost comparison were taken from the city of Sana'a in the Yemen Arab Republic, but are equally applicable in principle to other countries. Certain assumptions were made, for example the use of maturation ponds to follow the aerated lagoon, and the chlorination of the oxidation ditch and biological filter effluents, in order that the four processes would have an effluent of similar bacteriological quality so that fish farming and effluent reuse for irrigation were feasible. The design is based on a population of 250,000; a per caput flow and BOD contribution of 120 litres/day and 40 g/day respectively; influent and required effluent faecal coliform concentrations of 2×10^7 and 1×10^4 per 100 ml, respectively; and a required effluent BOD₅ of 25 mg/litre. The calculated land area requirements and total net present cost of each system (assuming an opportunity cost of capital of 12 per cent and land values of US\$ 5/m²) are shown in the Table opposite. Waste stabilization ponds are clearly the cheapest option.

The cost of chlorination accounts for US\$0.22 million per year of the operational costs of the last two options.

Clearly the preferred solution is very sensitive to the price of land, and the above cost of US\$ 5 per m² represents a reasonable value of low-cost housing estates in developing countries.

If the cost of land is allowed to vary, then the net present cost of each process varies as shown in Figure 1.1, for a discount rate (opportunity cost of capital) of 12 percent. Ponds are the cheapest option up to a land cost of US\$7.8 per m², above which oxidation ditches become the cheapest. In fact for discount rates between 5 and 15 percent the choice is always between WSP and oxidation ditches: the other two processes are always more expensive. Figure 1.2 shows the variation with discount rate of the land cost below which WSP are cheapest – between US\$ 5 and 15 per m² (US\$ 50,000 and 150,000 per ha).

Table 1.1 Costs and land area requirements of waste stabilization ponds and other treatment processes

	Waste stabilization pond system	Aerated lagoon system	Oxidation ditch system	Conventional treatment (biofilters)
Costs (million US\$)				
Capital	5.68	6.98	4.80	7.77
Operational	0.21	1.28	1.49	0.86
Benefits (million US\$)				
Irrigation income	0.43	0.43	0.43	0.43
Pisciculture income	0.30	0.30	-	-
Net present cost (million US\$)	5.16	7.53	5.86	8.20
Land area (ha)	46	50	20	25

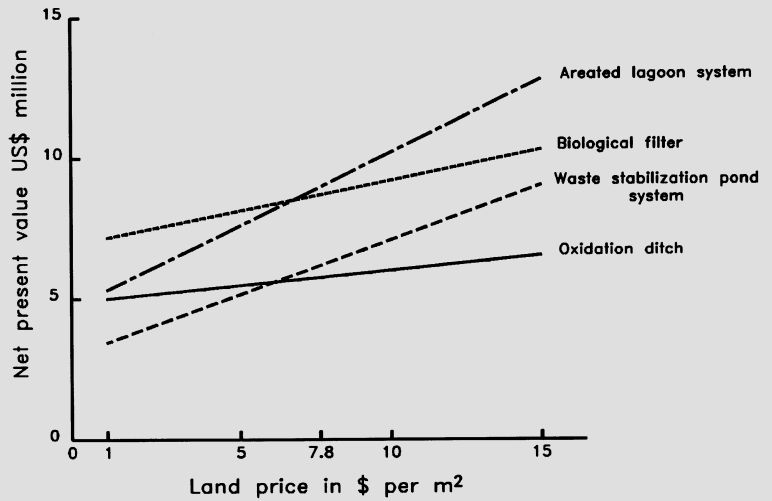
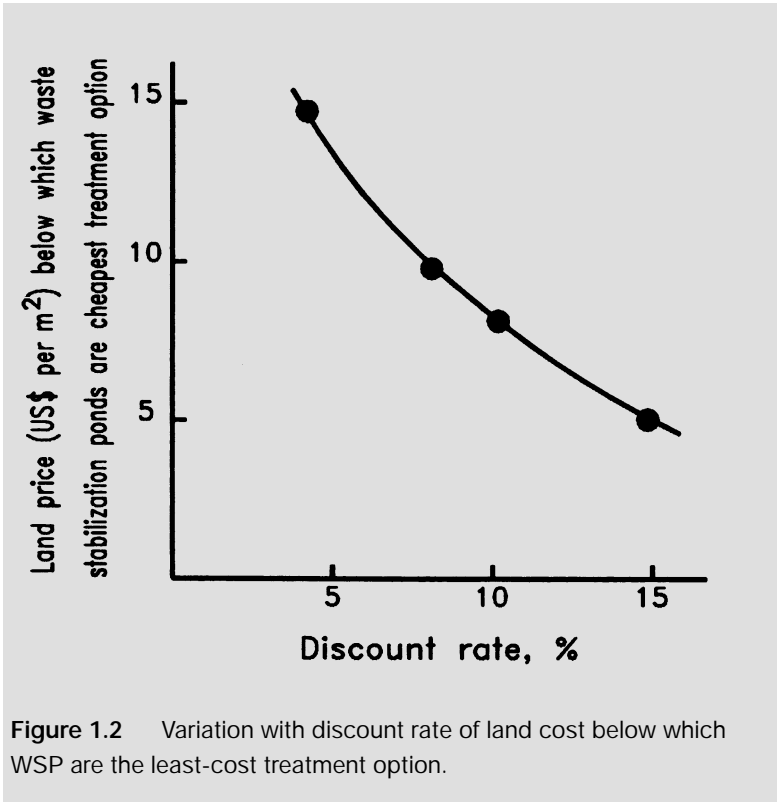


Figure 1.1 Variation in net present costs of the four treatment processes with land costs for a discount rate of 12 percent.



to the presence of algae in the final effluent (but, since algae are very different to the suspended solids in conventional secondary effluents, this is not cause for alarm: see Section 4.1). Total nitrogen removal is 70-90 percent, and total phosphorus removal 30-45 percent.

WSP are particularly efficient in removing excreted pathogens, whereas in contrast all other treatment processes are very inefficient in this, and require a tertiary treatment process such as chlorination (with all its inherent operational and environmental problems) to achieve the destruction of faecal bacteria. Activated sludge plants may, if operating very well, achieve a 99 percent removal of faecal coliform bacteria: this might, at first inspection, appear very impressive, but in fact it only represents a reduction from 10^8 per 100 ml to 10^6 per 100 ml (that is, almost nothing). A properly designed series of WSP, on the other hand, can easily reduce faecal coliform numbers from 10^8 per 100 ml to $<10^3$ per 100 ml (the WHO guideline value for unrestricted irrigation; see

Section 10.1), which is a removal of 99.999 percent (or 5 log₁₀ units).

A general comparison between WSP and conventional treatment processes for the removal of excreted pathogens is shown in Table 1.2; detailed information is given in Feachem *et al.* (1983).

Table 1.2 Removals of excreted pathogens achieved by waste stabilization ponds and conventional treatment processes

Excreted pathogen	Removal in WSP	Removal in conventional treatment
Bacteria	up to 6 log units ^a	1 – 2 log units
Viruses	up to 4 log units	1 – 2 log units
Protozoan cysts	100%	90-99%
Helminth eggs	100%	90-99%

^a1 log unit = 90 percent removal; 2 = 99 percent; 3 = 99.9 percent, and so on.

WSP are also extremely *robust*: due to their long hydraulic retention time, they can withstand both organic and hydraulic shock loads. They can also cope with high levels of *heavy metals*, up to 60 mg/l (Moshe *et al.*, 1972), so they can treat a wide variety of *industrial wastewaters* that would be too toxic for other treatment processes. Strong wastewaters from agro-industrial processes (for example, abattoirs, food canneries, dairies) are easily treated in WSP. Finally, WSP are the only secondary treatment process that can readily and reliably produce effluents safe for *reuse* in agriculture and aquaculture (see Section 10).

The principal requirements for WSP are that sufficient land is available and that the soil should preferably have a coefficient of permeability less than 10⁻⁷ m/s (to avoid the need for pond lining: see Section 5.2). The investment made by the sewerage authority in land for ponds can always be realised later. For example, the city of Concorde in California purchased land for ponds in 1955 at US\$ 50,000 per ha, and by 1975 it was worth US\$ 375,000 per ha (Oswald, 1976). Inflation during this 20 year period was exactly 100 percent, so the land increased in real value by 375 percent (or 6.8 percent per year).

1.3 ABOUT THIS MANUAL

This Manual is intended as a comprehensive guide for the design, operation and maintenance, monitoring and evaluation, and upgrading of WSP systems in India. Section 2 reviews WSP applicability and usage in India, and Section 3 provides a necessarily brief overview of the function and operation of each principal pond type.

The process design of the different types of pond (anaerobic, facultative and maturation) is described in detail in Section 4, and design examples are given in Annex I. Section 5 details the physical design of ponds and Section 6 their operation and maintenance requirements. Recommendations for routine effluent quality monitoring and WSP performance evaluation are given in Section 7.

Pond rehabilitation and upgrading is described in Section 8. Wastewater storage and treatment reservoirs, which are appropriate in arid and semi-arid areas when treated wastewater is in high demand for crop irrigation, are discussed in Section 9. Finally, Section 10 reviews the agricultural and aquacultural use of treated effluents, with emphasis on measures for the protection of public health.

2.

WSP applicability and usage in India

2.1 APPLICABILITY

Waste stabilization ponds are, as noted in Section 1, a low-cost, low-energy, low-maintenance and, above all, a *sustainable* method of wastewater treatment. They are highly appropriate under many conditions in India – not all, of course, but in the majority of cases an honest appraisal (see Box on pages 4–6) of wastewater treatment alternatives will undoubtedly indicate that WSP are the best option. Well designed WSP, provided they are constructed and maintained properly and not overloaded, will provide a high level of wastewater treatment for very many years. Other wastewater treatment processes can do this as well, of course, but not at the low cost of WSP, nor with their simplicity. This is an extremely important consideration in India, where there is a paucity of wastewater treatment plants, with most wastewater being discharged untreated into a surface watercourse. Effective treatment in low-cost WSP is thus a good way to improve the environment in general and environmental health in particular.

The climate in India, with the possible exception of that in the Northern mountainous areas, is very favourable for the efficient operation of WSP. The intense rainfall occurring during the monsoon is not a factor militating against the use of ponds, for it can easily be taken into account in both the process and the physical design of WSP (Sections 4 and 5). The high temperatures that occur throughout the year in much of India are especially favourable for anaerobic ponds.

2.1.1 Anaerobic ponds

Design engineers are often reluctant to use anaerobic ponds because of a fear that they will cause a significant level of odour nuisance. As noted in Section 3, this is *not* the case if they are properly designed. Anaerobic ponds are so efficient in removing BOD (see Sections 3 and 4) that really there should be no excuse for not using them. They are also very effective in removing heavy metals, which are precipitated as insoluble metal sulphides, and in degrading certain organic compounds (such as phenols) that would otherwise be toxic to the algae in the receiving facultative pond (see Mara and Mills, 1994). Yet in the past aerated lagoons have been favoured over anaerobic ponds, and current fashion is to consider UASBs as a preferable alternative to both aerated lagoons and anaerobic ponds.

Anaerobic ponds or UASBs?

Upflow anaerobic sludge blanket (UASB) reactors are an extremely efficient process for the treatment of high-strength industrial, including agro-industrial, wastewaters: BOD removals of >90 percent are achieved at very short retention times (<10 hours). A full description of UASBs in hot-climate countries is given by van Haandel and Lettinga (1994).

However, UASBs are less suitable for the treatment of domestic and municipal wastewaters, although nearly 20 such plants are presently under construction in India under the Yamuna Action Plan (R.P Sharma, pers. comm.). A common design assumption is that they achieve a 70 percent BOD removal at a retention time of 8 hours, and this has been realised in practice by the full-scale UASBs operating at Kanpur and Mirzapur (Hammad, 1996). An anaerobic pond in a hot climate can also achieve a 70 percent reduction in BOD, but at a retention time of 1 day, rather than 8 hours (see Section 4.3). It may therefore appear that an UASB is “better” than an anaerobic pond. However, when costs are taken into account this is not the case: it will always be less expensive to construct (essentially, excavate) a 1-day anaerobic pond, rather than to construct an UASB in reinforced concrete. Construction costs of the 5 Mld UASB at Kanpur were Rs 3.6 crore (including post-treatment in a 1-day “polishing pond”, but excluding land costs) (Hammad, 1994).

Average UASB construction costs in India are Rs. 35 lakh per Mld, excluding land costs (R P Sharma, pers. comm.).

Anaerobic ponds or aerated lagoons?

It is not uncommon to see domestic and municipal wastewaters being treated in aerated lagoons prior to treatment in facultative and maturation ponds (although the current popularity of UASBs means that this is less common now than it was 10-20 years ago). As noted in Section 1.2, the local sewerage authority often finds that it cannot afford the energy costs of the aerated lagoon, and the unaerated lagoon functions as an anaerobic pond.

Aerated lagoons are designed to achieve a BOD reduction of 70-85 percent at a retention time of 2-6 days (Mara, 1976). Anaerobic ponds in hot climates will achieve a 70 percent BOD removal at a retention time of 1 day (see Section 4.3). Thus they are rather more efficient than aerated lagoons, and they achieve this efficiency at zero energy cost.

2.2 USAGE

Waste stabilization ponds are not a new technology in India. The then Central Public Health Engineering Research Institute organised a Symposium on WSP over 30 years ago (CPHERI, 1963), and published a WSP guidance manual over 20 years ago (Arceivala *et al.*, 1972). Nevertheless, and certainly in recent years, little work on WSP in India has been published, as evidenced by the contents lists of such journals as the *Indian Journal of Environmental Health*. Many of the existing WSP systems in India are old, often poorly maintained and overloaded, and sometimes abandoned. They generally did not include anaerobic ponds.

One State where WSP are favoured is West Bengal. Four modern WSP systems have been installed in the Calcutta region (three within the metropolitan area, at Titagarh, Panihati and Ballay North Howrah, and one just outside, at Nabadwip); two of these are described below. Calcutta is also the site of the largest wastewater-fed fisheries in the world, and a brief description of the 3000 ha Calcutta East fishponds is also given.

2.2.1 Titagarh WSP

The WSP system at Titagarh, which was commissioned in 1995, comprises two series of anaerobic, facultative and a single maturation pond (Figure 2.1). The design flow was 14 Mld, raw wastewater BOD 200 mg/l and faecal coliform numbers 1×10^7 per 100 ml. The retention times at the design flow, the mid-depth pond areas and depths are:

Anaerobic ponds:	1 day	
	0.7 ha	2 m
Facultative ponds:	5 days	
	4.8 ha	1.5 m
Maturation ponds:	4 days	
	3.8 ha	1.5 m

The WSP were designed to produce an effluent suitable for aquaculture reuse, i.e. with a faecal coliform count below 10^4 per 100 ml. In fact, in accordance with the recommendations made by Ghosh (1996), fish culture is currently practised in both the facultative and maturation ponds (rather than in a dedicated fishpond, as recommended in Section 10). This is essentially an interim measure as the wastewater flow is currently around one-third of the design flow. Fish yields are approximately 7 tonnes per ha per year.

The Titagarh WSP are rented out to a local fish-farmer who pays Rs 50,000 p.a. to the local panchayat and Rs 120,000 p.a. to



Figure 2.1
View of WSP at
Titagarh in the
metropolitan
Calcutta area.

the Calcutta Metropolitan Water and Sanitation Authority. This fish culture enterprise is an excellent example of low-cost sustainable wastewater treatment and reuse which not only provides employment for 50 people, but also produces much high-quality animal protein for the local low-income community.

2.2.2 Ballay and North Howrah

The WSP system at Ballay and North Howrah (Figure 2.2), which was commissioned in 1996, is similar to that at Titagarh. The design flow was 30 Mld, and the BOD and FC numbers 150 mg/l and 1×10^7 per 100 ml, respectively. The system comprises three series of anaerobic and facultative ponds, which discharge into two maturation ponds in parallel. Retention times, areas and depths are:

Anaerobic ponds:	1 day	
	1.5 ha	2 m
Facultative ponds:	4 days	
	7.8 ha	1.5 m
Maturation ponds:	3 days	
	6.0 ha	1.5m

As at Titagarh, the Ballay and North Howrah WSP are currently receiving only a third of their design flow, and fish culture is



Figure 2.2
View of WSP at
Ballay North
Howrah in the
metropolitan
Calcutta area.

practised in the facultative and maturation ponds. However, there is one important difference: fish culture is practised not by a local entrepreneur but by a co-operative of local farmers. Thus at these two WSP sites, CMWSA is investigating two different management systems for aquacultural reuse. This will permit the appropriate replication to be made in future projects of this kind.

2.2.3 Calcutta East wastewater-fed fishponds

The Calcutta East wastewater-fed fisheries (Figure 2.3) are the largest example of wastewater-based aquaculture in the world. Some 3000 ha of fishponds are fed with approximately 550,000 m³/d of untreated wastewater by the local fisherman. They produce around 13,000 tonnes of fish (mainly Indian major carp, with some tilapia) per year. This represents 16 percent of the local demand for fish. Average yields are just over 4 tonnes of fish per ha per year, although some of the better managed fishponds produce over 7 tonnes per ha per year.

Currently Indian major carp (catla, *Catla catla*; mrigal, *Cirrhina mrigala*; and rohu, *Labeo rohita*) are stocked at around 3 fingerlings (weighing about 20 g) per m². The ponds are drained only very infrequently (once every 3-4 years), but fish of about 150-250 g (which is the size most commonly consumed by low-income communities) are harvested by sieving each pond 2-4 times per week, some 3 months after stocking. As noted above, yields from the better managed ponds are up to 7 tonnes of fish per ha per year, but this is probably the upper limit using current practices.

These Calcutta East fishponds were developed by the local fishermen some 80 years ago to produce fish, rather than to treat the wastewater. It is a highly successful local enterprise, employing some 4,000 people. As it happens the practice is safe from the point of view of public health, since there are no locally endemic trematode infections, and faecal coliform levels in the fishponds are usually around 1000 per 100 ml (see Section 10.1). Further health protection is given by the local practice of cooking the fish by simmering it for 2-3 hours.

Detailed descriptions of the Calcutta East wastewater-fed fisheries are given by Edwards and Pullin (1990), Edwards (1992), Furedy and Ghosh (1983), Ghosh and Sen (1987, 1992) and Ghosh (1996).



Figure 2.3 The Calcutta East wastewater-fed fishponds: general views (top). The effluent from the fishponds is used partly for crop irrigation but mainly for the cultivation of rice (bottom).

3.

Wastewater treatment in WSP

3.1 TYPES OF WSP AND THEIR FUNCTION

WSP systems comprise a single series of anaerobic, facultative and maturation ponds, or several such series in parallel. In essence, anaerobic and facultative ponds are designed for BOD removal and maturation ponds for pathogen removal, although some BOD removal occurs in maturation ponds and some pathogen removal in anaerobic and facultative ponds. In many instances only anaerobic and facultative ponds will be required: for example, prior to restricted crop irrigation (Section 10.1) and fishpond fertilization (Section 10.4), and also when a relatively weak wastewater (up to 150 mg/l) is to be treated prior to surface water discharge. In general maturation ponds will be required only when the treated wastewater is to be used for unrestricted irrigation and has to comply therefore with the WHO guideline of > 1000 faecal coliforms per 100 ml, and when stronger wastewaters (BOD >150 mg/l) are to be treated prior to surface water discharge. (Restricted irrigation refers to the irrigation of industrial crops, such as cotton and sunflower, and food crops not for direct human consumption, such as wheat. Unrestricted irrigation covers vegetable crops, including those eaten uncooked, such as salad crops.) However, if WSP effluents can be assessed on the basis of filtered BOD (see Section 4.1), anaerobic and facultative ponds will be sufficient without the need for maturation ponds for the treatment of wastewaters with a BOD up to 300 mg/l.

Designers should not be afraid of including anaerobic ponds. Their principal perceived disadvantage – odour release – can be

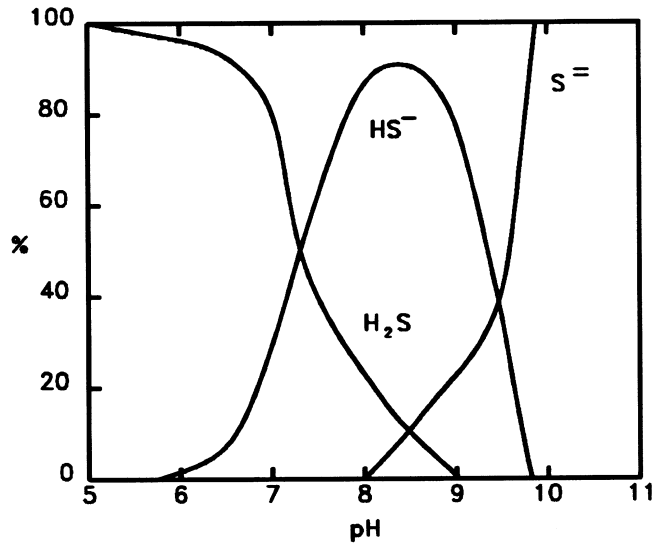
eliminated at the design stage (Section 4.3), and they are so efficient at removing BOD that their inclusion substantially reduces the land area required (see Design Example No. 1 in Annex I).

3.1.1 Anaerobic ponds

Anaerobic ponds are commonly 2-5 m deep (see Section 5.5) and receive such a high organic loading (usually >100 g BOD/m³ d, equivalent to >3000 kg/ha d for a depth of 3 m) that they contain no dissolved oxygen and no algae, although occasionally a thin film of mainly *Chlamydomonas* can be seen at the surface. They function much like open septic tanks, and their primary function is BOD removal (see Section 3.2). Anaerobic ponds work extremely well in warm climates: a properly designed and not significantly underloaded anaerobic pond will achieve around 60 percent BOD removal at 20°C and over 70 per cent at 25°C. Retention times are short: for wastewater with a BOD of up to 300 mg/l, 1 day is sufficient at temperatures $>20^{\circ}\text{C}$ (see Section 4.3).

Designers have in the past been too afraid to incorporate anaerobic ponds in case they cause odour. Hydrogen sulphide, formed mainly by the anaerobic reduction of sulphate by sulphate-reducing bacteria such as *Desulfovibrio*, is the principal potential source of odour. However in aqueous solution hydrogen sulphide is present as either dissolved hydrogen sulphide gas (H_2S) or the bisulphide ion (HS^-), with the sulphide ion (S^{2-}) only really being formed in significant quantities at high pH. Figure 3.1 shows how the distribution of H_2S , HS^- and S^{2-} changes with pH. At the pH values normally found in well designed anaerobic ponds (around 7.5), most of the sulphide is present as the odourless bisulphide ion. Odour is only caused by escaping hydrogen sulphide molecules as they seek to achieve a partial pressure in the air above the pond which is in equilibrium with their concentration in it (Henry's law). Thus, for any given total sulphide concentration, the greater the proportion of sulphide present as HS^- , the lower the release of H_2S . Odour is *not* a problem if the recommended design loadings (Table 4.1) are not exceeded and if the sulphate concentration in the raw wastewater is less than 300 mg SO_4/l (Gloyna and Espino, 1969). A small amount of sulphide is beneficial as it reacts with heavy metals to form insoluble metal sulphides which precipitate out, but con-

Figure 3.1
 Effect of pH on
 hydrogen sulphide
 – bisulphide –
 sulphide
 equilibrium
 (Sawyer *et al.*,
 1994).



centrations of 50-150 mg/l can inhibit methanogenesis (Pfeffer, 1970). A further important advantage of small concentrations (10-12 mg/l) of sulphide in anaerobic ponds is that they are rapidly lethal to *Vibrio cholerae*, the causative agent of cholera (Oragui *et al.*, 1993).

3.1.2 Facultative ponds

Facultative ponds (1-2 m deep) are of two types: primary facultative ponds which receive raw wastewater, and secondary facultative ponds which receive settled wastewater (usually the effluent from anaerobic ponds). They are designed for BOD removal on the basis of a relatively low surface loading (100 -400 kg BOD/ha d) to permit the development of a healthy algal population as the oxygen for BOD removal by the pond bacteria is mostly generated by algal photosynthesis (see Sections 3.2 and 4.4). Due to the algae facultative ponds are coloured dark green, although they may occasionally appear red or pink (especially when slightly overloaded) due to the presence of anaerobic purple sulphide-oxidising photosynthetic bacteria. The algae that tend to predominate in the turbid waters of facultative ponds (see Table 3.1) are the motile genera (such as *Chlamydomonas*, *Pyrobotrys* and *Euglena*) as these can optimise their vertical

Table 3.1 Examples of algal genera present in waste stabilisation ponds

Algae	Facultative Ponds	Maturation Ponds
Euglenophyta		
<i>Euglena</i>	+	+
<i>Phacus</i>	+	+
Chlorophyta		
<i>Chlamydomonas</i>	+	+
<i>Chlorogonium</i>	+	+
<i>Eudorina</i>	+	+
<i>Pandorina</i>	+	+
<i>Pyrobotrys</i>	+	+
<i>Ankistrodesmus</i>	⊗	+
<i>Chlorella</i>	+	+
<i>Micractinium</i>	⊗	+
<i>Scenedesmus</i>	⊗	+
<i>Selenastrum</i>	⊗	+
<i>Carteria</i>	+	+
<i>Coelastrum</i>	⊗	+
<i>Dictyosphaerium</i>	⊗	+
<i>Oocystis</i>	⊗	+
<i>Rhodomonas</i>	⊗	+
<i>Volvox</i>	+	⊗
Chrysophyta		
<i>Navicula</i>	+	+
<i>Cyclotella</i>	⊗	+
Cyanophyta		
<i>Oscillatoria</i>	+	+
<i>Anabaena</i>	+	+

+ = present; ⊗ = absent

position in the pond water column in relation to incident light intensity and temperature more easily than non-motile forms (such as *Chlorella*, although this is also fairly common in facultative ponds). The concentration of algae in a healthy facultative pond depends on loading and temperature, but is usually in the range 500-2000 µg chlorophyll *a* per litre.

As a result of the photosynthetic activities of the pond algae, there is a diurnal variation in the concentration of dissolved oxygen. After sunrise, the dissolved oxygen level gradually rises to a maximum in the mid-afternoon, after which it falls to a

minimum during the night. The position of the oxypause (the depth at which the dissolved oxygen concentration reaches zero) similarly changes, as does the pH since at peak algal activity carbonate and bicarbonate ions react to provide more carbon dioxide for the algae, so leaving an excess of hydroxyl ions with the result that the pH can rise to above 9 which kills faecal bacteria (see Section 3.3.1).

The wind has an important effect on the behaviour of facultative ponds, as it induces vertical mixing of the pond liquid. Good mixing ensures a more uniform distribution of BOD, dissolved oxygen, bacteria and algae and hence a better degree of waste stabilisation. In the absence of wind-induced mixing, the algal population tends to stratify in a narrow band, some 20 cm thick, during daylight hours. This concentrated band of algae moves up and down through the top 50 cm of the pond in response to changes in incident light intensity, and causes large fluctuations in effluent quality (especially BOD and suspended solids) if the effluent take-off point is within this zone (see Section 5).

3.1.3 Maturation ponds

A series of maturation ponds (1-1.5m deep) receives the effluent from a facultative pond, and the size and number of maturation ponds is governed mainly by the required bacteriological quality of the final effluent (see Sections 4.1 and 4.5). Maturation ponds usually show less vertical biological and physicochemical stratification and are well oxygenated throughout the day. Their algal population is thus much more diverse than that of facultative ponds (Table 3.1) with non-motile genera tending to be more common; algal diversity increases from pond to pond along the series.

The primary function of maturation ponds is the removal of excreted pathogens, and this is extremely efficient in a properly designed series of ponds (Table 3.2). Maturation ponds achieve only a small removal of BOD, but their contribution to nutrient (nitrogen and phosphorus) removal can be significant (see Sections 3.4 and 4.5.4).

3.2 BOD REMOVAL

In anaerobic ponds BOD removal is achieved (as in septic tanks) by sedimentation of settleable solids and subsequent anaerobic

Table 3.2 Geometric mean bacterial and viral numbers^a and percentage removals in raw wastewater (RW) and the effluents of five waste stabilisation ponds in series (P1-P5^b) at a mean mid-depth pond temperature of 26°C.

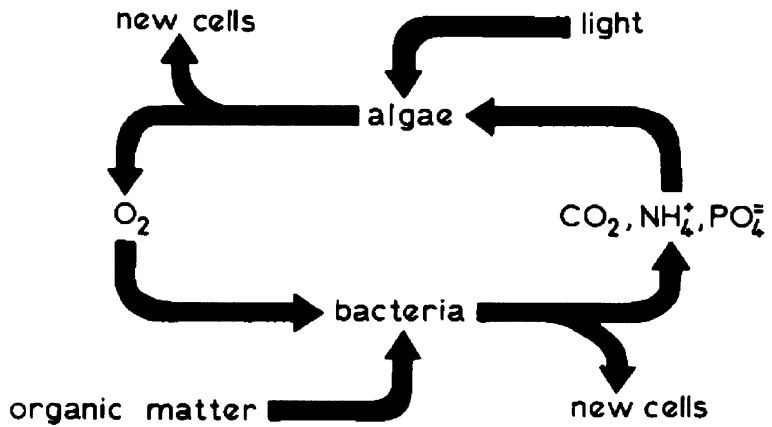
Organism	RW	P1	P2	P3	P4	P5	Percentage removal
Faecal coliforms	2×10^7	4×10^6	8×10^5	2×10^5	3×10^4	7×10^3	99.97
Faecal streptococci	3×10^6	9×10^5	1×10^5	1×10^4	2×10^3	300	99.99
<i>Clostridium perfringens</i>	5×10^4	2×10^4	6×10^3	2×10^3	1×10^3	300	99.40
Campylobacters	70	20	0.2	0	0	0	100.00
Salmonellae	20	8	0.1	0.02	0.01	0	100.00
Enteroviruses	1×10^4	6×10^3	1×10^3	400	50	9	99.91
Rotaviruses	800	200	70	30	10	3	99.63
BOD (mg/l)	215	36	41	21	21	18	92

^a Bacterial numbers per 100 ml, viral numbers per 10 litres.

^b P1 was an anaerobic pond with a mean hydraulic retention time of 1 day; P2 and P3-P5 were secondary facultative and maturation ponds respectively, each with a retention time of 5 days.

Source: Oragui *et al.* (1987).

Figure 3.2
Mutualistic
relationship
between pond
algae and pond
bacteria.



digestion in the resulting sludge layer : this is particularly intense at temperatures above 15°C when the pond surface literally bubbles with the release of biogas (around 70 percent methane and 30 percent carbon dioxide); methane production increases sevenfold for every 5°C rise in temperature (Marais, 1970).

The bacterial groups involved are the same as those in any anaerobic reactor – the anaerobic acidogens and the methanogens, and those in anaerobic ponds are equally sensitive to the same toxicants, one of which is low pH (< 6.2). Acidic wastewaters thus require neutralising prior to treatment in anaerobic ponds.

In secondary facultative ponds that receive settled wastewater (usually anaerobic pond effluent), the remaining non-settleable BOD is oxidised by the normal heterotrophic bacteria of wastewater treatment (*Pseudomonas*, *Flavobacterium*, *Archromobacter* and *Alcaligenes* spp.), but with one important difference : these bacteria obtain the oxygen they need not from mechanical aeration (as they do in aerated lagoons, oxidation ditches and activated sludge tanks), but from the photosynthetic activities of the micro-algae which grow naturally and profusely in facultative ponds, giving them their characteristic dark green colour. The algae, in turn, depend largely on the bacteria for the carbon dioxide which they photosynthetically convert into sugars:



So there exists a *mutualistic* relationship between the pond algae and the pond bacteria: the algae provide the bacteria with oxygen and the bacteria provide the algae with carbon dioxide (Figure 3.2). Of course some oxygen and carbon dioxide comes

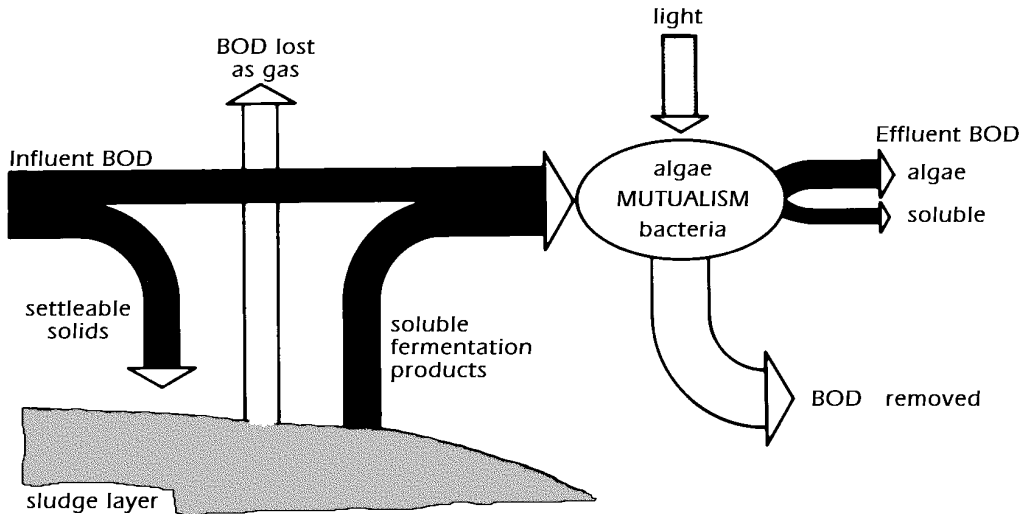


Figure 3.3 Pathways of BOD removal in primary facultative ponds (after Marais, 1970).

from the atmosphere by mass transfer, but the bulk is supplied by algal-bacterial mutualism.

In primary facultative ponds (those that receive raw wastewater) the above functions of anaerobic and secondary facultative ponds are combined, as shown in Figure 3.3. Around 30 percent of the influent BOD leaves a primary facultative pond in the form of methane (Marais, 1970).

As a result of these algal-bacterial activities, a high proportion of the BOD that does not leave the pond as methane ends up as algal cells. Thus in secondary facultative ponds (and in the upper layers of primary facultative ponds) “sewage BOD” is converted into “algal BOD” and this has important implications for effluent quality requirements (see Section 4.1).

In maturation ponds only a small amount of BOD removal occurs, principally as a result of lower algal concentrations (and hence lower “algal BOD”) which, in turn, result from a decreased supply of nutrients and predation by protozoa and micro-invertebrates such as *Daphnia* or by fish such as carp if these are present. Around 70-90 percent of the BOD of a maturation pond effluent is due to the algae it contains.

3.3 PATHOGEN REMOVAL

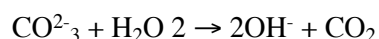
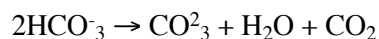
3.3.1 Bacteria

Faecal bacteria are mainly removed in facultative and especially maturation ponds whose size and number determine the numbers of faecal bacteria (usually modelled in terms of faecal coliforms) in the final effluent (Section 4.2.4.), although there is some removal in anaerobic ponds principally by sedimentation of solids-associated bacteria.

The principal mechanisms for faecal bacterial removal in facultative and maturation ponds are now known to be:

- (a) time and temperature,
- (b) high pH (> 9), and
- (c) high light intensity together with high dissolved oxygen concentration.

Time and temperature are the two principal parameters used in maturation pond design (Section 4.2.4.): faecal bacterial die-off in ponds increases with both time and temperature (Feachem *et al.*, 1983). High pH values above 9 occur in ponds due to rapid photosynthesis by the pond algae which consumes CO₂ faster than it can be replaced by bacterial respiration; as a result carbonate and bicarbonate ions dissociate:



The resulting CO₂ is fixed by the algae and the hydroxyl ions accumulate so raising the pH, often to above 10. Faecal bacteria (with the notable exception of *Vibrio cholerae*) die very quickly (within minutes) at pH > 9 (Pearson *et al.*, 1987c).

The role of high light intensity and high dissolved oxygen concentration has recently been elucidated (Curtis *et al.*, 1992). Light of wavelengths 425 – 700 nm can damage faecal bacteria by being absorbed by the humic substances ubiquitous in wastewater: these then enter an excited state for long enough to damage the cell. Light-mediated die-off is completely dependent on the presence of oxygen, and it is considerably enhanced at high pH. The sun thus plays a threefold role in promoting faecal bacterial removal in WSP (Figure 3.4): directly, by increasing the pond temperature ; and more indirectly, by providing the energy for rapid algal photosynthesis which not only raises the pond pH above 9 but also results in high dissolved oxygen concentrations

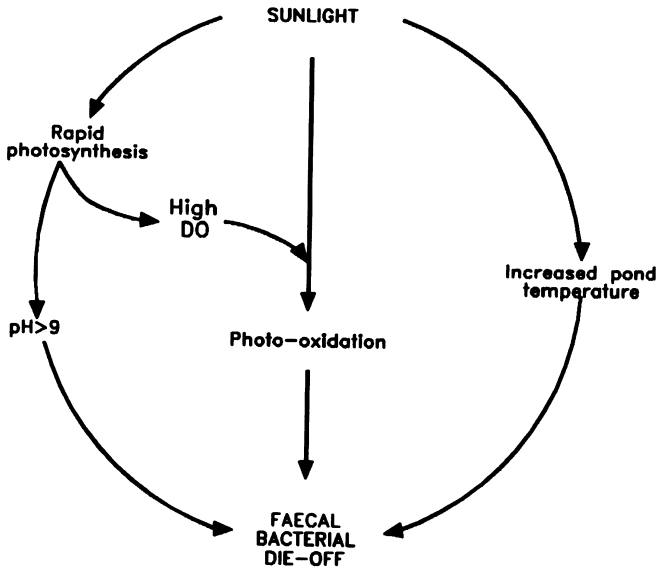


Figure 3.4
Conceptual mechanisms for faecal coliform die-off in ponds.

which are necessary for its third role, that in promoting photo-oxidative damage.

3.3.2 Viruses

Little is definitely known about the mechanisms of viral removal in WSP, but it is generally recognised that it occurs by adsorption on to settleable solids (including the pond algae) and consequent sedimentation.

3.3.3 Parasites

Protozoan cysts and helminth eggs are removed by sedimentation. Their settling velocities are quite high (for example, 3.4×10^{-4} m/s in the case of *Ascaris lumbricoides*), and consequently most removal takes place in the anaerobic and facultative ponds. It has recently become possible to design WSP for helminth egg removal (Ayres *et al.*, 1992a; see Section 4.5.2. and Design Example No. 2 in Annex I); this is necessary if the effluent is to be used for restricted crop irrigation (Section 10.1).

3.4 NUTRIENT REMOVAL

3.4.1 Nitrogen

In WSP systems the nitrogen cycle is at work, with the probable exception of nitrification and denitrification. In anaerobic ponds organic nitrogen is hydrolysed to ammonia, so ammonia concentrations in anaerobic pond effluents are generally higher than in the raw wastewater (unless the time of travel in the sewer is so long that all the urea has been converted before reaching the WSP). In facultative and maturation ponds, ammonia is incorporated into new algal biomass. Eventually the algae become moribund and settle to the bottom of the pond; around 20 percent of the algal cell mass is non-biodegradable and the nitrogen associated with this fraction remains immobilised in the pond sediment. That associated with the biodegradable fraction eventually diffuses back into the pond liquid and is recycled back into algal cells to start the process again. At high pH, some of the ammonia will leave the pond by volatilization.

There is little evidence for nitrification (and hence denitrification, unless the wastewater is high in nitrates). The populations of nitrifying bacteria are very low in WSP due primarily to the absence of physical attachment sites in the aerobic zone, although inhibition by the pond algae may also occur.

Total nitrogen removal in WSP systems can reach 80 percent or more, and ammonia removal can be as high as 95 percent. Equations for estimating total and ammoniacal nitrogen removals are given in Section 4.5.4.

3.4.2 Phosphorus

The efficiency of total phosphorus removal in WSP depends on how much leaves the pond water column and enters the pond sediments – this occurs due to sedimentation as organic P in the algal biomass and precipitation as inorganic P (principally as hydroxyapatite at pH levels above 9.5) – compared to the quantity that returns through mineralization and resolubilization. As with nitrogen, the phosphorus associated with the non-biodegradable fraction of the algal cells remains in the sediments. Thus the best way of increasing phosphorus removal in WSP is to increase the

number of maturation ponds, so that progressively more and more phosphorus becomes immobilized in the sediments. A first order plug flow model for phosphorus removal has been developed, (Huang and Gloyna, 1984), but it is not in a form useful for design. The model shows that, if the BOD removal is 90 percent, then phosphorus removal is around 45 percent.

3.5 ENVIRONMENTAL IMPACT OF WSP SYSTEMS

Adverse environmental impacts resulting from the installation of a waste stabilisation pond system should normally be minimal, and the positive impacts, such as alleviation of water pollution, should greatly outweigh any potential negative impacts such as odour nuisance or mosquito breeding (but these do *not* occur in well-designed and well-maintained WSP). However, environmental impact assessments (EIA) are now recognised as an essential component in any development project and as an important decision-making tool, and the appropriate procedures should be followed. Annex III outlines the guidelines recommended by UNEP (1990) for the preparation of an EIA document for a sewage treatment plant for cities with populations of 10,000 – 100,000 and 100,000 – 1,000,000. The reader is also referred to the *Environmental Assessment Sourcebook* published by the World Bank (1991).

4.

Process design of WSP

4.1 EFFLUENT QUALITY REQUIREMENTS

In India general standards for the discharge of treated wastewaters into inland surface waters are given in the Environment (Protection) Rules 1986 (see CPCB, 1996). The more important of these for WSP design are as follows:

BOD	30 mg/l
Suspended solids	100 mg/l
Total N	100 mg N/l
Total ammonia	50 mg N/l
Free ammonia	5 mg N/l
Sulphide	2 mg/l
pH	5.5 – 9.0

The sulphide standard means that effluents from anaerobic ponds (and indeed from other anaerobic reactors such as UASBs) are not suitable for discharge into surface waters as they generally contain 10-15 mg sulphide per litre. The other requirements are not particularly difficult for WSP effluents to comply with, except perhaps BOD when only anaerobic and facultative ponds are used to treat wastewaters with a BOD above 150 mg/l, in which case maturation ponds would be required to reduce the BOD to below 30 mg/l.

However, it must be remembered that 70-90 percent of the BOD of the final effluent from a series of well designed WSP is due to the algae it contains, and “algal BOD” is very different in nature to “sewage BOD”. Thus many countries permit a higher BOD in WSP effluents than they do in effluents from other types of treatment plant, or they make some other allowance for WSP effluents. In the European Union, for example, pond effluents

have to meet the same BOD requirement as other effluents (<25 mg/l) but with one very important difference: *filtered* samples are used to determine the BOD, which is therefore the residual non-algal BOD (Council of the European Communities, 1991), although of course filtration removes non-algal solids as well – but in WSP effluents the algae comprise most (>80%) of the suspended solids. Furthermore in the EU pond effluents can contain up to 150 mg SS per litre, whereas effluents from other treatment processes can contain only 35 mg SS/l. This recognises the distinctions between algal and sewage BOD and algal and sewage SS. The algae in WSP effluents readily disperse and are consumed by zooplankton in receiving waters, so they have little chance to exert their BOD, and during daylight hours they of course *produce* oxygen. In agricultural reuse schemes pond algae are very beneficial: they act as slow-release fertilizers and increase the soil organic matter, so improving its water-holding capacity.

In India, the Ministry of Urban Development (1995) recognises that unfiltered BOD is not an appropriate basis for evaluating the quality of pond effluents, and recommends the use of filtered BOD. This would mean that, if a 1-day anaerobic and 5-day facultative pond achieved a cumulative filtered BOD removal of 90% (based on filtered BOD for the pond effluent, but on unfiltered BOD for the raw wastewater), the general effluent requirement of 30 mg BOD/l would be achieved (but on a filtered basis) in these two ponds treating a raw wastewater with a BOD of up to 300 mg/l (i.e. equivalent to 45 grams of BOD per caput per day and 150 litres of wastewater per caput per day). Currently, however, CPCB recommends the use of unfiltered BOD (S.D. Makhijani, pers. comm.).

The above CPCB effluent quality requirements have, of course, a cost associated with them. Since in India there is currently very little treatment of wastewater – most is discharged into rivers untreated – it may be preferable in the short term to adopt a more pragmatic approach and decide to treat the wastewater to a lesser quality, at least initially. Thus, while treatment in anaerobic and facultative ponds may not comply with all the CPCB requirements, it does represent a considerable improvement over the discharge of untreated wastewater, and it may be all that a city can afford at present. Section 4.6 details the land area requirements for such partial treatment for a range of design temperatures.

4.2 DESIGN PARAMETERS

The four most important parameters for WSP design are temperature, net evaporation, flow and BOD. Faecal coliform and helminth egg numbers are also important if the final effluent is to be used in agriculture or aquaculture.

4.2.1 Temperature and net evaporation

The usual design temperature is the mean air temperature in the coolest month (or quarter). This provides a small margin of safety as pond temperatures are 2-3° C warmer than air temperatures in the cool season (the reverse is true in the hot season). Another design temperature commonly used is the air temperature in the coolest period of the irrigation season. Net evaporation (= evaporation – rainfall) has to be taken into account in the design of facultative and maturation ponds (Shaw, 1962), but not in that of anaerobic ponds, as these generally have a scum layer which effectively prevents significant evaporation. The net evaporation rates in the months used for selection of the design temperatures are used; additionally a hydraulic balance should be done for the hottest month (see Section 5.3).

A general description of the climate of India is given by Rao (1981). The India Meteorological Department is able to provide detailed data for most locations in India – its centre in Pune is currently publishing a volume containing comprehensive meteorological data up to the year 1985.

4.2.2 Flow

The mean daily flow should be measured if the wastewater exists. If it does not, it must be estimated very carefully, since the size of the ponds, and hence their cost, is directly proportional to the flow. The wastewater flow should not be based on the design water consumption per caput, as this is unduly high since it contains an allowance for losses in the distribution system. A suitable design value is 80 percent of the in-house water consumption, and this can be readily determined from records of water meter readings. If these do not exist, the actual average 24-hour wastewater flow from outfall drains can be measured; or alternatively the design

flow may be based on local experience in sewerred communities of similar socio-economic status and water use practice. The Ministry of Urban Development (1995) permits a wastewater design flow of 150 litres per caput per day to be used in the absence of any local data.

4.2.3 BOD

If the wastewater exists, its BOD may be measured using 24-hour flow-weighted composite samples (see Section 7.1). If it does not, it may be estimated from the following equation:

$$L_i = 1000 B/q \quad (4.1)$$

where L_i = wastewater BOD, mg/l
 B = BOD contribution, g/caput d
 q = wastewater flow, l/caput d

Values of B vary between 30 and 70 g per caput per day, with affluent communities producing more BOD than poor communities (Campos and von Sperling, 1996). A suitable design value for India is 45 g per caput per day (Ministry of Urban Development, 1995).

4.2.4 Nitrogen

The general standards for various forms of nitrogen in effluent discharged into inland surface waters (Section 4.1) are not likely to cause difficulty, although more stringent requirements may need to be considered if the effluent is to be discharged into a pristine lake that would be subject to serious eutrophication.

Total nitrogen and free ammonia (NH_3 , rather than NH_4^+ NH_3) are important in the design of wastewater-fed fishponds (Section 10.4.2). Concentrations of total nitrogen in raw domestic wastewater are 15-60 mg N/l, and total ammonia (NH_4^+ + NH_3) concentrations are 10 – 35 mg N/l.

4.2.5 Faecal coliforms

Faecal coliform numbers are important if the pond effluent is to be used for unrestricted crop irrigation or for fishpond fertilization

(Section 10). Grab samples of the wastewater may be used to measure the faecal coliform concentration if the wastewater exists. The usual range is 10^7 - 10^8 faecal coliforms per 100 ml, and a suitable design value is 5×10^7 per 100 ml.

4.2.6 Helminth eggs

Helminth egg numbers are also important when pond effluents are used for crop irrigation or fishpond fertilization (Section 10). If the wastewater exists, composite samples may be used to count the number of human intestinal nematodes eggs (see Ayres and Mara, 1996). The usual range is 100-1000 eggs per litre, with affluent communities producing much fewer eggs than newly sewered poor communities (although egg numbers from the latter will fall over time as the opportunities for reinfection will be greatly reduced by the provision of sewerage).

4.3 ANAEROBIC PONDS

No advice is given on the design of anaerobic ponds in the Government of India's *Manual on Sewerage and Sewage Treatment* (Ministry of Urban Development, 1995). However, they can be satisfactorily designed – and without risk of odour nuisance (see Section 3.1.1 and below) – on the basis of volumetric BOD loading (λ_v , $\text{g/m}^3\text{d}$), which is given by:

$$\lambda_v = L_i Q / V_a \quad (4.2)$$

where L_i = influent BOD, mg/l ($= \text{g/m}^3$)

Q = flow, m^3/d

V_a = anaerobic pond volume, m^3

The permissible design value of λ_v increases with temperature, but there are too few reliable data to permit the development of a suitable design equation. Mara and Pearson (1986) and Mara *et al.* (1997) recommend the design values given in Table 4.1 which may be safely used for design purposes in India. These recommendations were based on those of Meiring *et al.* (1968) that λ_v should lie between 100 and 400 $\text{g/m}^3\text{d}$, the former in order to maintain anaerobic conditions and the latter to avoid odour release (see also Mara and Mills, 1994). However, in Table 4.1 the upper limit for design is set at 350 $\text{g/m}^3\text{d}$ in order to provide an

adequate margin of safety with respect to odour. This is appropriate for normal domestic or municipal wastewaters which contain less than 300 mg SO₄/l.

Once a value of λ_V has been selected, the anaerobic pond volume is then calculated from equation 4.2. The mean hydraulic retention time in the pond (θ_a , d) is determined from:

$$\theta_a = V_a/Q \quad (4.3)$$

Retention times in anaerobic ponds <1 day should not be used. If equation 4.3 gives a value of θ_a <1 day, a value of 1 day should be used and the corresponding value of V_a recalculated from equation 4.2.

Table 4.1 Design values of permissible volumetric BOD loadings on and percentage BOD removal in anaerobic ponds at various temperatures

Temperature (°C)	Volumetric loading (g/m ³ d)	BOD removal (%)
<10	100	40
10-20	20T -100	2T +20
20-25	10T + 100	2T + 20
>25	350	70

T = temperature, °C.

Source: Mara and Pearson (1986) and Mara *et al.* (1997).

Table 4.2 Variation of BOD removal with retention time in anaerobic ponds in northeast Brazil at 25°C^a

Retention time (d)	Volumetric loading rate (g/m ³ /day)	BOD removal (%)
0.8	306	76
1.0	215	76
1.9	129	80
2.0	116	75
4.0	72	68
6.8	35	74

^a The ponds were located in Campina Grande, Paraiba State (latitude 7°13'11"S, longitude 35°52'31"W, altitude 550 m above m.s.l.). The mean BOD of the raw municipal wastewater was 230 – 290 mg/l.

Source: Silva (1982).

The performance of anaerobic ponds increases significantly with temperature, and the design assumptions for BOD removal (needed for the design of the receiving facultative pond) given in Table 4.1 can be confidently adopted. These are based on experience with anaerobic ponds in Germany in winter ($T < 10^{\circ}\text{C}$) (Bucksteeg, 1987), and in northeast Brazil at 25°C (Table 4.2) where conditions are very similar to those in India.

Anaerobic ponds in series

With domestic wastewater there is no advantage in having two anaerobic ponds in series (Silva, 1982). The first anaerobic pond, designed as described above, will reduce the BOD from, for example, 240 mg/l to 60 mg/l at 25°C (Table 4.2), and the second will reduce this to only 45 mg/l or so. However, with high-strength industrial wastewaters, two or more anaerobic ponds in series can be very advantageous. McGarry and Pescod (1970) describe a system of five anaerobic ponds in series for the treatment of a tapioca starch waste with a BOD of 3800 mg/l. The same volumetric BOD loading of $224 \text{ g/m}^3 \text{ day}$ (for a depth of 3 m) was applied to each pond, so their areas and thus retention times decreased along the series. The BOD was reduced to 255 mg/l, equivalent to a removal of 93%.

Subba Rao (1972) reported that two experimental anaerobic ponds in series, receiving volumetric BOD loadings of 600 and $700 \text{ g/m}^3 \text{ d}$ for the first and second pond respectively, were able to reduce the BOD of distillery wastewaters from $40,000 \text{ m}^3/\text{l}$ to 600 mg/l. This performance was better than that achieved at full scale: Subba Rao quoted a reduction from 1,000 to 1,800 mg BOD/l in seven anaerobic ponds in series treating spend wash and sugar factory wastes, and Rao and Viraraghavan (1985) describe the use of two anaerobic ponds in series for the treatment of distillery wastes in Tamil Nadu: the BOD was reduced from 40,000 mg/l to 5,000 mg/l by the first pond, and to 2,000 mg/l in the second pond. Further treatment was provided in an oxidation ditch to produce a final effluent of 100 mg/l (although presumably a third or even fourth anaerobic pond would have been effective in reducing the BOD to a level suitable for treatment in a facultative pond).

4.4 FACULTATIVE PONDS

Although there are several methods available for designing facultative ponds (Mara, 1976), it is recommended that they be designed on the basis of surface BOD loading (λ_S , kg/ha d), which is given by:

$$\lambda_S = 10 L_i Q / A_f \quad (4.4)$$

where A_f = facultative pond area, m^2

The Indian *Manual on Sewerage and Sewage Treatment* (Ministry of Urban Development, 1995) gives two methods of selecting the permissible design value of λ_S : one based on latitude, and one based on temperature.

4.4.1 Latitude

The variation of permissible design value for λ_S with latitude in India is given in Table 4.3 (Arceivala *et al.*, 1970). This relationship can be expressed mathematically as:

$$\lambda_S = 375 - 6.25 L \quad (4.5)$$

where L = latitude, °N (range considered for India : 8 – 36°N).

Table 4.3 Variation of design BOD loading on facultative ponds in India with latitude

Latitude (°N)	Design BOD loading (kg/ha day)
36	150
32	175
28	200
24	225
20	250
16	275
12	300
8	325

Source: Ministry of Urban Development (1993).

Table 4.3 and equation 4.5 are stated to be approximately valid for facultative ponds 0.9 – 1.5 m (3 – 5 ft) deep, which are located

at sea level in areas where the sky is clear for at least 75% of the days in a year (274 days). When the “sky clearance factor” is less than 75%, the value of λ_S given by equation 4.5 should be decreased by 3% for every 10% reduction in the sky clearance factor below 75%; and, to allow for elevations above sea level, the value given by equation 4.5 should be divided by the following factor:

$$[1 + (3 \times 10^{-4}) E]$$

where E = elevation above mean sea level, m

4.4.2 Temperature

Here the permissible design value of λ_S increases with temperature (T , °C). The earliest relationship between λ_S and T is that given by McGarry and Pescod (1970), but their value of λ_S is the *maximum* that can be applied to a facultative pond before it fails (that is, becomes anaerobic). Their relationship, which is therefore *an envelope of failure*, is:

$$\lambda_S = 60 (1.099)^T \quad (4.6)$$

An early design equation for λ_S was given by Mara (1976), and this is included in the *Manual on Sewerage and Sewage Treatment* (Ministry of Urban Development, 1995):

$$\lambda_S = 20T - 120 \quad (4.7)$$

However, a more appropriate global design equation was given by Mara (1987):

$$\lambda_S = 350 (1.107 - 0.002T)^{T-25} \quad (4.8)$$

Equations 4.6 – 4.8 are shown graphically in Figure 4.1, and Table 4.4 gives values of λ_S from equation 4.8 for the temperature range 11-30°C.

Table 4.5 gives a comparison between the design values of λ_S calculated by the two methods based on latitude and on temperature (*i.e.* from equations 4.5 and 4.8) for Calcutta, Chennai, Delhi and Mumbai. It is seen that the two methods are in agreement to within about 20%. Given that there is more global experience with equation 4.8 than with equation 4.4, it is recommended that the former be used for design in India. Furthermore, it automatically takes into account the decrease in temperature with increasing altitude.

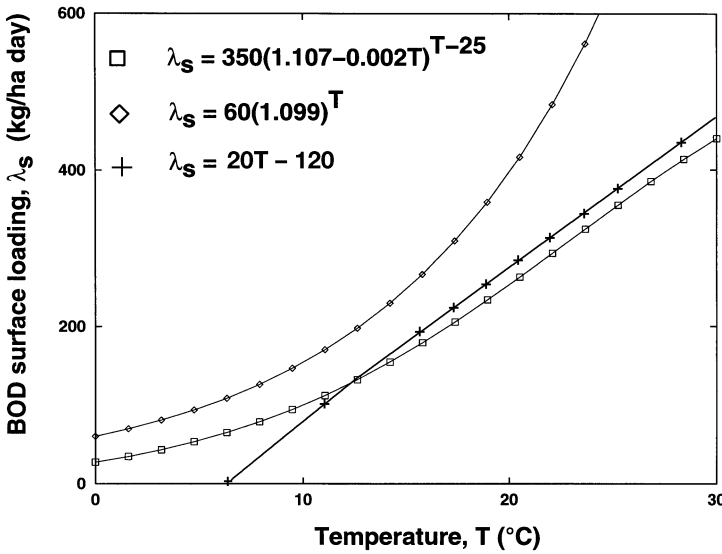


Figure 4.1
Variation of surface BOD loading on facultative ponds with temperature according to equations 4.6 – 4.8.

Table 4.4 Values of the permissible surface BOD loading on facultative ponds at various temperatures (calculated from equation 4.8)

T ($^{\circ}\text{C}$)	λ_s (kg/ha d)	T ($^{\circ}\text{C}$)	λ_s (kg/ha d)
11	112	21	272
12	124	22	291
13	137	23	311
14	152	24	331
15	167	25	350
16	183	26	369
17	199	27	389
18	217	28	406
19	235	29	424
20	253	30	440

Once a suitable value of λ_s has been selected, the pond area is calculated from equation 4.4 and its retention time (θ_f , d) from:

$$\theta_f = A_f D / Q_m \tag{4.9}$$

where D = pond depth, m (usually 1.5 m – see Section 5.1)
 Q_m = mean flow, m^3/day

Table 4.5 Comparison of design methods for surface BOD loading on facultative ponds based on latitude (equation 4.5) and on temperature (equation 4.8) for selected cities in India

City	Latitude Temperature	Design loading (kg/ha day)
Calcutta	22°32'N	234
	19°C	235
Chennai	13°04'N	294
	24°C	331
Delhi	28°35'N	183 ^a
	14°C	152
Mumbai	18°54'N	257
	23°C	311

^a Allowing for Delhi's altitude of 218 m.

The mean flow is the mean of the influent and effluent flows (Q_i and Q_e), the latter being the former less net evaporation and seepage. Thus equation 4.9 becomes:

$$\theta_f = A_f D / [^{1/2}(Q_i + Q_e)] \quad (4.10)$$

If seepage is negligible, Q_e is given by:

$$Q_e = Q_i - 0.001 A_f e \quad (4.11)$$

where e = net evaporation rate, mm/day. Thus equation 4.10 becomes:

$$\theta_f = 2A_f D / (2Q_i - 0.001A_f e) \quad (4.12)$$

A minimum value of θ_f of 5 days should be adopted for temperatures below 20°C, and 4 days for temperatures above 20°C. This is to minimise hydraulic short-circuiting and to give the algae sufficient time to multiply (i.e. to prevent algal washout).

The facultative pond area calculated from equation 4.4 (or from equation 4.12 if the minimum value for θ_f is adopted) should be used only for the facultative pond. This may sound obvious, but both the first and second editions of the *Manual on Sewerage and Sewage Treatment* (Ministry of Urban Development, 1987 and 1995) permit only 65-70% of the calculated area to be used for the facultative pond, with the remaining 30-35% to be used for a

maturation pond. This increases the BOD surface loading on the now smaller facultative pond by 43-54%, and the resulting higher loading is generally too close to the failure loading given by equation 4.6.

4.4.3 BOD Removal

The BOD removal in primary facultative ponds is usually in the range 70-80 percent based on unfiltered samples (that is, including the BOD exerted by the algae), and usually above 90 percent based on filtered samples. In secondary facultative ponds the removal is less, but the combined performance of anaerobic and secondary facultative ponds generally approximates (or is slightly better than) that achieved by primary facultative ponds.

Design Example No. 1 in Annex I shows how anaerobic and facultative ponds are designed to produce an effluent suitable for surface water discharge.

4.5 MATURATION PONDS

4.5.1 Faecal coliform removal

The method of Marais (1974) is generally used to design a pond series for faecal coliform removal. This assumes that faecal coliform removal can be modelled by first order kinetics in a completely mixed reactor. The resulting equation for a single pond is thus:

$$N_e = N_i / (1 + k_T \theta) \quad (4.13)$$

where N_e = number of FC per 100 ml of effluent
 N_i = number of FC per 100 ml of influent
 k_T = first order rate constant for FC removal, d^{-1}
 θ = retention time, d

For a series of anaerobic, facultative and maturation ponds, equation 4.13 becomes:

$$N_e = N_i / [(1 + k_T \theta_a)(1 + k_T \theta_f)(1 + k_T \theta_m)^n] \quad (4.14)$$

where N_e and N_i now refer to the numbers of FC per 100 ml of the final effluent and raw wastewater respectively; the sub-scripts a, f

and m refer to the anaerobic, facultative and maturation ponds; and n is the number of maturation ponds.

It is assumed in equation 4.14 that all the maturation ponds are equally sized: this is the most efficient configuration (Marais, 1974), but may not be topographically possible (in which case the last term of the denominator in equation 4.14 is replaced by $[(1+k_T\theta_{m1})(1+k_T\theta_{m2})\dots(1+k_T\theta_{mn})]$).

The value of k_T is highly temperature dependent. Marais (1974) found that:

$$k_T = 2.6 (1.19)^{T-20} \tag{4.15}$$

Thus k_T changes by 19 percent for every change in temperature of 1 degC (see Table 4.6).

Table 4.6 Values of the first order rate constant for faecal coliform removal at various temperatures (calculated from equation 4.15)

T(°C)	$k_T(\text{day}^{-1})$	T(°C)	$k_T(\text{day}^{-1})$
11	0.54	21	3.09
12	0.65	22	3.68
13	0.77	23	4.38
14	0.92	24	5.21
15	1.09	25	6.20
16	1.30	26	7.38
17	1.54	27	8.77
18	1.84	28	10.46
19	2.18	29	12.44
20	2.60	30	14.81

Maturation ponds require careful design to ensure that their FC removal follows that given by equations 4.14 and 4.15. If they are suboptimally loaded, then their FC removal performance may be correspondingly suboptimal.

Examination of equation 4.14 shows that it contains two unknowns, θ_m and n , since by this stage of the design process θ_a and θ_f will have been calculated, N_i measured or estimated (Section 4.2), N_e set (at, for example, 1000 per 100 ml for unrestricted irrigation; see Table 10.1) and k_T calculated from

equation 4.15. The best approach to solving equation 4.14 is to calculate the values of θ_m corresponding to $n = 1, 2, 3$ etc. and then adopt the following rules to select the most appropriate combination of θ_m and n :

- (a) $\theta_m > \theta_f$
- (b) $\theta_m < \theta_m^{\min}$

where θ_m^{\min} is the minimum acceptable retention time in a maturation pond. This is introduced to minimise hydraulic short-circuiting and prevent algal washout. Marais (1974) recommends a value for it of 3 days, although at temperatures below 20° values of 4-5 days are preferable.

The remaining pairs of θ_m and n , together with the pair θ_m^{\min} and \tilde{n} , where \tilde{n} is the first value of n for which θ_m is less than θ_m^{\min} , are then compared and the one with the least product selected as this will give the least land area requirements. A check must be made on the BOD loading on the first maturation pond: this must not be higher than that on the preceding facultative pond, and it is preferable that it is significantly lower. In this Manual the maximum permissible BOD loading on the first maturation pond is taken as 75 percent of that on the preceding facultative pond. (It is not necessary to check the BOD loadings on subsequent maturation ponds as the non-algal BOD contribution to the load on them is very low.)

The loading on the first maturation pond is calculated on the assumption that 80 percent of the BOD has been removed in the preceding anaerobic and facultative ponds (or 70% for temperatures below 20°C). Thus:

$$\lambda_{S(m1)} = 10 (0.2 L_i) Q/A_{m1} \quad (4.16)$$

or, since $Q\theta_{m1} = A_{m1}D$:

$$\lambda_{S(m1)} = 10 (0.2 L_i) D/\theta_{m1} \quad (4.17)$$

The maturation pond area is calculated from a rearrangement of equation 4.12:

$$A_m = 2Q_i\theta_m/(2D + 0.001e\theta_m) \quad (4.18)$$

Design Example No. 3 in Annex I shows how maturation ponds are designed to produce an effluent suitable for unrestricted irrigation.

4.5.2 Helminth egg removal

Helminth eggs are removed by sedimentation and thus most egg removal occurs in anaerobic or primary facultative ponds. However, if the final effluent is to be used for restricted irrigation (see Section 10), then it is necessary to ensure that it contains no more than one egg per litre (Table 10.1). Depending on the number of helminth eggs present in the raw wastewater and the retention times in the anaerobic and facultative ponds, it may be necessary to incorporate a maturation pond to ensure that the final effluent contains at most only one egg per litre. Analysis of egg removal data from ponds in Brazil, India and Kenya (Ayres *et al.*, 1992a) has yielded the following relationship (see Figure 4.2), which is equally valid for anaerobic, facultative and maturation ponds:

$$R = 100 [1 - 0.14\exp(-0.38\theta)] \quad (4.19)$$

where R = percentage egg removal
 θ = retention time, d

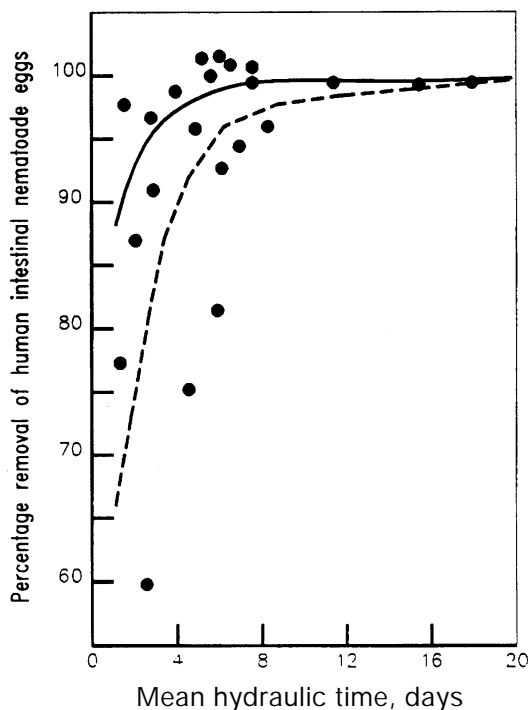


Figure 4.2
 Variation of percentage helminth egg removal with retention time. Solid line: equation 4.19; dotted line, equation 4.20.

The equation corresponding to the lower 95 percent confidence limit of equation 4.19 is:

$$R = 100 [1 - 0.41\exp(-0.49\theta + 0.0085\theta^2)] \quad (4.20)$$

Equation 4.20 is recommended for use in design (or Table 4.7 which is based on it); it is applied sequentially to each pond in the series, so that the number of eggs in the final effluent can be determined. An example of how it is used for restricted irrigation is given in Design Example No. 2 in Annex I.

Table 4.7 Design values of percentage helminth egg removal (R) in individual anaerobic, facultative or maturation ponds for hydraulic retention times (θ) in the range 1–20 days (calculated from equation 4.20)

θ	R	θ	R	θ	R
1.0	74.67	4.0	93.38	9.0	99.01
1.2	76.95	4.2	93.66	9.5	99.16
1.4	79.01	4.4	93.40		
1.6	80.87	4.6	94.85	10	99.29
1.8	82.55	4.8	95.25	10.5	99.39
2.0	84.08	5.0	95.62	11	99.48
2.2	85.46	5.5	96.42	12	99.61
2.4	87.72			13	99.70
2.6	87.85	6.0	97.06	14	99.77
2.8	88.89	6.5	97.57	15	99.82
3.0	89.82	7.0	97.99	16	99.86
3.2	90.68	7.5	98.32	17	99.88
3.4	91.45			18	99.90
3.6	92.16	8.0	98.60	19	99.92
3.8	92.80	8.5	98.82	20	99.93

4.5.3 BOD removal

Maturation ponds are not normally designed for BOD removal, yet it is often necessary to be able to estimate the BOD of the final effluent. BOD removal in maturation ponds is very much slower than in anaerobic and facultative ponds, and it is therefore

appropriate to estimate the filtered BOD of the final effluent on the assumption of 90 percent cumulative removal in the anaerobic and facultative ponds and then 25 percent in each maturation pond for temperatures above 20°C (80% and 20% respectively, for temperatures below 20°C) (Mara and Pearson, 1987).

4.5.4 Nutrient removal

There are very few data on nitrogen and phosphorus removal in WSP in India. For design recourse has to be made to equations developed in North America and designers should realise that these equations may not accurately predict performance in India.

Nitrogen

Pano and Middlebrooks (1982) present equations for ammonical nitrogen ($\text{NH}_3 + \text{NH}_4^+$) removal in individual facultative and maturation ponds. Their equation for temperatures below 20°C is:

$$C_e = C_i / \{1 + [(A/Q)(0.0038 + 0.000134T) \exp((1.041 + 0.044T)(\text{pH} - 6.6))]\} \quad (4.21)$$

and for temperatures above 20°C:

$$C_e = C_i / \{1 + [5.035 \times 10^{-3} (A/Q) [\exp(1.540 \times (\text{pH} - 6.6))]]\} \quad (4.22)$$

where C_e = ammoniacal nitrogen concentration in pond effluent, mg N/l

C_i = ammoniacal nitrogen concentration in pond influent, mg N/l

A = pond area, m^2

Q = influent flow rate, m^3/d

Reed (1985) presents an equation for the removal of total nitrogen in individual facultative and maturation ponds:

$$C_e = C_i \exp\{-[0.0064(1.039)^{T-20}] [\theta + 60.6(\text{pH} - 6.6)]\} \quad (4.23)$$

where C_e = total nitrogen concentration in pond effluent, mg N/l

C_i = total nitrogen concentration in pond influent, mg N/l

T = temperature, °C (range: 1-28°C)

θ = retention time, d (range 5- 231 d)

The pH value used in equations 4.21- 4.23 may be estimated from:

$$\text{pH} = 7.3\exp(0.0005\mathbf{A}) \quad (4.24)$$

where \mathbf{A} = influent alkalinity, mg CaCO_3/l

Equations 4.21 – 4.23 are applied sequentially to individual facultative and maturation ponds in the series, so that concentrations in the effluent can be determined. Design Example No. 4 in Annex I shows how these equations are used in the design of a wastewater-fed fishpond system (see also Section 10.4.2).

Phosphorus

There are no design equations for phosphorus removal in WSP. Huang and Gloyna (1984) indicate that, if BOD removal in a pond system is 90 percent, the removal of total phosphorus is around 45 percent. Effluent total P is around two-thirds inorganic and one-third organic.

4.6 INITIAL PARTIAL TREATMENT

If the more pragmatic approach outlined in Section 4.1 is adopted, then wastewater treatment in only anaerobic and facultative ponds is to be considered – at least initially. This initial partial treatment of wastewater is very much preferable to no treatment, and it enables cities to spread out over time their investments in wastewater treatment.

It may be of interest, really as an aide to the approximate estimation of land area requirements, to calculate the areas per caput for anaerobic and facultative ponds for design temperatures of 15, 20 and 25°C.

4.6.1 Anaerobic ponds

Equation 4.2 can be rewritten as:

$$A_a = L_i Q / \lambda_v D \quad (4.25)$$

where A_a = anaerobic pond area, m^2/caput

$L_i Q$ = quantity of BOD, $\text{g}/\text{caput day}$

λ_v = volumetric BOD loading, $\text{g}/\text{m}^3/\text{day}$

D = anaerobic pond depth, m

Assuming $L_iQ = 45$ g/caput day (Section 4.2.3) and $D = 3$ m, equation 4.25 becomes:

$$A_a = 15/\lambda_v \quad (4.26)$$

Using the values of λ_v for 15, 20 and 25°C derived from Table 4.1, the corresponding values of A_a can be calculated, as given in Table 4.7.

4.6.2. Facultative ponds

Equation 4.4 can be restated as:

$$A_f = 10L_iQ/\lambda_s \quad (4.4)$$

where A_f = facultative pond area, m²/caput
 L_iQ = quantity of BOD, g/caput day
 λ_s = surface BOD loading, kg/ha day

L_iQ is now the quantity of BOD entering the facultative pond – i.e. account has to be taken of the BOD removed in the anaerobic pond. Thus equation 4.4 can be rewritten as:

$$A_f = 450 a/\lambda_s \quad (4.27)$$

where $a = 0.5$ for 15°, 0.4 for 20° and 0.3 for 25°C (see Table 4.1).

Thus using the values of λ_s given in Table 4.4 for 15, 20 and 25°C, the corresponding values of A_f can be determined, as given in Table 4.8.

Table 4.8 Land area requirements per person for partial treatment in anaerobic and facultative ponds at 15, 20 and 25°C

	Land area, m ² per caput		
	15°C	20°C	25°C
Anaerobic pond	0.075	0.050	0.043
Facultative pond	1.345	0.712	0.386
Total pond area	1.42	0.76	0.43
Overall area ^a	1.78	0.95	0.54

^a Overall area = total pond area × 1.25 (see text).

The total pond areas given by the sum of equations 4.26 and 4.27 need to be multiplied by a factor of around 1.25 to give an estimate of the total land area requirement for this degree of initial partial treatment. (This factor is used to take into account the land area occupied by embankments and access roads. Its value varies with the size of the scheme: 1.25 is suitable for large systems, but a value of 1.5 may be more appropriate for small systems.)

Note

It is sometimes asked what is the lowest concentration of BOD at which WSP can operate. Generally speaking, WSP can operate satisfactorily at any level of BOD, although it is worth noting the following three points:

- (a) as noted in section 4.3, anaerobic ponds should have a minimum retention time of 1 day; however, if the resulting volumetric BOD loading is $<30 \text{ g/m}^3\text{d}$, then anaerobic ponds should not be used as there is essentially no experience of their satisfactory performance at lower loadings;
- (b) as noted in section 4.4, facultative ponds should have a minimum retention time of 4 days at design temperatures above 20°C and 5 days at lower temperatures; the resulting BOD loading may be much less than that permitted by equation 4.8 and Table 4.4 if the wastewater BOD is very low, but this does not matter – the algal population will adjust accordingly and the nominally facultative pond will function algologically more as a maturation pond, but treatment efficiency will not be seriously impeded; and
- (c) if the wastewater BOD is below, or only slightly above, the CPCB effluent discharge standard of 30 mg/l (which might be due to excessive infiltration in the sewer system, for example), then probably no treatment would be required.

5.

Physical design of WSP

The process design prepared as described in Section 4 must be translated into a physical design. Actual pond dimensions, consistent with the available site, must be calculated; embankments and pond inlet and outlet structures must be designed and decisions taken regarding preliminary treatment, parallel pond systems and whether or not to line the ponds. By-pass pipes, security fencing and notices are generally required, and operator facilities must be provided.

The physical design of WSP must be carefully done: it is at least as important as process design and can significantly affect treatment efficiency.

5.1 POND LOCATION

Ponds should be located at least 200 m (preferably 500 m) downwind from the community they serve and away from any likely area of future expansion. This is mainly to discourage people from visiting the ponds (see Section 5.9). Odour release, even from anaerobic ponds, is most unlikely to be a problem in a well-designed and properly maintained system, but the public may need assurance about this at the planning stage, and a minimum distance of 200 m normally allays any fears.

There should be vehicular access to the ponds and, so as to minimise earthworks, the site should be flat or gently sloping. The soil must also be suitable (see Section 5.2). Ponds should not be located within 2 km of airports, as any birds attracted to the ponds may constitute a risk to air navigation.

5.2 GEOTECHNICAL CONSIDERATIONS

Geotechnical aspects of WSP design are very important. In Europe, for example, half of the WSP systems that malfunction do so because of geotechnical problems which could have been avoided at the design stage.

The principal objectives of a geotechnical investigation are to ensure correct embankment design and to determine whether the soil is sufficiently permeable to require the pond to be lined. The maximum height of the groundwater table should be determined, and the following properties of the soil at the proposed pond location must be measured:

- (a) particle size distribution;
- (b) maximum dry density and optimum moisture content (modified Proctor test);
- (c) Atterberg limits;
- (d) organic content; and
- (e) coefficient of permeability.

At least four soil samples should be taken per hectare, and they should be as undisturbed as possible. The samples should be representative of the soil profile to a depth 1 m greater than the envisaged pond depth.

Organic, for example peaty and plastic soils and medium-to-coarse sands, are not suitable for embankment construction. If there is no suitable local soil with which at least a stable and impermeable embankment core can be formed, it must be brought to the site at extra cost and the local soil, if suitable, used for the embankment slopes.

Ideally, embankments should be constructed from the soil excavated from the site, and there should be a balance between cut and fill, although it is worth noting that ponds constructed completely in cut may be a cheaper alternative, especially if embankment construction costs are high. The soil used for embankment construction should be compacted in 150-250 mm layers to 90% of the maximum dry density as determined by the modified Proctor test. Shrinkage of the soil occurs during compaction (10-30 percent) and excavation estimates must take this into account. After compaction, the soil should have a coefficient of permeability, as determined *in situ*, of $<10^{-7}$ m/s (see Section 5.3). Wherever possible, and particularly at large pond

installations, embankment design should allow for vehicle access to facilitate maintenance.

Embankment slopes are commonly 1 to 3 internally and 1 to 1.5-2 externally. Steeper slopes may be used if the soil is suitable; slope stability should be ascertained according to standard soil mechanics procedures for small earth dams. Embankments should be planted with grass to increase stability: a slow-growing rhizomatous species should be used to minimise maintenance (see Section 6.2).

External embankments should be protected from stormwater erosion by providing adequate drainage. Internal embankments require protection against erosion by wave action, and this is best achieved by precast concrete slabs (Figure 5.1) or stone rip-rap (Figure 5.2) at top water level. Such protection also prevents vegetation from growing down the embankment into the pond, so preventing the development of a suitable habitat for mosquito or snail breeding.

5.3 HYDRAULIC BALANCE

To maintain the liquid level in the ponds, the inflow must be at least greater than net evaporation and seepage at all times. Thus:

$$Q_i \geq 0.001A(e + s) \quad (5.1)$$

where Q_i = inflow to first pond, m^3/d
 A = total area of pond series, m^2
 e = net evaporation (i.e. evaporation less rainfall), mm/d
 s = seepage, mm/d

Seepage losses must be at least smaller than the inflow less net evaporation so as to maintain the water level in the pond. The maximum permissible permeability of the soil layer making up the pond base can be determined from d'Arcy's law:

$$k = [Q_s / (86,400A)] [\Delta l / \Delta h] \quad (5.2)$$

where k = maximum permissible permeability, m/s
 Q_s = maximum permissible seepage flow
 (= $Q_i - 0.001Ae$), m^3/d
 A = base area of pond, m^2
 Δl = depth of soil layer below pond base to aquifer or more permeable stratum, m
 Δh = hydraulic head (= pond depth + Δl), m



Figure 5.1 Embankment protection by precast concrete slabs laid at top water level.



Figure 5.2 Embankment protection by stone rip-rap.

If the permeability of the soil is more than the maximum permissible, the pond must be lined. A variety of lining materials is available and local costs dictate which should be used. Satisfactory lining has been achieved with ordinary portland cement (8 kg/m²), plastic membranes (Figures 5.3 and 5.4) and 150-300 mm layers of low-permeability soil. As a general guide, the following interpretations may be placed on values obtained for the *in situ* coefficient of permeability:

$k > 10^{-6}$ m/s: the soil is too permeable and the ponds must be lined;

$k > 10^{-7}$ m/s: some seepage may occur but not sufficiently to prevent the ponds from filling;

$k < 10^{-8}$ m/s: the ponds will seal naturally;

$k < 10^{-9}$ m/s: there is no risk of groundwater contamination (if $k > 10^{-9}$ m/s and the groundwater is used for potable supplies, further detailed hydrogeological studies may be required).



Figure 5.3 Anaerobic pond lined with an impermeable plastic membrane.

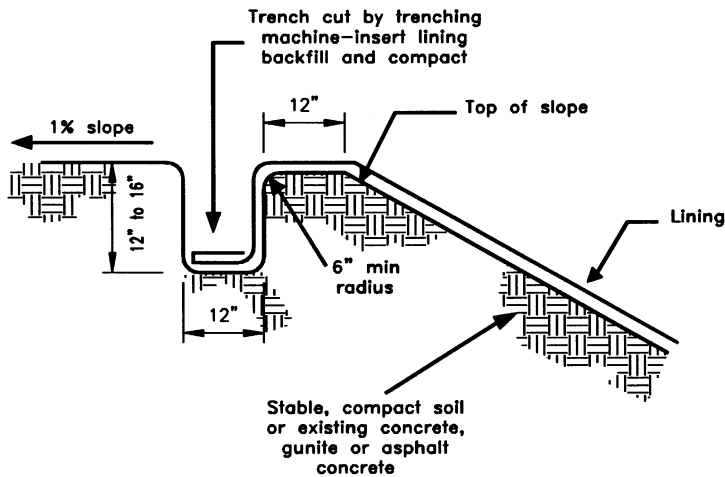


Figure 5.4
Anchoring of pond
lining at top of
embankment
(EPA, 1983).

5.4 PRELIMINARY TREATMENT

Adequate screening and grit removal facilities must be installed at all but very small systems (those serving <1000 people). Design should follow standard procedures (for example, IWEM, 1992; Marais, 1971; Marais and van Haandel, 1996; Metcalf & Eddy, Inc., 1991). Adequate provision must be made for the hygienic disposal of screenings and grit; haulage to a sanitary landfill or on-site burial in trenches are usually the most appropriate method.

Wastewater flows up to 6 times dry weather flow should be subjected to screening and grit removal. Any flows in excess of 6 DWF should be discharged via a stormwater overflow to a receiving watercourse. Anaerobic ponds should not receive more than 3 DWF, in order to prevent washout of acidogens and methanogens; so excess flows between 3 and 6 DWF are diverted via an overflow weir to the facultative ponds.

After screening and grit removal and, if installed, the >6 DWF overflow weir, the wastewater flow should be measured in a standard Venturi or Parshall flume. This is essential in order to assess pond performance (Section 7). Flow-recording devices may be installed, but these require careful calibration and regular maintenance. Often it is better to read the upstream channel depth from a calibrated brass rule and then calculate the flow from standard flume formulae (see IWEM, 1992; Metcalf & Eddy, Inc., 1991).

5.5 POND GEOMETRY

There has been little rigorous work done on determining optimal pond shapes. The most common shape is rectangular, although there is much variation in the length-to-breadth ratio. Clearly, the optimal pond geometry, which includes not only the shape of the pond but also the relative positions of its inlet and outlet, is that which minimises hydraulic short-circuiting.

In general, anaerobic and primary facultative ponds should be rectangular, with length-to-breadth ratios of 2 – 3 to 1 so as to avoid sludge banks forming near the inlet. Secondary facultative and maturation ponds should, wherever possible, have higher length-to-breadth ratios (up to 10 to 1) so that they better approximate plug flow conditions. Ponds do not need to be strictly rectangular, but may be gently curved if necessary or if desired for aesthetic reasons. A single inlet and outlet are usually sufficient, and these should be located just away from the base of the embankment in diagonally opposite corners of the pond (the inlet should *not* discharge centrally in the pond as this maximises hydraulic short-circuiting). The use of complicated multi-inlet and multi-outlet designs is unnecessary and not recommended.

To facilitate wind-induced mixing of the pond surface layers, the pond should be located so that its longest dimension (diagonal) lies in the direction of the prevailing wind. If this is seasonally variable, the wind direction in the hot season should be used as this is when thermal stratification is at its greatest. To minimise hydraulic short-circuiting, the inlet should be located such that the wastewater flows in the pond against the wind.

The areas calculated by the process design procedure described in Section 4 are mid-depth areas, and the dimensions calculated from them are thus mid-depth dimensions. These need to be corrected for the slope of the embankment, as shown in Figure 5.5.

A more precise method is advisable for anaerobic ponds, as these are relatively small. The following formula is used (EPA, 1983):

$$V_a = [(LW) + (L-2sD)(W-2sD) + 4(L-sD)(W-sD)] [D/6] \quad (5.3)$$

where V_a = anaerobic pond volume, m^3

L = pond length at TWL, m

W = pond width at TWL, m

s = horizontal slope factor (i.e. a slope of 1 in s)

D = pond liquid depth, m

With the substitution of L as nW , based on a length to breadth ratio of n to 1, equation 5.3 becomes a simple quadratic in W .

The dimensions and levels that the contractor needs to know are those of the base and the top of the embankment; the latter includes the effect of the freeboard. The minimum freeboard that should be provided is decided on the basis of preventing waves, induced by the wind, from overtopping the embankment. For small ponds (under 1 ha in area) 0.5 m freeboard should be provided; for ponds between 1 ha and 3 ha, the freeboard should be 0.5-1 m, depending on site considerations. For larger ponds, the freeboard may be calculated from the equation (Oswald, 1975):

$$F = (\log_{10}A)^{1/2} - 1 \quad (5.4)$$

where F = freeboard, m

A = pond area at TWL, m^2

Pond liquid depths are commonly in the following ranges:

- anaerobic ponds: 2-5 m
- facultative ponds: 1-2 m
- maturation ponds: 1-1.5 m

The depth chosen for any particular pond depends on site considerations (presence of shallow rock, minimisation of earthworks). The depth of facultative and maturation ponds should not be less than 1 m so as to avoid vegetation growing up from the pond base, with the consequent hazard of mosquito and snail breeding.

At WSP systems serving more than around 10,000 people, it is often sensible (so as to increase operational flexibility) to have two or more series of ponds in parallel. The available site

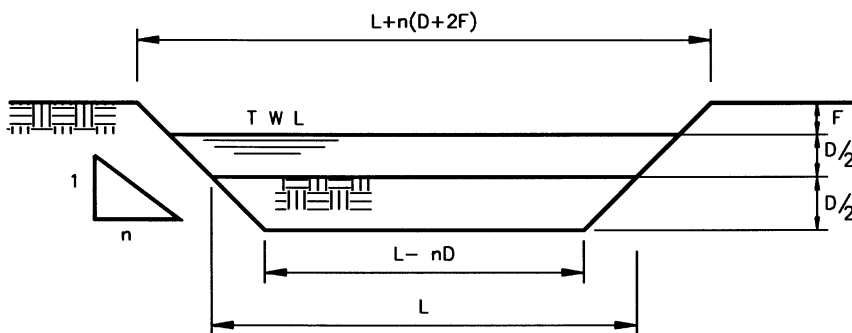


Figure 5.5 Calculation of top and bottom pond dimensions from those based on mid-depth.

topography may in any case necessitate such a subdivision, even for smaller systems. Usually the series are equal, that is to say they receive the same flow, and arrangements for splitting the raw wastewater flow into equal parts after preliminary treatment must be made (see Stalzer and von der Emde, 1972). This is best done by providing weir penstocks ahead of each series.

5.6 INLET AND OUTLET STRUCTURES

There is a wide variety of designs for inlet and outlet structures, and provided they follow certain basic concepts, their precise design is relatively unimportant. Firstly, they should be simple and inexpensive; while this should be self-evident, it is all too common to see unnecessarily complex and expensive structures. Secondly, they should permit samples of the pond effluent to be taken with ease. The inlet to anaerobic and primary facultative ponds should discharge well below the liquid level so as to minimise short-circuiting (especially in deep anaerobic ponds) and thus reduce the quantity of scum (which is important in facultative ponds). Inlets to secondary facultative and maturation ponds should also discharge below the liquid level, preferably at mid-depth in order to reduce the possibility of short-circuiting. Some simple inlet designs are shown in Figures 5.6 and 5.7.

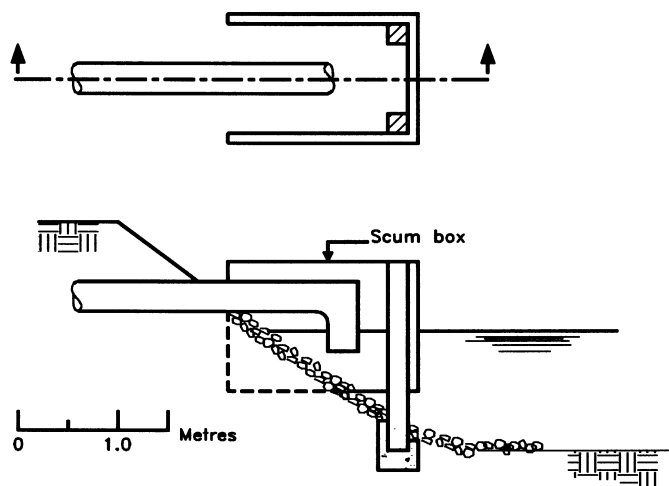


Figure 5.6 Inlet structure for anaerobic and primary facultative ponds. The scum box retains most of the floating solids, so easing pond maintenance (ALTB/CTGREF, 1979).

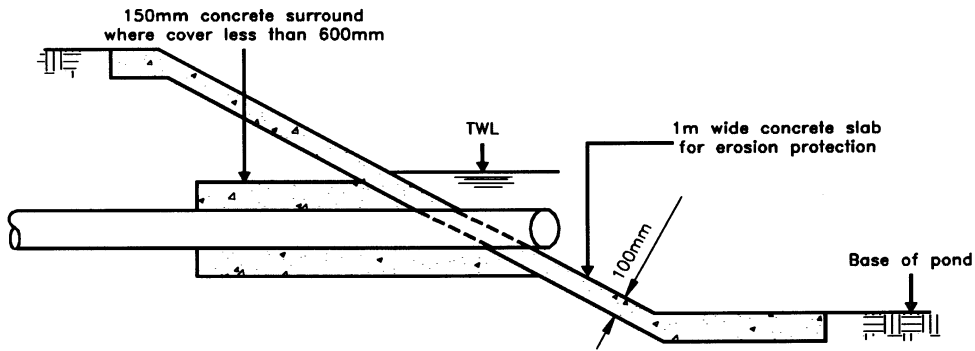


Figure 5.7 Inlet structure for secondary facultative and maturation ponds. This would receive the discharge from the outlet structure shown in Figure 5.8.

The outlet of all ponds should be protected against the discharge of scum by the provision of a scum guard. The take-off level for the effluent, which is controlled by the scum guard depth, is important as it has a significant influence on effluent quality. In facultative ponds, the scum guard should extend just below the maximum depth of the algal band when the pond is stratified so as to minimize the daily quantity of algae, and hence BOD, leaving the pond. In anaerobic and maturation ponds, where algal banding is irrelevant, the take-off should be nearer the surface: in anaerobic ponds it should be well above the maximum depth of sludge but below any surface crust, and in maturation ponds it should be at the level that gives the best possible microbiological quality. The following effluent take-off levels are recommended:

anaerobic ponds:	300 mm
facultative ponds:	600 mm
maturation ponds:	50 mm

The installation of a variable height scum guard is recommended, since it permits the optimal take-off level to be set once the pond is operating.

A simple outlet weir structure is shown in Figure 5.8. The following formula should be used to determine the head over the weir and so, knowing the pond depth, the required height of the weir above the pond base can be calculated:

$$q = 0.0567 h^{3/2} \quad (5.5)$$

where q = flow per metre length of weir, l/s
 h = head of water above weir, mm

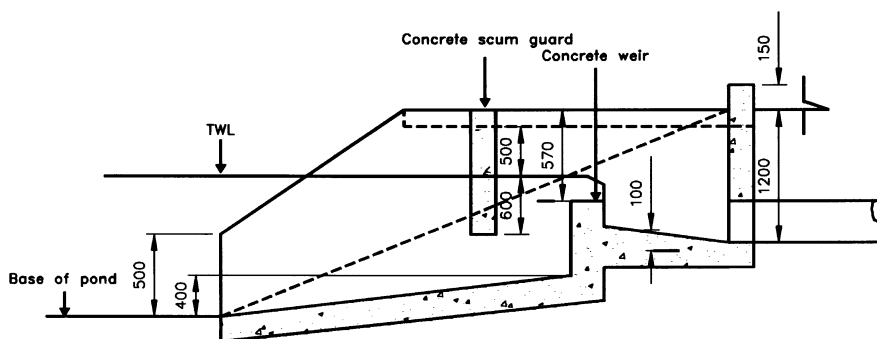


Figure 5.8 Outlet weir structure. The weir length is calculated from equation 5.5. The discharge pipe would connect with the inlet structure shown in Figure 5.7. The concrete scum guard depth should be as described in Section 5.6 (here it is 600 mm, suitable for facultative ponds). As an alternative a variable-depth wooden scum guard may be used.

The outlet from the final pond in a series should discharge into a simple flow-measuring device such as a triangular or rectangular notch. Since the flow into the first pond is also measured, this permits the rate of evaporation and seepage to be calculated or, if evaporation is measured separately, the rate of seepage.

5.7 BY-PASS PIPEWORK

It is necessary to bypass anaerobic ponds so that facultative ponds may be commissioned first (see Section 6.1) and also during desludging operations (Section 6.3). Figure 5.9 shows schematically a by-pass arrangement for two series of WSP in parallel.

5.8 RECIRCULATION

If the incoming raw wastewater is septic, it may be necessary to achieve odour control by recirculating up to 50 percent of the final effluent. This should be pumped back and mixed with the raw wastewater immediately after preliminary treatment (i.e. before the wastewater enters the first pond). The final effluent acts to oxygenate the septic wastewater, and it may help to increase BOD removal. The process design of the ponds has to be altered to allow for the recirculated flow, and clearly recirculation, with its attendant problems of pump O&M, should only be considered as a measure of the last resort.

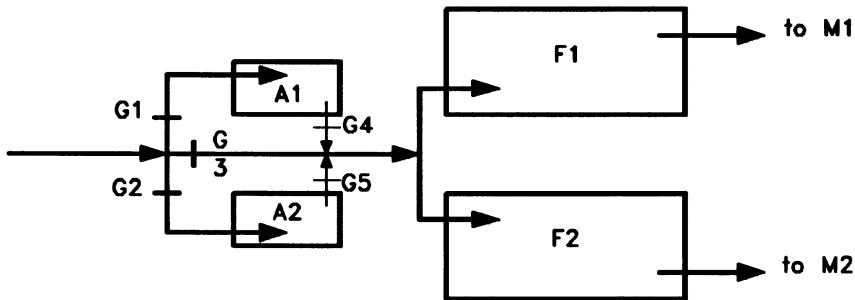


Figure 5.9 By-pass pipework for anaerobic ponds. During normal operation gates G1 and G3 are open and gates G2 and G4 closed. To by-pass one (or both) of the anaerobic pond(s) gates G1 and G3 are closed and gate G2 and one (or both) of gates G4 open.

5.9 TREEBELT

In desert areas a treebelt should be provided to prevent wind-blown sand from being deposited in the ponds. Treebelts may also be desired for aesthetic reasons if the WSP site is close to human habitation. They should be planted upwind of the WSP and comprise up to five rows, as follows (from the upwind side):

- (a) 1-2 rows of mixed shrubs (<5 m) such as *Acacia bivenosa*, *Zizphus spina-christi*, *Hibiscus* and *Nerium oleander*, none of which is eaten by goats;
- (b) 1-2 rows of 5-15 m tall trees such as *Acacia salicina*, *Cassia siamea*, *Sesbania grandiflora* and *Zizyphus mauritania*; and
- (c) 1 row of mixture of taller (>15 m) trees such as *Acacia mearnsii*, *Albizia lebbek*, *Casuarina equisetifolia*, *Casuarina cristata* and *Terminalia catappa*.

Local botanists will be able to advise on which species are most appropriate; those given above are suitable for use in northwest India. Such a treebelt is around 40-60 m wide.

5.10 SECURITY

Ponds (other than very remote installations) should be surrounded by a chain-link fence and gates should be kept padlocked.

Warning notices, in English, Hindi and the appropriate local language(s), attached to the fence and advising that the ponds are a wastewater treatment facility, and therefore potentially hazardous to health, are essential to discourage people from visiting the ponds, which if properly maintained (see Section 6) should appear as pleasant, inviting bodies of water. Children are especially at risk, as they may be tempted to swim in the ponds. Birdwatchers and hunters are also attracted to ponds by the often rich variety of wildlife, and they may not be aware that the ponds are treating wastewater.

5.11 OPERATOR FACILITIES

The facilities to be provided for the team of pond operators depend partly on their number (see Section 6.3), but would normally include the following:

- (a) first-aid kit (which should include a snake bite kit);
- (b) strategically placed lifebuoys;
- (c) wash-basin and toilet; and
- (d) storage space for protective clothing, grass-cutting and scum-removal equipment, screen rakes and other tools, sampling boat (if provided) and life-jackets.

With the exception of the lifebuoys, these can be accommodated in a simple building. This can also house, if required, sample bottles and a refrigerator for sample storage. Laboratory facilities, offices and a telephone may also be provided at large installations. Adequate space for car parking should be provided.

At very large WSP sites consideration should also be given to providing housing for the relatively large number of operators employed.

6.

Operation and maintenance

6.1 START-UP PROCEDURES

Pond systems should preferably be commissioned at the beginning of the hot season so as to establish as quickly as possible the necessary microbial populations to effect waste stabilization. Prior to commissioning, all ponds must be free from vegetation. Facultative ponds should be commissioned before anaerobic ponds: this avoids odour release when anaerobic pond effluent discharges into an empty facultative pond. It is best to fill facultative and maturation ponds with freshwater (from a river, lake or well; mains water is not necessary) so as to permit the gradual development of the algal and heterotrophic bacterial populations. Primary facultative ponds may advantageously be seeded in the same way as anaerobic ponds (see below). If freshwater is unavailable, facultative ponds should be filled with raw sewage and left for three to four weeks to allow the microbial population to develop; a small amount of odour release is inevitable during the period.

Anaerobic ponds should be filled with raw sewage and seeded, where possible, with *digesting* sludge from, for example, an anaerobic digester at a conventional sewage treatment works or with sludge from local septic tanks. The ponds should then be gradually loaded up to the design loading rate over the following week (or month if the ponds are not seeded). Care should be taken to maintain the pond pH above 7 to permit the development of methanogenic bacteria, and it may be necessary during the first month or so to dose the pond with lime or soda ash. If, due to an initially low rate of sewer connections in newly sewered towns

POND MAINTENANCE RECORD SHEET

Pond location:

Date and Time:..... Air temperature:.....°C

Weather conditions:

Pumping station (if there is one):

- elapsed time meter reading: No. 1..... No. 2.....
- electricity meter reading:
- observations: (flooding)

.....

Access road: state (vegetation, damage) maintenance carried out

.....

Pond site: state; maintenance carried out

.....

Pretreatment works: state; maintenance carried out

- screen(s):.....
- other (grit, grease removal):.....

.....

VISUAL INSPECTION OF PONDS

POND NUMBER	1	2	3	OBSERVATIONS
<u>Colour of water</u> (green, brown/grey, pink/red, milky/clear)				
<u>Odour</u>				
<u>Scum, foam</u>				
<u>Rooted macrophytes</u>				
<u>State of embankments</u> (erosion, rodent damage, vegetation)				
<u>Inlet and outlet</u> (blockage)				
<u>Water level</u> (high, normal, low)				
GENERAL OBSERVATIONS, other maintenance carried out:				

Figure 6.1 Example of a routine pond maintenance record sheet (CEMAGREF, 1985).

the sewage is weak or its flow low, it is best to by-pass the anaerobic ponds until the sewage strength and flow is such that a loading of at least $50 \text{ g/m}^3 \text{ d}$ can be applied to them. (It is also necessary to by-pass an anaerobic pond whilst it is being desludged (Section 6.4), so the by-pass should be a permanent facility: see Section 5.7).

6.2 ROUTINE MAINTENANCE

The maintenance requirements of ponds are very simple, but they must be carried out regularly. Otherwise, there will be serious odour, fly and mosquito nuisance. Maintenance requirements and responsibilities must therefore be clearly defined at the design stage so as to avoid problems later. Routine maintenance tasks are as follows:

- (a) removal of screenings and grit from the inlet works;
- (b) cutting the grass on the embankments and removing it so that it does not fall into the pond (this is necessary to prevent the formation of mosquito-breeding habitats; the use of slow-growing grasses minimises this task – see Section 5.2).
- (c) removal of floating scum and floating macrophytes, e.g. *Lemna*, from the surface of facultative and maturation ponds (this is required to maximize photosynthesis and surface re-aeration and prevent fly and mosquito breeding);
- (d) spraying the scum on anaerobic ponds (which should not be removed as it aids the treatment process), as necessary, with clean water or pond effluent, or a suitable biodegradable larvicide, to prevent fly breeding;
- (e) removal of any accumulated solids in the inlets and outlets;
- (f) repair of any damage to the embankments caused by rodents, rabbits or other animals; and
- (g) repair of any damage to external fences and gates.

The operators must be given precise instructions on the frequency at which these tasks should be done, and their work must be constantly supervised. The supervisor/ foreman should be required to complete at weekly intervals a pond maintenance record sheet, an example of which is given in Figure 6.1. The operators may also be required to take samples and do some routine measurements (see Section 7).

6.3 STAFFING LEVELS

In order that the routine O&M tasks can be properly done, WSP installations must be adequately staffed. The level of staffing depends on the type of inlet works (for example, mechanically raked screens and proprietary grit removal units require an electromechanical technician, but manually raked screens and manually cleaned grit channels do not), whether there are on-site laboratory facilities, and how the grass is cut (manually or by mechanical mowers). Recommended staffing levels are given in Table 6.1 for WSP systems serving populations up to 250,000; for larger systems the number of staff should be increased *pro rata*.

Table 6.1 Recommended staffing levels for WSP systems

Population Served	10,000	25,000	50,000	100,000	250,000
Foreman/ Supervisor	-	-	1	1	1
Mechanical engineer ^a	-	-	-	1	1
Laboratory technician ^b	-	1	1	1	2
Assistant foreman	-	1	2	2	2
Labourers	1	2	4	6	10
Driver ^c	-	1	1	1	2
Watchman ^d	1	1	3	5	5
Total	2	6	10	15	23

^a Dependent upon amount of mechanical equipment used.

^b Dependent upon existence of laboratory facilities.

^c Dependent upon use of vehicle-towed lawn mowers, etc.

^d Dependent upon location and amount of equipment used.

Source: Arthur (1983).

6.4 DESLUDGING AND SLUDGE DISPOSAL

Anaerobic ponds require desludging when they are one third full of sludge (by volume). This occurs every n years where n is given by:

$$n = V_a/3Ps \quad (6.1)$$

where V_a = volume of anaerobic pond, m³

P = population served

s = sludge accumulation rate, m³/caput year



Figure 6.2 Pond desludging using a raft-mounted sludge pump. Detail: sludge suction head.

The usual design value of s is $0.04\text{m}^3/\text{caput year}$. Thus, for temperatures above 20°C ($\lambda_v = 300 \text{ g/m}^3\text{d}$) and a BOD contribution of 45 g/person d , desludging would be required annually ($n = 1.25$ years). The precise requirement for desludging can be determined by the “white towel” test (Section 7.2), but it should be borne in mind that a task to be done annually has more chance of being done on time than one to be done at less regular intervals.

Sludge removal can be readily achieved by using a raft-mounted sludge pump. These are commercially available (e.g. Brain Associates Ltd., Kilgetty, Dyfed SA68 0UJ, UK); or they can be assembled locally (Figure 6.2 shows one such unit being used on a primary facultative pond in France). The sludge is discharged into either an adjacent sludge lagoon or tankers to transport it to a landfill site, agricultural land or other suitable disposal location. Although pond sludge has a better microbiological quality than that from conventional treatment works, its disposal must be carried out in accordance with any local regulations governing sludge disposal.

7.

Monitoring and evaluation

Once a WSP system has been commissioned, a routine monitoring programme should be established so that the actual quality of its effluent can be determined.

Routine monitoring of the final effluent quality of a pond system permits a regular assessment to be made of whether the effluent is complying with the local discharge or reuse standards. Moreover, should a pond system suddenly fail or its effluent start to deteriorate, the results of such a monitoring programme often give some insight into the cause of the problem and generally indicate what remedial action is required.

The evaluation of pond performance and behaviour, although a much more complex procedure than the routine monitoring of effluent quality, is nonetheless extremely useful as it provides information on how underloaded or overloaded the system is, and thus by how much, if any, the loading on the system can be safely increased as the community it serves expands, or whether further ponds (in parallel or in series) are required (see Section 8.2). It also indicates how the design of future pond installations in the region might be improved to take account of local conditions.

7.1. EFFLUENT QUALITY MONITORING

Effluent quality monitoring programmes should be simple, but should none-the-less provide reliable data. Two levels of effluent monitoring are recommended (reference should also be made to the routine pond maintenance record sheets to be completed by the pond supervisor – see Section 6.2 and Figure 6.1):

- a) *Level 1*: representative samples of the final effluent should be taken at least monthly intervals; they should be analysed for those parameters for which effluent discharge or reuse requirements exist;
- b) *Level 2*: when level 1 monitoring shows that a pond effluent is failing to meet its discharge or reuse quality, a more detailed study is necessary. Table 7.1 gives a list of parameters whose values are required, together with directions on how they should be obtained.

Table 7.1 Parameters to be determined in a "Level 2" effluent quality monitoring programme

Parameter	Sample type ^a	Remarks
Flow	-	Measure both raw wastewater and final effluent flows
BOD	C	Unfiltered samples ^b
COD	C	Unfiltered samples ^b
Suspended solids	C	
Ammonia	C	
pH	G)	Take two samples, one at 08.00-10.00 h
Temperature	G)	and the other at 14.00-16.00 h
Faecal coliforms	G	Take sample between 08.00 and 10.00 h
Total nitrogen	C)	Only when effluent being used (or
Total phosphorus	C)	being assessed for use) for crop
Chloride	C)	irrigation. Ca, Mg and Na are
Electrical conductivity	C)	required to calculate the sodium
Ca, Mg, Na	C)	absorption ratio ^d
Boron	C)	
Helminth eggs ^c	C)	

^a C = 24-hour flow-weighted composite sample; G = grab sample.

^b Also on filtered samples if the discharge requirements are so expressed.

^c *Ascaris lumbricoides*, *Trichuris trichiura*, *Ancylostoma duodenale* and *Necator americanus*.

^d SAR = $(0.044\text{Na})/[0.5(0.050\text{Ca} + 0.082\text{Mg})]^{0.5}$ where Na, Ca and Mg are the concentrations in mg/l.

Since pond effluent quality shows a significant diurnal variation (although this is less pronounced in maturation ponds than in facultative ponds), 24-hour flow-weighted composite

samples are preferable for most parameters, although grab samples are necessary for some (pH, temperature and faecal coliforms). Composite samples should be collected in one of the following ways:

- a) in an automatic sampler, which takes grab samples every one or two hours, with subsequent manual flow-weighting if this is not done automatically by the sampler;
- b) by taking grab samples every one to three hours with subsequent manual flow-weighting; or
- c) by taking a column sample (see Section 7.2) near the outlet of the final pond; this can be done at any time of day and gives a good approximation to the mean daily effluent quality (Pearson *et al.*, 1987b)

7.2 EVALUATION OF POND PERFORMANCE

A full evaluation of the performance of a WSP system is a time-consuming and expensive process, and it requires experienced personnel to interpret the data obtained. It is in many ways close to research, but it is the only means by which pond designs can be optimised for local conditions. It is often therefore a highly cost-effective exercise. The recommendations given below constitute a level 3 monitoring programme, and they are based on the guidelines for the minimum evaluation of pond performance given in Pearson *et al.* (1987a), which should be consulted for further details.

It is not intended that all pond installations be studied in this way, but only one or two representative systems in each major climatic region. This level of investigation is most likely to be beyond the capabilities of local organizations, and it would need to be carried out by a state or national body, or by a university under contract to such a body. This type of study is also necessary when it is required to know how much additional loading a particular system can receive before it is necessary to extend it.

Samples should be taken and analysed on at least five days over a five-week period at both the hottest and coldest times of the year. Samples are required of the raw wastewater and of the effluent of each pond in the series and, so as to take into account most of the weekly variation in influent and effluent quality, samples should be collected on Monday in the first week, Tuesday in the second week and so on (local factors, such as a high influx of visitors at

weekends, may influence the choice of days on which samples are collected). Table 7.2 lists the parameters whose values are required. Generally the analytical techniques described in the current edition of *Standard Methods* (APHA, 1995) are recommended, although the procedures detailed in Annex II should be followed for chlorophyll *a*, algal genera and sulphide. The modified Bailenger technique should be used for counting the number of helminth eggs (Ayres and Mara, 1996). Faecal coliforms should be counted by the methods detailed in *Report 71* (HMSO, 1994; see also Ayres and Mara, 1996); alternatively, the procedures detailed in ISI (1982) may be followed.

Composite samples, collected as described in Section 7.1, are necessary for most parameters; grab samples are required for pH and faecal coliforms; and samples of the entire pond water column should be taken for algological analyses (chlorophyll *a* and algal genera determination), using the pond column sampler shown in Figure 7.1. Pond column samples should be taken from a boat or from a simple sampling platform (or the outlet structure) that extends beyond the embankment base. Data on at least maximum and minimum air temperatures, rainfall and evaporation should be obtained from the nearest meteorological station.

On each day that samples are taken, the mean mid-depth temperature of each pond, which closely approximates the mean daily pond temperature, should be determined by suspending a maximum-and-minimum thermometer at mid-depth of the pond at 08.00-09.00 h and reading it 24 hours later.

On one day during each sampling period, the depth of sludge in the anaerobic and facultative ponds should be determined, using the "white towel" test of Malan (1964). White towelling material is wrapped along one third of a sufficiently long pole, which is then lowered vertically into the pond until it reaches the pond bottom; it is then slowly withdrawn. The depth of the sludge layer is clearly visible since some sludge particles will have been entrapped in the towelling material (Figure 7.2). The sludge depth should be measured at least five points in the pond, away from the embankment base, and the mean depth calculated.

It is also useful to measure on at least three occasions during each sampling season the diurnal variation in the vertical distribution of pH, dissolved oxygen and temperature. Profiles should be obtained at 08.00, 12.00 and 16.00 h. If submersible electrodes are not available, samples should be taken manually every 20 cm.

Table 7.2 Parameters to be determined for minimum evaluation of pond performance

Parameter	To be determined for ^a	Type of sample ^b	Remarks
Flow	RW, FE	-	
BOD	RW, all pond effluents ^c	C	Unfiltered and filtered samples
COD	RW, all pond effluents	C	Unfiltered and filtered samples
Suspended solids	RW, all pond effluents ^c	C	
Faecal coliforms	RW, all pond effluents	G	
Chlorophyll <i>a</i>	All F and M pond contents	P	
Algal genera	All F and M pond contents	P	
Ammonia	RW, all pond effluents ^c	C	
Nitrate	RW, FE	C	
Total phosphorus	RW, FE	C	
Sulphide	RW, A pond effluent, F pond contents or depth profile	G,P	Only if odour nuisance present or facultative pond effluent quality poor. A depth profile is preferable
pH	RW, all pond effluents	G	
Temperature (mean daily)	RW, all pond effluents	-	Use maximum-minimum thermometers suspended in RW flow and at mid-depth in ponds
Dissolved oxygen ^d	Depth profile in all F and M ponds	-	Measure at 08.00, 12.00 and 16.00 h on at least three occasions
Sludge depth	A and F ponds	-	Use "white towel" test (see text)
Electrical conductivity	FE	C)	
Chloride	RW, FE	C)	
Ca, Mg and Na	FE	C)	Only if effluent used or to be used for crop irrigation. Ca, Mg and Na required to calculate the sodium absorption ratio ^f
Boron	FE	C)	
Helminth eggs ^e	RW, all pond effluents	C)	

^a RW = raw wastewater; FE = final effluent of pond series; A = anaerobic; F = facultative; M = maturation.

^b C = 24-hour flow-weighted composite sample; G = grab sample taken when pond contents most homogeneous; P = pond column sample.

^c Alternatively RW, A, F and final M pond effluents only, if there are more than two maturation ponds.

^d Measure depth profiles of pH and temperature at same times, if possible.

^e *Ascaris lumbricoides*, *Trichuris trichiura*, *Ancylostoma duodenale* and *Necator americanus*.

^f SAR = $(0.044 \text{ Na})/[0.5 (0.050 \text{ Ca} + 0.082 \text{ Mg})]^{0.5}$ where Na, Ca and Mg are the concentrations in mg/l.



Figure 7.2 The “white towel” test for measuring sludge depth.

7.3 DATA STORAGE AND ANALYSIS

It is advisable to store all data in a microcomputer using a spreadsheet such as EXCEL, so that simple data manipulation can be performed. From the data collected in each sampling season (or month if sampling is done throughout the year), mean values should be calculated for each parameter. Values, based on these means, can then be calculated for:

- (a) hydraulic retention times (= volume/ flow) in each pond;
- (b) volumetric BOD and COD loadings on anaerobic ponds;
- (c) surface BOD and COD loading on facultative ponds; and
- (d) percentage removals of BOD, COD, suspended solids, ammonical nitrogen, total phosphorus, faecal coliforms and helminth eggs in each pond and in each series of ponds.

A simple kinetic analysis, based on (for example) a first order reaction in a completely mixed or plug flow reactor (for length to breadth ratios less or greater than 4 respectively) may be attempted if desired (see Mara, 1976). The responsible local or State governmental agency should record and store all the information and data collected from each pond complex, together with an adequate description of precisely how they were obtained, in such a way that design engineers and research workers can have ready access to them. It would also be sensible for such reports to be deposited with the National River Conservation Directorate in New Delhi and in the library of the National Environmental Engineering Research Institute in Nagpur.

8.

Rehabilitation and upgrading

8.1 REHABILITATION

Some WSP systems may not be functioning properly. This may simply be due to overloading (in which case the WSP system needs extending – see Section 8.2), but it can often be the result of:

- (a) improper process and/or physical design;
- (b) poor design and/or operation of the inlet works; and/or
- (c) inadequate maintenance of the ponds.

The effects can be quite serious: odour release from both anaerobic and facultative ponds; fly breeding in anaerobic ponds; floating macrophytes or emergent vegetation in facultative and maturation ponds leading to mosquito breeding; and in extreme cases the ponds can silt up and completely “disappear”.

Rehabilitation is achieved by a combination of the following:

- (a) a complete overhaul (or redesign) of the inlet works, replacing any units that cannot be satisfactorily repaired;
- (b) repairing or replacing any flow measuring devices;
- (c) ensuring that any flow-splitting devices actually split the flow into the required proportions;
- (d) desludging the anaerobic or primary facultative ponds, and any subsequent ponds if necessary;
- (e) unblocking, repairing or replacing pond inlets and outlets;
- (f) repositioning any improperly located inlets and/or outlets, so that they are in diagonally opposite corners of each pond;
- (g) repairing, replacing or providing effluent scum guards;
- (h) preventing “surface streaming” of the flow when the pond is stratified by discharging the influent at mid-depth (or by installing a baffled inlet to achieve the same effect);

- (i) removing scum and floating or emergent vegetation from the facultative and maturation ponds;
- (j) checking embankment stability, and repairing, replacing or installing embankment protection;
- (k) checking for excessive seepage (>10 percent of inflow) and lining the ponds if necessary;
- (l) cutting the embankment grass; and
- (m) repairing or replacing any external fences and gates; fences may need to be electrified to keep out wild and domestic animals.

As rehabilitation can be expensive, good routine maintenance is very much more cost-effective.

8.2 UPGRADING AND EXTENDING EXISTING WSP

Prior to upgrading or extending a WSP system its performance should be evaluated as described in Section 7.2, as this will generally permit the correct decision about how to upgrade and/or extend the system to be made.

A number of strategies can be used to upgrade and extend WSP systems. In addition to any rehabilitation measures needed (Section 8.1), these include:

- (a) provision of anaerobic ponds;
- (b) provision of additional maturation ponds;
- (c) provision of one or more additional series of ponds; and/or
- (d) alteration of pond sizes and configuration – for example, removal of an embankment between two ponds to create a larger one.

Figure 8.1 shows how (a), (b) and (d) above can be combined to upgrade a single series of WSP to receive twice its original design flow – at a lower overall retention time, and with the production of a higher quality effluent.

8.3 ALGAL REMOVAL

The algae in a WSP effluent contribute to both its suspended solids content and BOD. If the local regulatory agency does not

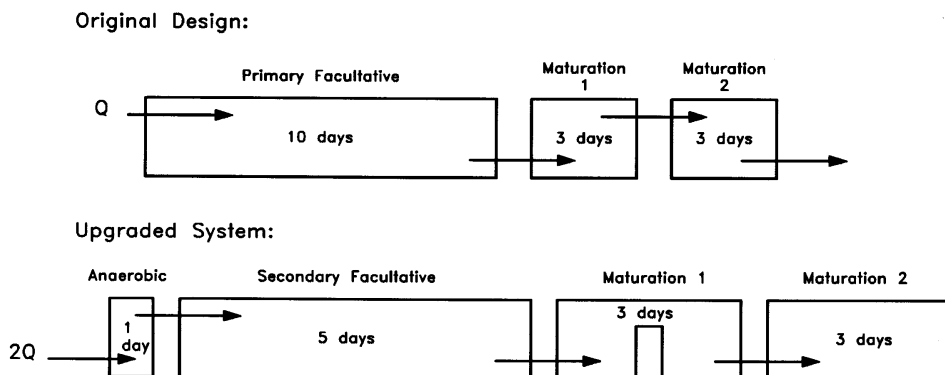


Figure 8.1 Upgrading a WSP series to treat twice the original flow. The embankment between the original maturation ponds becomes a baffle in the upgraded first maturation pond. The total retention time is reduced from 16 to 12 days and the improvement in microbiological quality can be illustrated as follows, by using equation 4.14 with $N_i = 5 \times 10^7$ per 100 ml and $k_T = 6.2 \text{ d}^{-1}$ (i.e. for 25°C):

$$\begin{aligned} \text{Original system: } N_e &= 5 \times 10^7 / [1 + (6.2 \times 10)](1 + (6.2 \times 3))^2 \\ &= 2066 \text{ per 100 ml} \end{aligned}$$

$$\begin{aligned} \text{Upgraded system: } N_e &= 5 \times 10^7 / [1 + (6.2 \times 1)](1 + (6.2 \times 5)) \\ &\quad (1 + (6.2 \times 3))^2 \\ &= 575 \text{ per 100 ml} \end{aligned}$$

make allowance for the inherent difference between algal SS and BOD and “ordinary” effluent SS and BOD (see Section 4.1), it may be necessary to incorporate an algal removal technique to “polish” the WSP effluent. The most appropriate technique for this is a rock filter, although it should be noted that algal removal is not necessary if the effluent is used for crop irrigation or fish culture (Section 10).

Rock filters consist of a submerged porous rock bed within which algae settle out as the effluent flows through. The algae decompose releasing nutrients which are utilized by bacteria growing on the surface of the rocks. In addition to algal removal, significant ammonia removal may also take place through the activity of nitrifying bacteria growing on the surface of the filter medium.

Performance depends on loading rate, temperature and rock size and shape. Permissible loading increases with temperature, but in general an application rate of 1.0 m^3 of pond effluent per m^3



Figure 8.2 Rock filter installed in the corner of a pond at Veneta, Oregon, USA.

rock bed per day should be used. Rock size is important, as surface area for microbial film formation increases with decreasing rock size but, if the rocks are too small, then problems can occur with clogging. Rock size is normally 75 – 100 mm, with a bed depth of 1.5-2.0 m. A typical rock filter is shown in Figure 8.2. The effluent should be introduced just below the surface layer because odour problems are sometimes encountered with cyanobacterial films developing on wet surface rocks exposed to the light.

Construction costs are low and very little maintenance is required, although periodic cleaning to remove accumulated humus is necessary, but this can be carried out during the cooler months when algal concentrations are lowest. BOD and SS removals of 50 and 70 percent have been reported for maturation pond effluents in the USA (Middlebrooks, 1988).

9.

Wastewater storage and treatment reservoirs

While it is true that waste stabilization ponds can more easily produce effluents suitable for agricultural reuse (principally crop irrigation – see Section 10) than other wastewater treatment processes, they share the same disadvantage with these other processes, namely that their effluent can only be used for crop irrigation during the irrigation season. During the other months of the year, the effluents are discharged, essentially to waste, to a surface watercourse.

Wastewater storage and treatment reservoirs (WSTR), also called effluent storage reservoirs, were originally developed in Israel to overcome this disadvantage and permit the whole year's treated wastewater to be used for crop irrigation during the irrigation season. WSTR are especially advantageous in arid and semi-arid areas (such as Israel) where agricultural production is limited by the quantity of water (including treated wastewater) available for irrigation. Wastewater is too valuable to waste in arid and semi-arid areas, and the use of WSTR prevents such waste.

9.1 SINGLE-WSTR SYSTEM

In Israel, where treated wastewater is extensively reused, mainly for the irrigation of cotton, the practice is to treat the wastewater in an anaerobic pond and to discharge its effluent into a single WSTR which is 5-15 m deep (Figure 9.1). The irrigation season in Israel is four months long, and so the single WSTR has a storage capacity equivalent to eight months wastewater flow. It is full at the start of the irrigation season, and empty at the end of it.

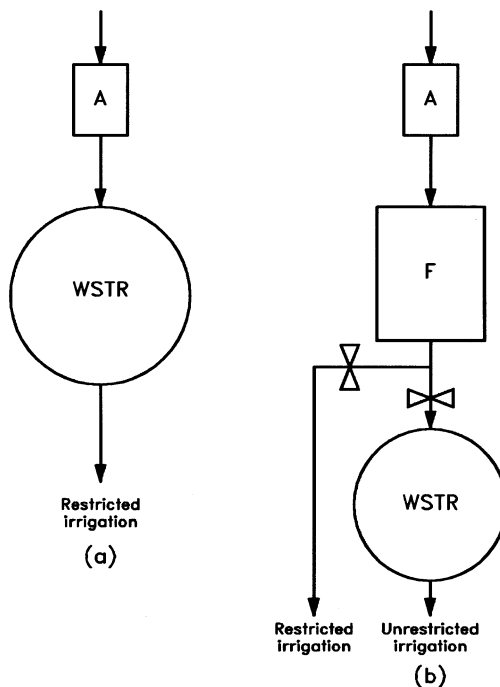


Figure 9.1

(a) single-WSTR system for restricted irrigation, (b) hybrid WSP-WSTR system for both restricted and unrestricted irrigation. A, anaerobic pond; F, facultative pond. In (b) a maturation may be necessary after the facultative pond – see text.

In this way three times as much land can be irrigated, and three times as much cotton (or other crops) produced. Further details are given in Juanico and Shelef (1991, 1994) and Juanico (1995).

Design Example No. 5(a) in Annex I shows how a single-WSTR system is designed for restricted irrigation.

9.2 HYBRID WSP-WSTR SYSTEM

The Israeli system described above is for restricted irrigation (see Section 10.1), and the long retention time in the WSTR ensures that the effluent contains > 1 intestinal nematode egg per litre, which is the WHO (1989) guideline for restricted irrigation (Table 10.1). However, if farmers wish to practise unrestricted irrigation (i.e. the irrigation of vegetables, including salad crops eaten raw), the above single-WSTR system is not suitable as the effluent will contain $> 1,000$ faecal coliform bacteria per 100 ml, which is the WHO (1989) guideline for unrestricted irrigation (Table 10.1).

For unrestricted irrigation, two WSTR options are available:

- (a) three or four sequential batch-fed WSTR (Mara and Pearson, 1992), and
- (b) a “hybrid” WSP-WSTR system.

Only option (b) is described here as the O&M requirements of option (a) are somewhat complicated. Furthermore all the effluent produced by option (a) is suitable for unrestricted irrigation, whereas option (b) produces roughly equal proportions of effluent suitable for restricted and unrestricted irrigation, which is what most agricultural production systems need. Option (b) is highly cost-effective (see Mara *et al.*, 1997) and cheaper than option (a), and only slightly more expensive than the Israeli single-WSTR system (which produces effluent suitable only for restricted irrigation).

The hybrid WSP-WSTR system is shown in Figure 9.1. The wastewater is treated in an anaerobic and facultative pond. During the months when effluent is not required for irrigation, the facultative pond effluent is discharged into a single WSTR; during this period the long retention time ensures that faecal coliform numbers in the WSTR fall to below 1000 per 100 ml. During the irrigation season the facultative pond effluent is used for restricted irrigation, and the WSTR contents for unrestricted irrigation.

Depending on the retention times in the anaerobic and facultative ponds, and the number of intestinal nematode eggs in the raw wastewater, it may be necessary to have a single maturation pond between the facultative pond and the WSTR. This is to ensure that the effluent used for restricted irrigation contains ≤ 1 intestinal nematode egg per litre (see Section 4 and Design Example No. 2 in Annex I).

Thus if, for example, the irrigation season is six months long, the hybrid WSP-WSTR system permits twice the area of land to be irrigated – half for restricted irrigation and half for unrestricted irrigation. As noted in Section 10, discussions must be held with the local farmers to ensure that they are aware of these two irrigation water qualities. In order to protect public health the facultative (or maturation) pond effluent can *only* be used for restricted irrigation.

Design Example No. 5(b) in Annex I shows how a hybrid WSP-WSTR system is designed.

10.

Effluent reuse

10.1 MICROBIOLOGICAL QUALITY GUIDELINES

Crop irrigation

The World Health Organization's (1989) guidelines for the microbiological quality of treated wastewaters to be used for crop irrigation are given in Table 10.1. They are based on a rigorous appraisal of the available epidemiological evidence (see Shuval *et al.*, 1986), which showed that the excreted pathogens of most concern in crop irrigation are the human intestinal nematodes and faecal bacteria. The nematode guideline of no more than one egg per litre is required for both restricted and unrestricted irrigation to protect fieldworkers and, in the latter case, also the consumers (restricted irrigation refers to crops not grown for direct human consumption; unrestricted irrigation includes vegetables and salad crops eaten raw). (There is, however, some evidence that, for restricted irrigation only, the guideline could be safely relaxed to 10 eggs per litre; see Ayres *et al.*, 1992*b*.)

Irrigation with untreated wastewater is very hazardous to health, with both fieldworkers and crop consumers being at high risk of helminthic infections; consumers are also at high risk of bacterial infection such as cholera and typhoid fever (see Shuval *et al.*, 1986). In India, irrigation with untreated wastewater is common at "sewage farms," and this practice has been shown to have high health risks (Krishnamoorthi *et al.*, 1973; see Figure 10.1).

The faecal coliform guideline of no more than 1000 per 100 ml is to protect consumers from bacterial diseases (these are not a risk to fieldworkers). This is much less stringent than earlier WHO

(1973) recommendations (≤ 100 per 100 ml), but is justified because:

- a) the data presented in Table 3.2 show that pond effluents containing 7000 FC per 100 ml do not contain bacterial pathogens;
- b) swimming (i.e. whole body immersion) in recreational waters containing up to 2000 FC per 100 ml is permitted in Europe (Council for the European Communities, 1976);
- c) irrigation with river water containing up to 1000 FC per 100 ml is allowed in the United States (EPA, 1973); and
- d) food eaten raw is allowed to contain up to 100,000 FC per 100 g (wet weight), but preferably less than 1000 FC per 100 g (ICMSF, 1974; see also Mara, 1995).

Table 10.1 Microbiological quality guidelines for treated wastewater used for irrigation

Reuse conditions	Exposed group	Intestinal nematodes ^a (arithmetic mean no. of eggs per litre)	Faecal coliforms (geometric mean no. per 100 ml)
Unrestricted irrigation (crops likely to be eaten uncooked, sports fields, public parks)	Workers, consumers, public	≤ 1	$\leq 1000^b$
Restricted irrigation (cereal crops, industrial crops, fodder crops, pasture and trees ^c)	Workers	≤ 1	No guideline required

^a *Ascaris lumbricoides*, *Trichuris trichiura* and the human hookworms.

^b A more stringent guideline (≤ 200 faecal coliforms per 100 ml) is appropriate for public lawns, such as hotel lawns, with which the public may come into direct contact.

^c In the case of fruit trees, irrigation should cease two weeks before fruit is picked, and no fruit should be picked off the ground. Sprinkler irrigation should not be used.

Source: WHO (1989).

Figure 10.1
Prevalences of
ascariasis and
hookworm
infection in
sewage farm
workers and a
control group in
India
(Krishnamoorthi
et al., 1973).



Health risks

Shuval (1996) has quantified the annual health risks which result from the consumption of raw salad crops irrigated with wastewaters treated to various faecal coliform levels, and these are compared to the EPA's (1989) acceptable annual risk of waterborne disease, as follows:

- (a) EPA's (1989) acceptable annual risk of waterborne disease: 10^{-4}
- (b) Consumption of raw salad crops irrigated with raw wastewater (10^7 FC per 100 ml):
 - annual risk of hepatitis A: 10^{-2}
- (c) Consumption of raw salad crops irrigated with wastewater treated to the WHO (1989) guideline level of 1000 FC per 100 ml:
 - annual risk of hepatitis A: $10^{-6} - 10^{-7}$
 - annual risk of rotavirus infection: $10^{-5} - 10^{-6}$

Thus, as noted by WHO (1989), irrigation with untreated wastewaters is dangerous. However, irrigation with wastewaters treated to 1000 FC per 100 ml is safer than drinking potable water by 1-3 orders of magnitude.

Fishpond fertilization

The WHO guidelines for effluents to be used for fishpond fertilization are an absence of trematode eggs (*Schistosoma* spp., *Clonorchis sinensis* and *Fasciolopsis buski*, but these parasites are very rare in India) and no more than 1000 FC per 100 ml of fishpond water. No trematodes eggs are permitted because of the high asexual multiplication of the parasite in its intermediate aquatic host (water snails). The FC guideline refers to their numbers *in* the fishpond, so effluents discharging into these can contain up to 10,000 FC per 100 ml, as there will be a one log unit reduction in the fishpond (if temperature considerations suggest that the reduction will be greater – use equations 4.14 and 4.15 – then the effluent can, of course, contain more than 10,000 FC per 100 ml).

10.2 PHYSICOCHEMICAL QUALITY GUIDELINES

The microbiological quality guidelines are for the protection of human health; those for physicochemical quality are to protect plant health and maintain crop yields. In general the physicochemical quality of treated wastewaters used for crop irrigation should comply with FAO's recommendations for the quality of *water* used for irrigation (Ayers and Westcot, 1985). For effluents from WSP treating industrial wastewaters (or municipal wastewaters containing an appreciable proportion of industrial wastes) these recommendations should be carefully checked, particularly with respect to heavy metals and other toxicants. For effluents from WSP treating domestic or normal municipal wastewaters it is generally only necessary to consider the following five parameters:

- (a) electrical conductivity (as a convenient measure of total dissolved solids and hence of the salinity hazard to the crop), measured in millisiemens per metre at 25°C;
- (b) sodium absorption ratio (as a measure of the sodium or alkali hazard to the crop), defined as:

$$\text{SAR} = \text{Na} / [1/2(\text{Ca} + \text{Mg})]^{1/2}$$

where Na, Ca and Mg are expressed in milli-equivalents per litre (= concentration in mg/l \times 0.044, 0.050 and 0.082 for Na, Ca and Mg respectively).

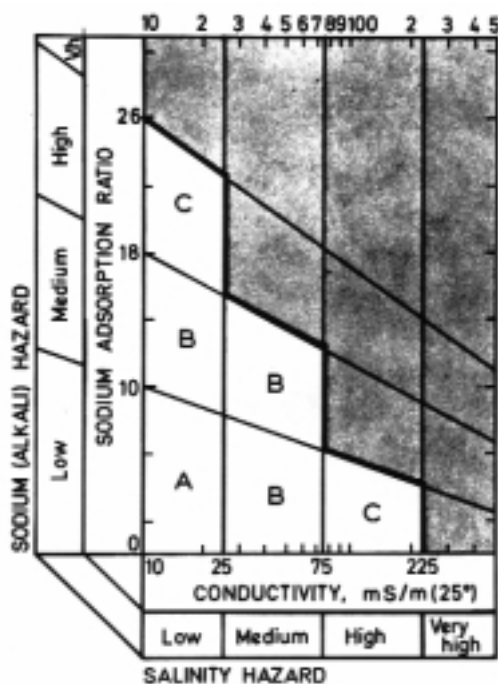
The values of EC and SAR are interdependent – see Figure 10.2.

- (c) pH : the permissible range is 6.5 – 8.4.
- (d) Total nitrogen: too much nitrogen can reduce crop yields, even though there may be a more luxuriant growth of the non-useful parts of the crop. Most crops are unaffected by up to 30 mg N/l, but sensitive crops (refer to Ayers and Westcot, 1985) can tolerate only up to 5 mg N/l.
- (e) Boron: citrus and deciduous fruits and nuts are sensitive to concentrations of boron (derived from synthetic detergents) above 0.5 mg/l, but most crops can tolerate up to 2 mg/l (Ayers and Westcot (1985) give more detailed information).

With effluents from WSP treating domestic or normal municipal wastewaters there are few, if any, physicochemical problems. Nonetheless it is always prudent to analyse samples regularly for the above five parameters.

There is no need to consider, in the case of agricultural reuse, the effluent BOD. However, when the effluent is to be reused in aquaculture, its unfiltered BOD should not exceed 50 mg/l to prevent deoxygenation and subsequent fish kills.

Figure 10.2
 Classification of irrigation waters based on EC and SAR. Waters in regions A and B are suitable for almost all purposes. Those in region C should be avoided wherever possible, or used only under expert advice, and those in the shaded area should not be used at all.



10.3 AGRICULTURAL REUSE

Irrigation with WSP effluents, as with other suitability treated wastewaters, provides a good balance of plant nutrients (principally N, P and K salts), which can markedly increase crop production and reduce the requirements for expensive artificial fertilizers (Table 10.2). WSP effluents bring additional benefits since the algae they contain add to the organic (humus) content of the soil and improve soil structure and its water-holding capacity. The algae also act as “slow-release” fertilizers, releasing plant nutrients as they slowly decompose in the soil even after irrigation has ceased.

Table 10.2 Crop yields (tonnes per hectare per year) on experimental plots in India

Irrigation water	Wheat (8) ^a	Moong beans (5)	Rice (7)	Potato (4)	Cotton (3)
Raw wastewater	3.34	0.90	2.97	23.11	2.56
Settled wastewater	3.45	0.87	2.94	20.78	2.30
Waste stabilization pond effluent	3.45	0.78	2.98	22.31	2.41
Fresh water + NPK	2.70	0.72	2.03	17.16	1.70

^a Years of harvest used to calculate average yield.

Source: Shende (1985).

An important point to consider in the design of WSP systems is that overall retention times, and therefore the land areas required, can be greatly reduced if the effluent is to be reused for restricted irrigation as opposed to unrestricted irrigation (see Design Example Nos. 2 and 3 in Annex I).

10.4 AQUACULTURAL REUSE

10.4.1 Traditional practice

Although aquaculture (literally “water farming”) can also refer to the cultivation of aquatic vegetation, the term is primarily used to

describe the cultivation of fish. Wastewater-fed aquaculture is an age-old practice throughout India, China and south-east Asia where it is an important source of high quality animal protein for low-income families. It has also been practised on a commercial scale for more than 50 years in Germany and Hungary and there is now increasing interest in the USA and many other countries.

Wastewater-fed fisheries produce large amounts of fish. The Calcutta East wastewater-fed fishponds (see Section 2.2.3) produce 4-5 tonnes of fish per hectare per year, although in the better managed ponds yields are 7 t/ha yr. Improved design (see Section 10.4.2) has the potential of increasing yields to over 10 t/ha yr, while at the same time ensuring the microbiological safety of the fish.

10.4.2 Improved fishpond design

In order to be able to be more certain about the safety of wastewater-fed aquaculture, Mara *et al.* (1993) proposed the following design procedure for wastewater-fed fishponds, which was based on their work on the Calcutta East wastewater-fed fisheries. This design procedure (modified to include consideration of the free ammonia concentration in the fishpond) is for the *minimal* treatment of the wastewater (in anaerobic and facultative ponds) and the *maximal* production of microbiologically safe fish.

The design steps that should be followed are:

- (a) Design an anaerobic pond and a facultative pond, as detailed in Sections 4.3 and 4.4.
- (b) Use equation 4.23 to determine the total nitrogen concentration in the facultative pond effluent (C , mg N/l).
- (c) Design the wastewater-fed fishpond, which receives the facultative pond effluent, on the basis of a surface loading of total nitrogen of 4 kg total N per ha day. Too little nitrogen results in a low algal biomass in the fishpond and consequently small fish yields. Too much nitrogen gives rise to high concentrations of algae, with the resultant high risk of severe dissolved oxygen depletion at night and consequent fish kills. A loading of around 4 kg total N/ha day is optimal.

The fishpond area is given by the following version of equation 4.4:

$$A_{fp} = 10 CQ/\lambda_s^{TN} \quad (10.1)$$

Use equation 4.12 to calculate the retention time in the fishpond (θ_{fp} , days), with a fishpond depth of 1 m.

- (d) Use the following version of equation 4.14 to calculate the number of faecal coliform bacteria per 100 ml of fishpond water (N_{fp}):

$$N_{fp} = N_i / (1 + k_T \theta_a)(1 + k_T \theta_f)(1 + k_T \theta_{fp}) \quad (10.2)$$

Check that N_{fp} is ≤ 1000 per 100 ml. If it is not, increase θ_{fp} until it is (or consider having a maturation pond ahead of the fishpond).

- (e) Use equation 4.21 or 4.22 to determine the concentration of $\text{NH}_3 - \text{N}$ first in the facultative pond effluent (assume that the conversion of total nitrogen in the anaerobic pond to ammonia produces an ammonia concentration in the effluent of the anaerobic pond – that is, in the influent to the facultative pond – equal to 75% of the total nitrogen concentration in the raw wastewater), and then in the fishpond. The ammonia concentration is the total concentration of NH_3 and NH_4^+ , sometimes termed “free and saline ammonia”. In order to protect the fish from free ammonia (NH_3) toxicity, the concentration of NH_3 should be less than 0.5 mg N/l. The percentage (p) of free ammonia in aqueous ammonia solutions depends on temperature (T , °K) and pH, as follows (Emerson *et al.*, 1975; see also Erickson, 1985):

$$p = 1 / [10^{(pK_a - \text{pH})} + 1] \quad (10.3)$$

where pK_a is given by:

$$pK_a = 0.09018 + (2729.92/T) \quad (10.4)$$

Equations 10.3 and 10.4 (or Table 10.3 which is derived from them) should be used to determine the free ammonia concentration in the fishpond, assuming a pH of 7.5 (the pH range in wastewater-fed fishponds is usually 6.5-7.5).

Design Example No. 5 in Annex I shows how these equations are used.

Table 10.3 Percentage of free ammonia (NH₃) in aqueous ammonia (NH₃ + NH₄⁺) solutions for 15-29°C and pH 7.0-8.5)

Temperature (°C)	Percentage of free ammonia in aqueous ammonia solutions at pH			
	7.0	7.5	8.0	8.5
15	0.273	0.859	2.67	7.97
16	0.294	0.925	2.87	8.54
17	0.317	0.996	3.08	9.14
18	0.342	1.07	3.31	9.78
19	0.368	1.15	3.56	10.50
20	0.396	1.24	3.82	11.20
21	0.425	1.33	4.10	11.90
22	0.457	1.43	4.39	12.70
23	0.491	1.54	4.70	13.50
24	0.527	1.65	5.03	14.40
25	0.566	1.77	5.38	15.30
26	0.607	1.89	5.75	16.20
27	0.651	2.03	6.15	17.20
28	0.691	2.17	6.56	18.20
29	0.747	2.32	7.00	19.20

Source: Emerson *et al.* (1975).

Improved fish yields

Improved fishpond management can be achieved by having small ponds, up to 1 ha in area, that can be stocked with fingerlings, fertilized with facultative pond effluent and then harvested 3 months after stocking. During this time the fingerlings will have grown from 20 g to 150-250 g (Figure 10.3). Partially draining the pond will ensure that almost all the fish can be harvested. This cycle can be done 3 times per year. Allowing for a 25% fish loss due to mortality, poaching and consumption by fish-eating birds, the annual yield is:

$$\begin{aligned}
 & (3 \times 200 \text{ g fish per m}^2) (10^{-6} \text{ tonnes/g}) (10^4 \text{ m}^2/\text{ha}) \\
 & \times (3 \text{ harvests per year}) (0.75, \text{ to allow for the 25\% loss}) \\
 & = 13.5 \text{ tonnes of fish per hectare per year.}
 \end{aligned}$$

This is 2-3 times the yields currently achieved in the Calcutta East wastewater-fed fishponds (if all the existing 3000 ha of



Figure 10.3
Harvesting Indian major carp at the Calcutta East wastewater-fed fishponds.

wastewater-fed fishponds in Calcutta East were managed in this improved way, they could supply nearly 50% of the local demand for fish).

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Annex I

WSP process design examples

1. SURFACE WATER DISCHARGE

Design a WSP system to treat 10,000 m³/day of a wastewater which has a BOD of 200 mg/l. The design temperature is 25°C and the net evaporation rate is 5 mm/day..

Solution

(a) With anaerobic ponds

Anaerobic pond

From Table 4.1 the design loading for 25°C is 300 g BOD/m³ day.

Substitution of equation 4.2 into equation 4.3 gives the following alternative expression for the anaerobic pond retention time, θ_a :

$$\begin{aligned}\theta_a &= L_i/\lambda_v \\ &= 200/300 \\ &= 0.67 \text{ day}\end{aligned}$$

As $\theta_a < 1$ day, adopt $\theta_a = 1$ day. Thus the anaerobic pond volume (V_a) is given by:

$$\begin{aligned}V_a &= Q/\theta_a \\ &= 10,000 \times 1 \\ &= 10,000 \text{ m}^3\end{aligned}$$

Assuming a depth of 4 m, the anaerobic pond area is 2,500 m².

At 25°C the BOD removal (Table 4.1) is 70%, so the BOD of the anaerobic pond effluent is (0.3 × 200), i.e. 60 mg/l.

Facultative Pond

From Table 4.3 the design loading for 25°C is 350 kg BOD/ha day.

Thus the facultative pond area is given by equation 4.4 as:

$$\begin{aligned} A_f &= 10L_iQ/\lambda_S \\ &= 10 \times 60 \times 10,000/350 \\ &= 17,143 \text{ m}^2 \end{aligned}$$

Calculate the retention time in the facultative pond from equation 4.12:

$$\theta_f = 2A_fD/(2Q_i - 0.001A_fe)$$

Taking the depth as 1.5 m:

$$\begin{aligned} \theta_f &= 2 \times 17,143 \times 1.5 / [(2 \times 10,000) - \\ &\quad (0.001 \times 17,143 \times 5)] \\ &= 2.6 \text{ days} \end{aligned}$$

This is too low. Adopt for 25°C a minimum value of 4 days and calculate the area of the facultative pond from a rearrangement of equation 4.12 (i.e. use equation 4.18 with θ_f in place of θ_m):

$$\begin{aligned} A_f &= 2Q_i\theta_f/(2D + 0.001e\theta_f) \\ &= 2 \times 10,000 \times 4 / [(2 \times 1.5) + (0.001 \times 5 \times 4)] \\ &= 26,490 \text{ m}^2 \end{aligned}$$

The cumulative filtered BOD removal in the anaerobic and facultative ponds is 90% for $T > 20^\circ\text{C}$, so the facultative pond effluent has a filtered BOD of (0.1 × 200), i.e. 20 mg/l, which is suitable for river discharge.

(b) Without anaerobic ponds*Facultative pond*

$$\begin{aligned}
 A_f &= 10 L_i Q / \lambda_s \\
 &= 10 \times 200 \times 10,000 / 350 \\
 &= 57,143 \text{ m}^2
 \end{aligned}$$

Comparison of designs

The two designs, with and without anaerobic ponds, have the following mid-depth area requirements:

With anaerobic ponds:

Anaerobic pond:	2,500 m ²
Facultative pond:	26,490 m ²
Total:	28,990 m ²

Without anaerobic ponds:

Facultative pond: 57,143 m²

Thus the use of anaerobic ponds results in a land saving of 49%. This confirms the observation of Professor Gerrit Marais (1970) that **“anaerobic pretreatment is so advantageous that the first consideration in the design of a series of ponds should always include anaerobic pretreatment.”**

2. RESTRICTED IRRIGATION

Design a WSP system as in Design Example No. 1, but for restricted irrigation. Assume that the wastewater contains 750 intestinal nematode eggs per litre.

Solution

The anaerobic and facultative ponds are as calculated in Design Example No. 1. The retention times in the anaerobic and

facultative ponds are 1 and 4 days, respectively. From Table 4.6 the percentage egg removals in the ponds are:

Anaerobic pond:	74.67
Facultative pond:	93.38

Thus the anaerobic pond effluent contains (0.2533×750) , i.e. 190 eggs per litre, and the facultative pond effluent contains (0.066×190) , i.e. 13 eggs per litre. A maturation pond is therefore required to reduce the number of eggs to 1 per litre for restricted irrigation (Table 10.1).

The required percentage egg removal in the maturation pond is:

$$100[(13 - 1)/13]$$

i.e. 92%. So, from Table 4.7, choose $\theta_m = 3.6$ days. The maturation pond area is given by equation 4.18 as:

$$A_m = 2Q_i\theta_m/(2D + 0.0001e\theta_m)$$

Q_i is the effluent flow from the facultative pond, and is therefore given by:

$$\begin{aligned} Q_i &= 10,000 - 0.001 A_f e \\ &= 10,000 - (0.001 \times 26,490 \times 5) \\ &= 9,867 \text{ m}^3/\text{day} \end{aligned}$$

Therefore, taking the depth as 1.5 m:

$$\begin{aligned} A_m &= 2 \times 9,867 \times 3.6 / [2 \times 1.5 \\ &\quad + (0.001 \times 5 \times 3.6)] \\ &= 23,540 \text{ m}^2 \end{aligned}$$

The final effluent flow for restricted irrigation is given by:

$$\begin{aligned} Q_e &= 9,867 - (0.001 \times 23,540 \times 5) \\ &= 9,749 \text{ m}^3/\text{day} \end{aligned}$$

Thus only 2.5% of the flow is lost due to evaporation.

Thus, for restricted irrigation, the mid-depth area requirements are:

Anaerobic pond:	2,500 m ²
Facultative pond:	26,490 m ²
Maturation pond:	23,540 m ²
Total:	52,530 m ²

3. UNRESTRICTED IRRIGATION

Design a WSP system as in Design Example No. 1, but for unrestricted irrigation. Assume that the wastewater contains 5×10^7 faecal coliforms per 100 ml.

Solution

The anaerobic and facultative ponds are as calculated in Design Example No. 1. The retention times in the anaerobic and facultative ponds are 1 and 4 days, respectively.

Maturation ponds

Use the following rearrangement of equation 4.14 to calculate θ_m :

$$\theta_m = \{ [N_i/N_e(1 + k_T\theta_a)(1 - k_T\theta_f)]^{1/n} - 1 \} / k_T$$

At 25°C $k_T = 6.2 \text{ day}^{-1}$ (Table 4.6). Therefore the above equation can be solved for the following values of n as follows, with $N_e = 1000$ for unrestricted irrigation (Table 10.1):

$$\begin{aligned} \theta_m &= \{ [5 \times 10^7 / 1000 (1 + 6.2 \times 1)(1 + 6.2 \times 4)]^{1/n} - 1 \} / 6.2 \\ &= 43.3 \text{ days for } n = 1 \\ &= 2.5 \text{ days for } n = 2 \end{aligned}$$

Choose 2 ponds each with a retention time of 3 days ($= \theta_m^{\min}$). Check BOD loading on the first maturation pond from equation 4.17, assuming 80% cumulative removal in the anaerobic and facultative ponds and a depth of 1.5 m:

$$\begin{aligned} \lambda_{s(m1)} &= 10 \times (0.2 \times 200) \times 1.5 / 3 \\ &= 200 \text{ kg/ha day} \end{aligned}$$

This is satisfactory as it is less than 75% of the permissible design loading on facultative ponds at 25°C (350 kg/ha day; Table 4.3).

The area of the first maturation pond is given by equation 4.18 as:

$$\begin{aligned} A_{m1} &= 2Q_i\theta_m / (2D + 0.001e\theta_m) \\ &= 2 \times 9,867 \times 3 / [(2 \times 1.5) + (0.001 \times 5 \times 3)] \\ &= 19,636 \text{ m}^2 \end{aligned}$$

The effluent flow is given by:

$$\begin{aligned} Q_e &= Q_i - 0.001A_{m1}e \\ &= 9,867 - (0.001 \times 19,636 \times 5) \\ &= 9,769 \text{ m}^3/\text{day} \end{aligned}$$

Similarly the area of the second maturation pond and its effluent flow are given by:

$$\begin{aligned} A_{m2} &= 2 \times 9,769 \times 3 / [(2 \times 1.5) + (0.002 \times 5 \times 3)] \\ &= 19,441 \text{ m}^2 \end{aligned}$$

$$\begin{aligned} Q_e &= 9,769 - (0.001 \times 19,441 \times 5) \\ &= 9,672 \text{ m}^3/\text{day} \end{aligned}$$

Thus only 3% of the flow is lost due to evaporation.

BOD removal

Assuming a 90% cumulative removal of filtered BOD in the anaerobic and facultative ponds, and 25% in each of the two maturation ponds, the final effluent will have a filtered (i.e. non-algal) BOD of:

$$200 \times 0.1 \times 0.75 \times 0.75 = 11 \text{ mg/l}$$

Summary

Thus, for unrestricted irrigation, the mid-depth area requirements are:

Anaerobic pond:	2,500 m ²
Facultative pond:	26,490 m ²
First maturation pond:	19,636 m ²
Second maturation pond:	19,441 m ²
Total:	68,067 m ²

This is 30% more than required for restricted irrigation (Design Example No. 2).

4. FISH CULTURE

Design a WSP system as in Design Example No. 1, but for fish culture. Assume that the total nitrogen and ammonia concentrations in the wastewater are 25 and 15 mg N/l, respectively.

Solution

The anaerobic and facultative ponds are as culated in Design Example No. 1. Assume that there is no total N removal in the anaerobic pond, and that there is an increase in the ammonia concentration in the anaerobic pond effluent to 20 mg N/l.

Calculate the total N and ammonia concentrations in the effluent of the facultative pond using equations 4.23 and 4.22, respectively, assuming the pH is 8:

$$\begin{aligned} C_e &= C_i \exp \{-[0.0064(1.039)^{T-20}] \\ &\quad [\theta + 60.6(\text{pH} - 6.6)]\} \\ &= 25 \exp\{-[0.0064(1.039)^5] [4 + 60.6(8 - 6.6)]\} \\ &= 12.6 \text{ mg total N/l} \end{aligned}$$

$$\begin{aligned} C_e &= C_i \{1 + [5.035 \times 10^{-3} (A_f/Q)] \\ &\quad [\exp (1.504 \times (\text{pH} - 6.6))]\} \\ &= 20/\{1 + [5.035 \times 10^{-3} (26,490/10,000)] [\exp \\ &\quad (1.504 \times (8 - 6.6))]\} \\ &= 18.0 \text{ mg } (\text{NH}_3 + \text{NH}_4^+) - \text{N/l} \end{aligned}$$

Fishpond

Calculate the area of the fishpond on the basis of a surface loading of total nitrogen of 4 kg/ha day:

$$\begin{aligned} A_{fp} &= 10C_iQ/\lambda_s \\ &= 10 \times 12.6 \times 9,867/4 \\ &= 310,811 \text{ m}^2 \end{aligned}$$

The retention time in the fishpond is given by equation 4.12 as:

$$\theta_{fp} = 2A_{fp}D/(2Q_i - 0.001A_{fp}e)$$

Assuming the depth is 1 m:

$$\begin{aligned} \theta_{fp} &= 2 \times 310,811 \times 1 / [(2 \times 9,867) \\ &\quad - (0.001 \times 310,811 \times 5)] \\ &= 34 \text{ days} \end{aligned}$$

Check the concentration of faecal coliform bacteria in the fishpond, using equation 4.14:

$$\begin{aligned} N_e &= N_i / (1 + k_T\theta_a)(1 + k_T\theta_f)(1 + k_T\theta_{fp}) \\ &= 5 \times 10^7 / (1 + 6.2 \times 1)(1 + 6.2 \times 4)(1 + 6.2 \times 34) \\ &= 1271 \text{ per } 100 \text{ ml} \end{aligned}$$

This is just above 1000 per 100 ml, the WHO guideline for wastewater-fed aquaculture, but safe enough. The WHO guideline is really only refers to the order of magnitude, and 1271 is effectively 10^3 and, of course, $\ll 10^4$.

Check the ammonia concentration in the fishpond, using equation 4.22 and assuming the pH is 7.5:

$$\begin{aligned} C_e &= 18 / \{1 + [5.035 \times 10^{-3} (310,811/9,867)] \\ &\quad [\exp(1.504 \times (7.5 - 6.6))]\} \\ &= 11 \text{ mg } (\text{NH}_3 + \text{NH}_4^+) - \text{N/l} \end{aligned}$$

From Table 10.3 the percentage of free ammonia at pH 7.5 and 25°C is 1.77, so the concentration of free ammonia in the fishpond is (0.0177×11) , i.e. 0.2 mg N/l, which is not toxic to fish.

Summary

Anaerobic pond:	2,500 m ²
Facultative pond:	26,490 m ²
Fishpond:	310,811 m ²
Total:	339,801 m ²

Thus only 8.5% of the total pond area is used for pretreatment prior to fish culture. Of course, the cost of the fishpond is not part of the cost of treatment and should be met by the fishfarmers, not the wastewater treatment authority.

5. WASTEWATER STORAGE AND TREATMENT RESERVOIRS

Design a wastewater storage and treatment reservoir system for the wastewater given in Design Example No. 1. Assume the irrigation season is 6 months.

Solutions

(a) Restricted irrigation

Pretreat the wastewater in an anaerobic pond, i.e. as calculated in Design Example No. 1.

The WSTR must be full at the start of the irrigation season and empty at the end of it, so its volume is equal to 6 months wastewater flow:

$$\begin{aligned} V &= (365/2) \times 10,000 \\ &= 1,825,000 \text{ m}^3 \end{aligned}$$

Assuming a depth of 10 m, the WSTR area is 18.25 ha.

(b) Restricted and unrestricted irrigation

Assume that the local farmers wish to use half the treated wastewater for restricted irrigation and half for unrestricted irrigation.

Use the hybrid WSP-WSTR system shown in Figure 9.1, i.e. use the anaerobic, facultative and maturation ponds calculated in Design Example No. 2, and calculate the WSTR volume for 6 months storage of the facultative pond effluent:

$$\begin{aligned} V &= (365/2) \times 9,749 \\ &= 1,779,193 \text{ m}^3 \end{aligned}$$

i.e. an area of 17.8 ha, assuming a depth of 10 m.

Annex II

Analytical techniques

1. CHLOROPHYLL A

The methanol extract technique described in Pearson, Mara and Bartone (1987) should be used.

Materials and equipment

- (a) 1% (w/v) aqueous suspension of MgCO_3 ;
- (b) 90% (v/v) aqueous methanol;
- (c) 25 mm glass fibre filter papers (e.g. Whatman GF/C);
- (d) compatible filtration system (e.g. Whatman 1960 032 with a 250-1000 ml filter flask) and vacuum source;
- (e) simple spectrophotometer (663 and 750 nm);
- (f) small bench centrifuge (500g)

Different sized filter papers may be used, and if glass fibre filter papers are not available a good quality general purpose paper (e.g. Whatman grade 2) may be used. The centrifuge is not essential, but improves the spectrophotometry by removing any turbidity present.

Method

- (a) Filter 2.5 ml of the MgCO_3 suspension (this aids retention of the algae and maintains alkaline conditions to prevent denaturation of the chlorophyll during extraction).
- (b) Filter a known volume (at least 10 ml and preferably close to 50 ml) of well-stirred pond column subsample.
- (c) Place the filter paper in a glass test tube and add 10 ml of 90% methanol. Boil for 2 minutes to extract the chlorophyll

(the solvent boils at around 65°C, so it can be boiled in a hot water bath). The filter paper should become white; if it does not, macerate it with a glass rod to aid extraction.

- (d) If a centrifuge is available, centrifuge the extract at 500 *g* for 10 minutes. Otherwise leave the extract for 15 minutes in the dark to allow most of the debris to settle out.
- (e) Make up the extract volume to exactly 10 ml by adding 90% methanol, and transfer a portion of the extract to a 1 cm cuvette.
- (f) Set the wavelength on the spectrophotometer to 663 nm (or 665 nm if calibrated in 5 nm divisions). Zero with 90% methanol, and read the absorbency of the chlorophyll extract (the absorbency should be between 0.2 and 0.8; if it is less, re-extract using a larger sample volume; if more, dilute with a known volume of methanol). Set the wavelength to 750 nm, re-zero and read the absorbency of the extract (this corrects for turbidity by measuring non-specific absorbency).
- (g) Calculate the concentration of chlorophyll *a* from:

$$\text{Chl } a \text{ (}\mu\text{g/l)} = [\text{OD}_{663} - \text{OD}_{750}]/77][V/S] \times 10^6$$

where OD₆₆₃ and OD₇₅₀ are the absorbency readings at 663 and 750 nm, and V and S are respectively the solvent extract volume and original sample volume, both in ml. The figure of 77 is the extinction coefficient for chlorophyll *a* in 90% methanol in l/g cm. If the path length of the cuvette used is not 1 cm, then the absorbency difference should be divided by the path length in cm.

- (h) Pond samples should not be stored prior to analysis for longer than 6 hours. In the field the best stage for storage is after filtration. The filter papers should be dried in the dark and at as low a temperature as possible (preferably 4°C). If they are then kept in the dark (e.g. wrapped in foil), they may be stored for several weeks prior to spectrophotometric analysis with a maximum absorbency loss of only 10%.

2. ALGAL IDENTIFICATION

Microscopic examination should first be carried out using a magnification of × 100, (usually × 10 objective and × 10

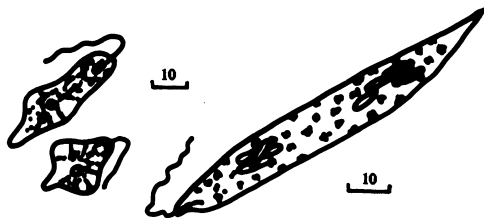
eyepiece), which will enable the detection of large algal cells such as *Euglena*. However smaller cells, such as *Chlorella*, and certain cell constituents, such as the spines of *Micractinium* or *Scenedesmus* and the eyespot of *Euglena*, can only be observed using a magnification of $\times 400$. Cell sizes can be determined using an eyepiece graticule that has been calibrated using a stage micrometer (this is a microscope slide on which a 1 mm line, divided into 100 equal divisions has been etched).

If it is not possible to examine the samples immediately, then they can be preserved with either 4% formalin or 0.7% Lugol's iodine. Ideally two subsamples should be taken and on preserved with each. Formalin preservation results in a more natural colouration, while iodine has the advantage of acting as a cytological stain for starch granules and aoligosaccharides, the location of which within the cell can aid identification. Iodine preservation also results in increasing the density of the cells, which can aid in the concentration and sedimentation of algal cells.

In the figures below, the bar dimensions are micrometres.

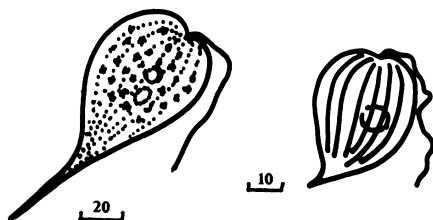
Euglena

Probably the most commonly occurring waste stabilization pond alga. Often present as a surface-stratified layer, especially in facultative ponds. Large cells (up to 150 μm long). Can be elongated and highly motile or amoeboid and slow moving, often rounding up and remaining motionless on microscope slides. Very green with a usually conspicuous red eyespot.



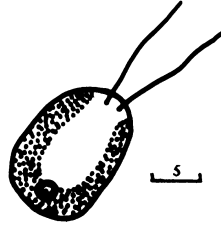
Phacus

Can be difficult to distinguish from *Euglena*, but possesses conspicuous tapering tail and striated body. Also cells often with pronounced dorsoventral flattening (leaf-like in shape), often with some part of cell twisted.



Chlamydomonas

Very common in anaerobic or highly loaded facultative ponds. Very small, ovoid with anterior end rounded. 5-10 μm diameter. 10-20 μm broad. Highly motile. Two flagella but not usually visible.



Pandorina

Common in highly loaded ponds. Cells pear shaped and embedded in a spherical mass of mucilage with 8 or 16 (rarely 32) cells per colony. Cells 8-15 μm broad. Colonies 20-50 μm diameter. Most distinctive feature is tumbling motion of colonies through the water.



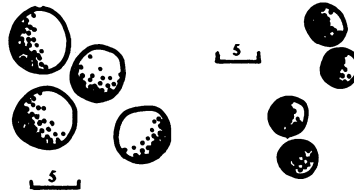
Chlorella

Very common in all types of aerobic ponds.

Unicellular. Spherical. Very small.

C.pyrenoidosa : 3-5 μm diameter

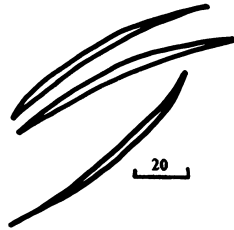
C.vulgaris : 5-10 μm diameter.



Ankistrodesmus

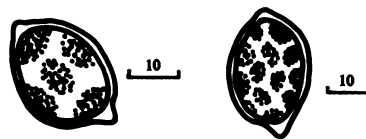
Sometimes found in maturation ponds.

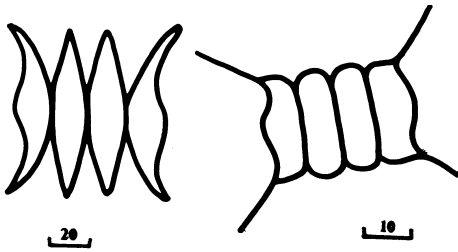
Cells needle-like (25-100 μm long by 2-6 μm broad).



Oocystis

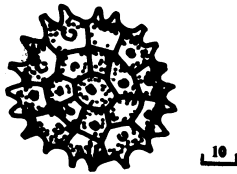
Common in all types of aerobic ponds. Solitary or in groups still enclosed in mother cell wall. Up to 8 cells per colony. Polar nodules on individual cells. Most common pond species is *O.crassa*, approximately 10-20 μm broad, 14-26 μm long.





Scenedesmus

Very common in maturation ponds. Can exist as unicells or colonies of up to 16 cells. Spines often present. Most common pond species is *S. quadricauda* with 4 cells (colony approximately $7 \times 16 \mu\text{m}$) and two spines on each terminal cell.



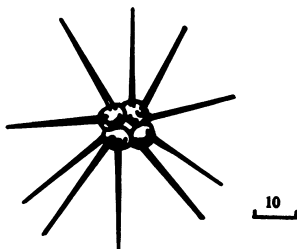
Pediasstrum

Sometimes found in maturation ponds. Colonial. Cells on outer edge have two blunt projections. Individual cell approximately $15 \mu\text{m}$ diameter. Most common pond species is *P. boryanum* with 36 cells per colony.



Selenastrum

Sometimes found in maturation ponds. Cells lunate to arcuate (strongly curved). Often in aggregates of 4, 8 or 16 cells. $2\text{-}8 \mu\text{m}$ broad, $7\text{-}38 \mu\text{m}$ long.

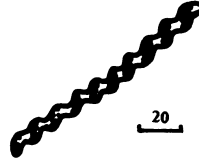


Micractinium

Very common in maturation ponds. Colonial. 4-16 cells in pyramid or square. Outside cells have fine tapering spines or setae (1-5 in number) $10\text{-}35 \mu\text{m}$ long. Individual cells spherical, approximately $3\text{-}7 \mu\text{m}$ diameter.

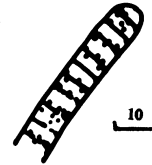
Spirulina

Regularly spirally coiled cylinder in which individual cells are not obvious. 1-15 μm broad. Common in still waters and can be strongly dominant in ponds having long retention times and whose water is rich in dissolved solids due to concentration by evaporation.



Oscillatoria

Filamentous, individual cells, truncate. Approximately 10 μm diameter and 3-5 μm long. Able to move actively through oscillatory motion.



Anabaena

Filamentous. Cells spherical to cylindrical, 3-5 μm long. Able to move actively through oscillatory motion.

Diatoms are also common in waste stabilization ponds on some occasions. Their occurrence is partly determined by the silicon content of the water as this element is a necessary constituent of the cell wall.



3. SULPHIDE

Sulphide analysis should be carried out using the following procedure:

Reagents

- a) Phenylenediamine: 0.2% w/v N, N-dimethyl-p-phenylenediamine sulphate in 20% (v/v) H_2SO_4 . Dissolve

2 g compound in 200 ml distilled water and add 200 ml of conc. H_2SO_4 . Allow to cool and dilute with distilled water to 1000 ml. **Caution:** *this is very poisonous by skin absorption.*

- b) Ferric reagent: 10% w/v ammonium ferric sulphate in 2% (v/v) H_2SO_4 . To 10 g $\text{Fe}_3(\text{NH}_4)(\text{SO}_4)_2 \cdot 12\text{H}_2\text{O}$ add 2 ml conc. H_2SO_4 . Dilute to 100 ml with distilled water. Heating will be required to dissolve the compound.

Method

- 1) Sulphide is rapidly oxidised to sulphate when oxygen is present. Once samples have been taken, it is therefore essential to fix the sulphide immediately. This can be done by adding the first reagent to the samples. 10 ml volumes of reagent (a) should be dispensed into 100 ml volumetric flasks and these taken to the sampling points. 10 ml volumes of sample should then be dispensed into these volumetric flasks immediately after the samples have been taken. The sulphide fixed in this way will be stable for at least one hour, but stability beyond this time has not yet been evaluated.
- 2) Add 2 ml of reagent (b) and leave for ten minutes. A pink colour will develop initially but this should only be transitory. The presence of sulphide will then be indicated by the development of a deep blue colour.
- 3) Dilute samples to 100 ml and read absorbance at 670 nm. Blanks should consist of 10 ml of sample in 90 ml of distilled water. Readings can be converted into sulphide concentrations using a standard curve.

Preparation of standard curve

Dissolve 0.75606 g of $\text{Na}_2\text{S} \cdot 9\text{H}_2\text{O}$ in distilled water and make up to 100 ml in a volumetric flask. This stock solution will contain

100 μg total sulphide per ml. Using this stock solution, make up the following solutions in 100 ml volumetric flasks:

Stock solution (ml)	Distilled water (ml)	Reagent (a) ml	Reagent (b) ml
0.1	87.9	10	2
0.2	87.8	10	2
0.3	87.7	10	2
0.4	87.6	10	2
0.5	87.5	10	2

Reagent (b) should be added last of all. This will produce a standard curve which is linear up to an absorbance of about 0.7, i.e. within a total sulphide range of 0-50 μg per 100 ml final volume. This procedure should be carried out as quickly as possible so as to avoid oxidation of the sulphide in the stock solution.

Annex III

Environmental impact of WSP systems

This Annex reproduces, as a checklist for the environmental impact assessment of WSP systems, Section 4.3 and 4.4 of the UNEP publication *An Approach to Environmental Impact Assessment for Projects Affecting the Coastal and Marine Environment* (UNEP Regional Sea Reports and Studies No. 112).

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I. General guidelines for the preparation of an Environmental Impact Assessment Document for a sewage treatment plant for a city with between 100,000 and 1,000,000 inhabitants

A. DESCRIPTION OF THE PROPOSED PROJECT

The proposed treatment plant should be described, accompanied by plans, preferably on a scale of 1:2500, including the following:

- a) Types of sewage to be treated (industrial, domestic, agricultural).
- b) Number of inhabitants to be served by the plant.
- c) Types of clients to be served, e.g. industrial, residential, commercial, hospitals.
- d) Quantity of sewage (cubic metres per day or per year).
- e) Quality of sewage to be treated, including suspended solids (mg/litre), settleable solids (mg/litre), pH, turbidity, conductivity, BOD (mg/litre), COD (mg/litre), nitrogen, ammonia, phosphate, oil, surfactants, and heavy metals such as arsenic, cadmium, copper, lead, nickel and mercury.

- f) Method to be used in treatment of sewage.
- g) Layout of the plant (including treatment facilities and service area).
- h) Use of effluents (agriculture, recharging aquifer, disposal to sea or to nearest river).
- i) Description of the plant's recipient body of water, if any.
- j) Sludge quantity and quality.
- k) Method of sludge treatment and disposal.
- l) Chemical, physical and bacteriological characteristics of effluents such as suspended solids, settleable solids, pH, turbidity, conductivity, BOD, COD, nitrogen, ammonia, phosphate, oil, surfactants, and heavy metals such as arsenic, cadmium, copper, lead, nickel and mercury, total coliforms, faecal coliforms and faecal streptococci.
- m) Programme for operation and maintenance of sewage treatment plants.

B. REASONS FOR SELECTING THE PROPOSED SITE AND THE TECHNOLOGIES

The reasons for selecting the proposed site and the technology proposed to be applied, including the short description of alternatives which have been considered, should be provided under this section.

C. DESCRIPTION OF THE ENVIRONMENT

A description of the environment of the site without the proposed sewage treatment plant should concentrate on the immediate surroundings of the proposed project. The size of the area described will be determined by the predicted effects of the proposed plant.

- a) Physical site characteristics
 - (i) Site location on a map at a scale of 1:10,000 or 1:50,000 including residential areas, industrial areas and access roads.
- b) Climatological and meteorological conditions
 - (i) Basic meteorological data such as wind direction and wind velocity.

- (ii) Special climatic conditions such as storms, inversions, trapping and fumigation, proximity to seashore, average yearly rainfall and number of rainy days per year.
 - (iii) Existing sources of air pollution, especially of particulates and odours.
- c) Geological and hydrological conditions
 - (i) Geological structure of proposed area, including hydrology and aquifers.
 - (ii) Existing uses of water bodies and the proposed site and the quality of the water.
- d) Present land use of the site and its surroundings.
- e) Characteristics of sea area which will be recipient of discharged treated sewage
 - (i) Sea circulation, existence and characteristics of the thermocline, thermohaline structure, dissolved oxygen and nutrients concentration, microbial pollution, fishing grounds, aquaculture sites, marine habitats.
- f) Existence of endemic water borne bodies.

D. IDENTIFICATION OF POSSIBLE IMPACTS

An assessment of anticipated or forecasted positive or negative impacts, using accepted standards whenever possible, of short term impacts associated with the activities related to the construction of the plant and long term impacts related to the functioning of the treatment plant should be given, including the following.

- a) Odours and air pollution from the plant and from the disposal of effluents and sludge.
- b) Infiltration of sewage into topsoil, aquifer or water supply and impact on drinking water quality.
- c) Mosquito breeding and diseases transmitted by mosquitoes.
- d) Pollution of water bodies such as rivers, lakes or sea by effluents and impact on bathing water quality.
- e) Flora and fauna.
- f) Fruit and vegetable safety, if land disposal of effluent or sludge.

- g) Noise levels around plant and its sources.
- h) Solid waste disposal of sludge and other wastes.
- i) Devaluation of property values.
- j) Tourist and recreation areas such as nature reserves, forests, parks, monuments, sports centres, beaches and other open areas which would be impacted.
- k) Possible emergencies and plant failure, the frequency at which they may occur, and possible consequences of such emergencies.
- l) Anticipated or foreseeable impacts on the areas outside of national jurisdiction.

E. PROPOSED MEASURES TO PREVENT, REDUCE OR MITIGATE THE NEGATIVE EFFECTS OF THE PROPOSED PLANT

This section should describe all measures – whether technical, legal, social, economic or other – to prevent, reduce or mitigate the negative effects of the proposed sewage treatment plant.

F. PROPOSED PROGRAMME FOR MONITORING OF THE ENVIRONMENTAL IMPACT OF THE PROJECT

Measures to be used to monitor the effects on a long term basis, including the collection of data, the analysis of data, and the enforcement procedures which are available to ensure implementation of the measures.

II. General guidelines for preparation of an Environmental Impact Assessment document for a sewage treatment plant for a city with between 10,000 and 100,000 inhabitants

These are a slightly simplified version of I above. The principal differences are noted below:

Section A

Items (e) and (l) are less extensive, as follows:

- e) Quality of sewage to be treated, including suspended solids (mg/litre), settleable solids (mg/litre), pH, turbidity, conductivity, BOD (mg/litre), COD (mg/litre), nitrogen and oil.
- l) Chemical, physical and bacteriological characteristics of effluents such as suspended solids, settleable solids, pH, turbidity, BOD, COD, nitrogen and oil.

Section C

Item (a)(i): map scale to be 1:10,000

Section D

Short-term impacts associated with plant construction do not have to be included, and item (l) is excluded.