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^{o} 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 sewered 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:

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

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, g/m^{3}d), which is given by:

$$\lambda_V = L_i Q / V_a \tag{4.2}$$

where L_i = influent BOD, mg/l (= 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 g/m^{3}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 g/m^{3}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 SO_4/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:

$$\theta_a = V_a/Q \tag{4.3}$$

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^3d)	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/m3/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, longtitude 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 < 10ºC) (Bucksteeg, 1987), and in northeast Brazil at 25ºC (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 25ºC (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 g/m^3 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 g/m^3 d for the first and second pond respectively, were able to reduce the BOD of distillery wastewaters from 40,000 m^3/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:

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

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:

$$\lambda_S = 375 - 6.25\ \mathbf{L} \qquad (4.5)$$

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:

$$\lambda_S = 60\ (1.099)^T \tag{4.6}$$

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):

$$\lambda_S = 20T - 120 \tag{4.7}$$

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

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

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.

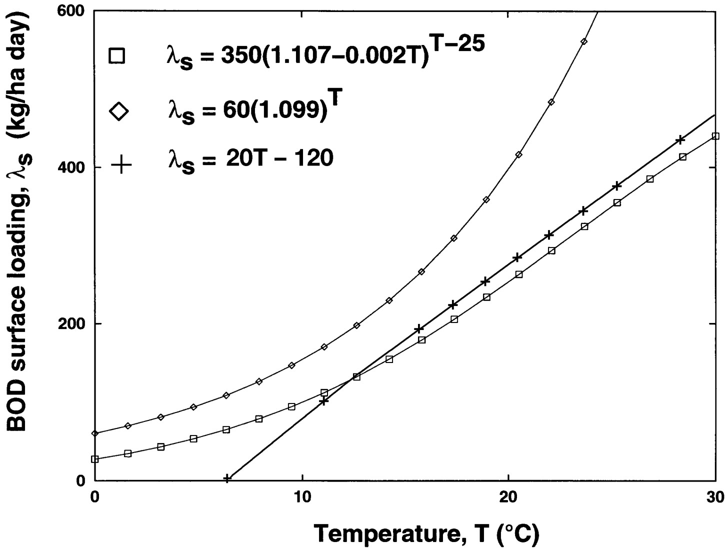

Figure 4.1 Variation of surface BOD loading on facultative ponds with temperature according to equations 4.6 – 4.8.

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

T (°C)	λ_S (kg/ha d)	T (°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:

$$\theta_f = A_f D / Q_m \qquad (4.9)$$

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:

$$\theta_f = A_f D/[{}^1/_2(Q_i+Q_e)] \qquad (4.10)$$

If seepage is negligible, Q_e is given by:

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

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

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

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:

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

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:

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

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 $[(1+k_T\theta_{m1})\,(1+k_T\theta_{m2})...(1+k_T\theta_{mn})]$).

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(day^{-1})	T(°C)	k_T(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	0.09	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:

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

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

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

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

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

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.*, 1992*a*) has yielded the following relationship (see Figure 4.2), which is equally valid for anaerobic, facultative and maturation ponds:

$$R = 100\,[1 - 0.14\exp(-0.38\theta)] \tag{4.19}$$

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

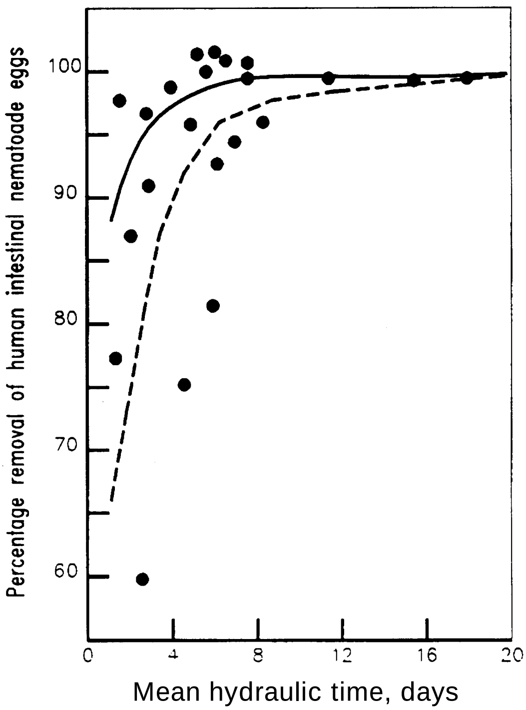

Figure 4.2
Variation of percentage helminth egg removal with retention time. Solid line: equation 4.19; dotted line, equation 4.20.

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

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

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 (NH_3 + NH_4^+) removal in individual facultative and maturation ponds. Their equation for temperatures below 20°C is:

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

and for temperatures above 20°C:

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

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:

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

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:

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

where **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 in 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 25ºC.

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, g/caput day

λ_v = volumetric BOD loading, g/m^3/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:

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

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:

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

where A_f = facultative pond area, m^2/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:

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

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 g/m^3d, 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.