OPTIMIZATION OF DISPERSED FLOW POND FOR BACTERIAL REMOVAL

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ABSTRACT

The design of waste stabilization ponds is seldom done with consideration of optimum pond dimensions. The analysis of pond dimensions within practical limits in dispersed flow ponds on the basis of minimum cost for the predetermined bacterial removal efficiency, with the aid of computer is presented. The data collected in the pilot and full scale waste stabilization ponds at the University of Dar es Salaam was used as input to the computer program. The results shows that pond configuration and climatic conditions which have influence on bacterial mortality rate have a remarkable influence on pond cost.

KEY WORDS

Bacterial removal, computer aided optimization, waste stabilization ponds, bacterial mortality rate, dispersed flow ponds, tropical climates.

INTRODUCTION

Waste stabilization ponds are looked on as presenting the sanitary engineer with the cheapest and efficient means of domestic wastewater treatment. Waste stabilization ponds are relatively inexpensive and they do not require sophisticated technology and artificial energy sources. These criterias has been advocated for their adaptation in tropical and sub-tropical developing countries where climate is favourable for their application.

The experience in Tanzania has shown that many of the waste stabilization ponds deteriorate because of inadequate maintenance. Many other planned sites have never been constructed because of inadequate funding. The optimization of pond cost during the design stage is therefore of considerable importance in developing countries like Tanzania where inadequate funding is considered a bottleneck in most development projects.

In this paper, vertically mixed dispersed flow ponds which represent the most common flow regime have been chosen to demonstrate cost minimization of waste stabilization ponds. Because of the nature of equations involved, computer programme in PASCAL was developed to aid the optimization process. The data collected between 1987 and 1991 at the University of Dar es Salaam pilot and full scale sewage ponds were used as design parameters.

DISPERSED FLOW EQUATIONS

The hydraulic flow regime through biological reactors are normally approximated to either near plug flow conditions or near complete mixing regime. In actual practice it is not easy to build and operate ponds with no mixing in the axial direction (dispersion number, d = 0) which are termed as plug flow and a reactor with complete mixing of the contents (dispersion number, d = infinite) which are known as completely mixed reactors. The hydraulic flow regime between these two extremes is the most common in biological reactors especially in waste stabilization ponds.

In dispersed flow ponds in which first order biological reactions occurs, the bacterial mortality is given by the Wehner-Wilhelm (1956) equation as shown by equation 1:

$$\frac{N_e}{N_i} = \frac{4\ a\ e^{(1/2d)}}{(1+a)^2\ e^{(a/2d)} - (1-a)\ e^{(-a/2d)}} \qquad \text{--- eq. 1}$$

where: N_e = effluent bacterial concentration (no./100 ml)

N_i = influent bacterial concentration (no./100 ml)

d = dimensionless dispersion number

$$d = \frac{D}{U\ L_p} \qquad \text{--- eq. 2}$$

D = axial dispersion coefficient (m^2/h)

U = fluid velocity (m/h)

L_p = length of fluid travel path from inlet to outlet (m)

$$a = [1 + 4\ k\ d\ t]^{1/2} \qquad \text{--- eq. 3}$$

t = mean detention time (d)

k = first order bacterial die-off rate constant (d^{-1})

The dispersion number, d, depends on the degree of mixing of the pond contents. If the mixing is negligible, the dispersion number approaches zero (plug flow ponds) and

$$\frac{N_e}{N_i} = e^{-kt} \qquad \text{--- eq. 4}$$

But if complete mixing is attained in the pond, dispersion number approaches infinite (completely mixed ponds) and

$$\frac{N_e}{N_i} = \frac{1}{1 + k\ t}. \qquad \text{--- eq. 5}$$

The dispersion number may best be obtained by tracer studies. However, for ponds in the design stage, dispersion number may be predicted using Polprasert and Bhattarai (1985) semi-empirical equation given by equation 6:

$$d = \frac{E\ [t\ v_T\ (B + 2H)]^{0.489}\ W^{1.511}}{(L\ H)^{1.489}} \qquad \text{--- eq. 6}$$

where: B = breadth of the pond (m)

v_T = kinematic viscosity of sewage at temperature T (m^2/d)
L = length of the pond (m)
E = constant, depending on configuration of model pond.

DEPTH DEPENDENT BACTERIAL MORTALITY EQUATION

The bacterial mortality rate density, k, is defined by equation 7:

$$k = k_d + \frac{k_s S_o (1 - l_c)}{K H} [1 - e^{-KH}] \qquad \text{--- eq. 7}$$

where: H = liquid depth in the pond (m)
S_o = daily average solar intensity received at the pond surface (cal/cm^2 d)
K = light attenuation coefficient (m^{-1})
k_d = bacterial die-off rate constant in the dark (d^{-1})
k_s = bacterial light mortality term constant (cm^2/cal)
l_c = the surface layer effect coefficient.

The bacterial mortality rate shown in equation 7 depends on pond depth and climatic conditions of a particular area such as solar intensity and temperature.

The term e^{-KH} in equation 7 is generally small for ponds in the practical depth. For ponds greater than 0.9 m, e^{-KH} can be neglected with an error of less than 1%. Similarly the term l_c was reported by Qin et al. (1991) to range from 0 to 0.03 and may therefore be neglected. Because of these reasons, for the pond depth greater than 0.9 m, equation 7 may be simplified to:

$$k = k_d + \frac{k_s S_o}{K H} = k_d + k_I \qquad \text{--- eq. 8}$$

where k_I = bacterial die-off rate constant in the light (d^{-1})

The data obtained from the pilot scale studies carried out between

1987 and 1988 by Mayo (1988) at the University of Dar es Salaam indicates that the faecal coliform mortality rate follows equation 9:

$$k = 0.108 + 5.79 \times 10^{-4} \left(\frac{S_o}{H}\right) \qquad \text{--- eq. 9}$$

Further experimentation was carried out by the author in the field scale ponds which comprises of three facultative ponds and four maturation ponds at the University of Dar es Salaam between 1988 and 1991. Results very closely comparable to those of pilot scale pond were obtained. Faecal coliform mortality rate obtained from the combined data from field scale and pilot scale ponds was found to be represented by equation 10:

$$k = 0.120 + 5.50 \times 10^{-4} \left(\frac{S_o}{H}\right) \qquad \text{--- eq. 10}$$

Solar intensity was found to contribute between 60 and 75% of the total faecal coliform mortality rate for the practical pond depth range of 1 to 2 m. The bacterial mortality rate in the darkness k_d was found by Mayo (1989) to have a decreasing trend with increasing temperature although no mathematical model was developed.

POND COST EQUATION

Whereas the above given equations are sufficient to determine the pond dimensions, they do not take into account the effect of cost on the pond dimension. Sarikaya and Saatci (1988) suggested a modified cost function M_c defined by equation 11 to be used for determination of optimum pond dimensions:

$$M_c = \frac{C_Q}{C_A} = t\left(\frac{1}{H} + R\right) \qquad \text{--- eq. 11}$$

where: M_c = modified cost function (d/m)

C_Q = cost per unit flow rate (Sh./m^3 s^{-1})

C_A = cost per unit area which includes cost of land, lining and levelling (Sh./m^2)

R = ratio of unit costs ($R = C_V/C_A$) (m^{-1})

C_V = cost per unit volume which includes cost of excavation and haulage.

The optimum pond dimensions were obtained by solving a combination of equation 1 through 11. However, the nature of the equations will make solving by calculator cumbersome since it involves trial and error and iteration process. A computer programme using PASCAL was therefore developed to aid the design process.

Some of the inputs required for the equations above are dependent on climatic conditions, wastewater characteristics etc. and ought to be predetermined by physical measurements. It is recommended to collect data from existing field scale ponds or running pilot scale studies if design equations suitable for local climatic conditions are not readily available. Typical design values which may be used for the design of domestic sewage ponds in climatic condition similar to those of Dar es Salaam are shown in table 1.

Table 1: Values of Parameters to be Used for The Pond Design.

Parameter	kd	ks/K	So	R	K
Unit	per day	sq.cm/cal	cal/sq.cm per day	per m	per m
Value	0.108 to 0.12	5.5E-4 to 5.79E-4	340 to 675	0.12 to 2.0	7.8 to 16
Reference	Mayo, 1988; Mayo, 1991			Mayo, 1990	Moeller & Calkins, 1980

With the known values of parameters given in table 1, some of the inputs such as pond depth H, and hydraulic detention time t, were assumed for iteration purposes. The graph of modified cost function M_c, against pond depth H, was then drawn and the optimum pond depth

H_{opt} at a point where M_c is minimum was determined.

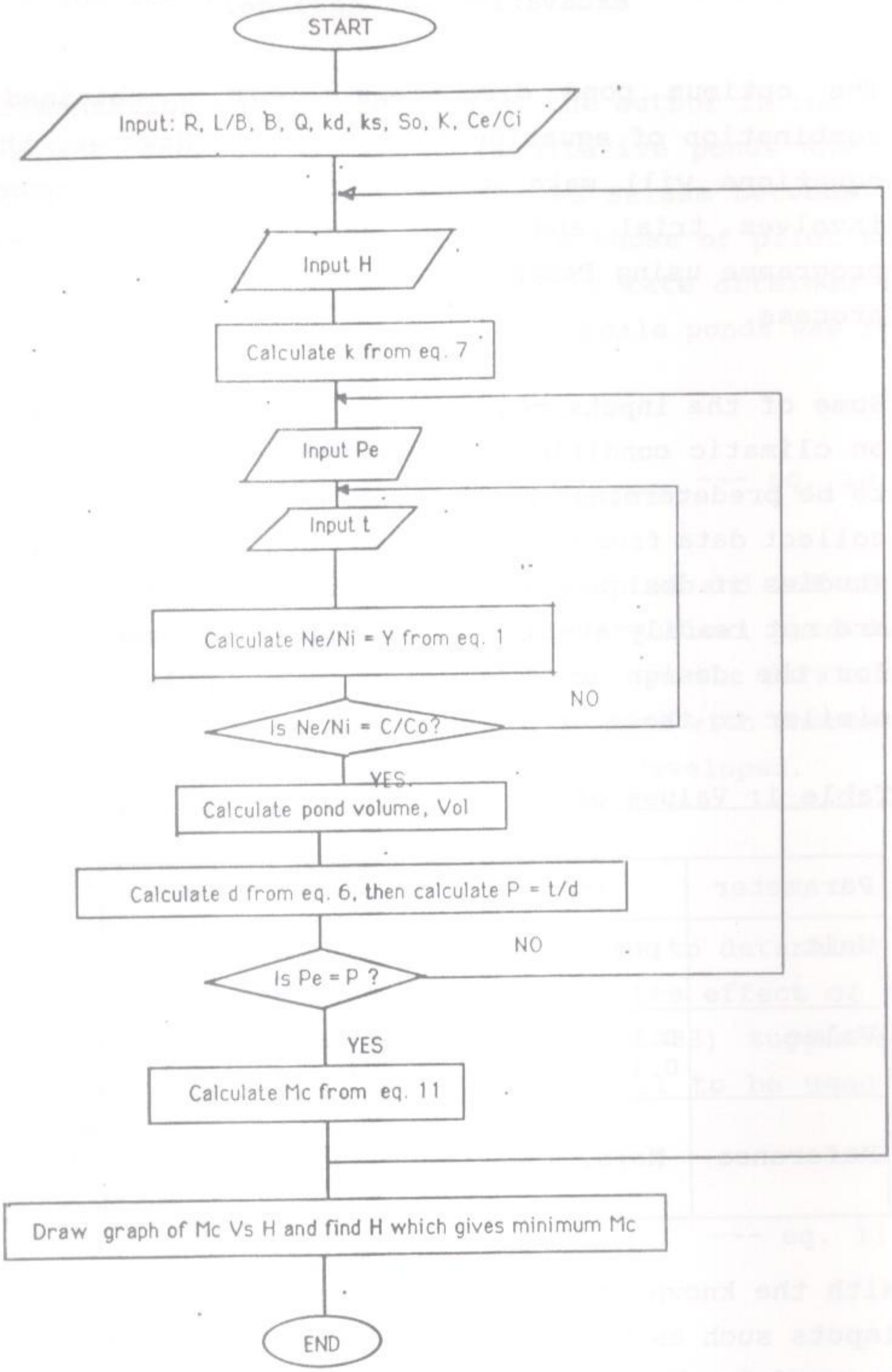

Fig. 1: Design flow chart with the aid of computer.

DISCUSSION

Effect of the Ratio of Unit Cost

The effect of the ratio of unit costs, R, in dispersed flow ponds behaves in a similar manner as for plug flow ponds regime (Mayo, 1990). As the ratio of the cost per unit volume C_V to the cost per unit area C_A increases, the pond cost increases as shown by figure 2.

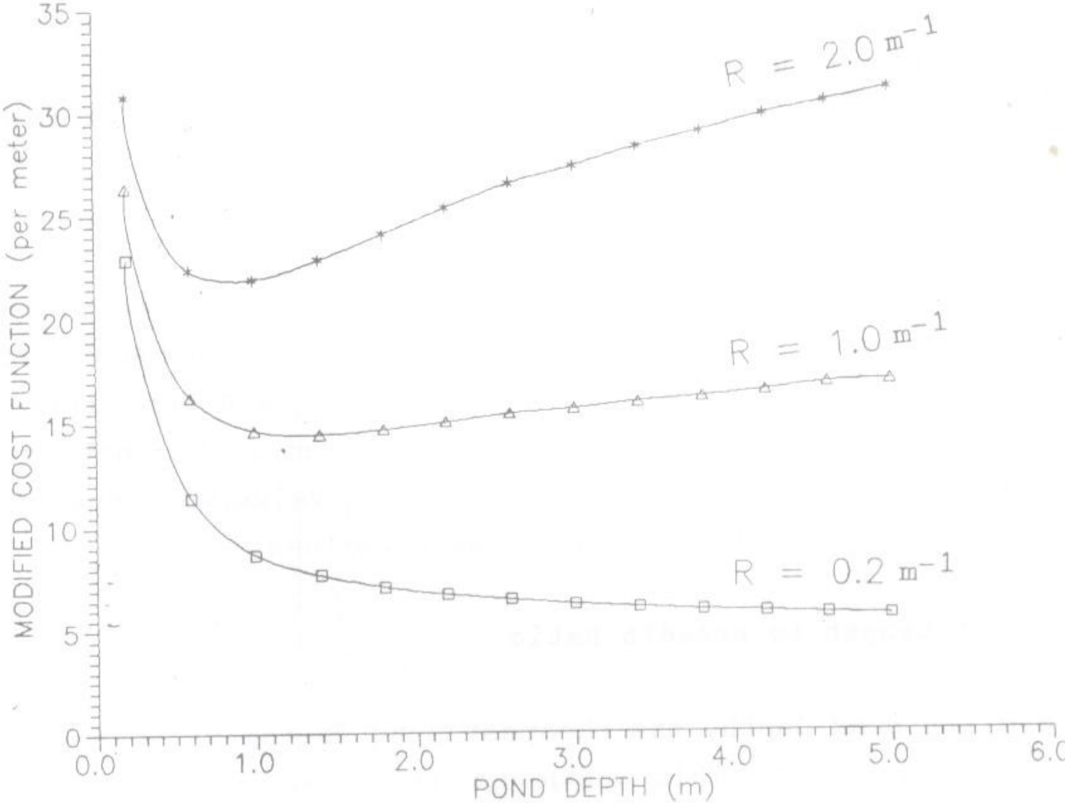

Fig. 2: Effect of Pond depth and Ratio of unit costs on Pond Cost for k_d = 0.108 d^{-1}, k_s = 0.006948 cm^2/cal, K = 12 m^{-1}, S_o = 500 cal/cm^2 d, N_e/N_i = 0.1, Q = 300 m^3/d, L/B = 1.5, Visc. = 0.86 m^2/s, E = 0.184.

As the ratio of cost per unit volume to cost per unit area R increases, the optimum pond depth decreases as depicted in figure 2. Within the pond depth used in practice, optimum pond depth is obtained at higher R values. At low R values the pond cost does not considerably vary at pond depths greater than 1.4 m.

For $R = 0.2\ m^{-1}$, the pond cost decreases with increasing pond depth, but is more sensitive at shallow ponds than in deep ponds. In general, for $R = 0.2\ m^{-1}$, pond depths between 1.8 and 5.0 m may practically be used with a difference in cost of only 15%. Thus, within this range, the increase in excavation and haulage cost is more or less offset by the decrease in cost related to reduction in land area.

The depths of facultative and maturation ponds used in practice varies from 1.0 to 1.5 m. This rule of thumb does not mean these pond depths are always optimum. For $R = 0.2\ m^{-1}$ for instance, the cost of 1.0 m deep pond is about 56% more expensive than that of 5.0 m deep pond as shown by figure 2.

It should however, be noted that the value of R depends on ratio of C_V to C_A values but the actual cost of pond will depend on absolute values of C_A and C_V. Contractors with the same R but different C_A and C_V values have the same value of modified cost function M_c but different total pond cost. Comparison of M_c value will therefore be realistic when compared for the same C_A values.

Effect of Length to Breadth Ratio

For a given pond depth, the cheapest pond is the narrowest pond as shown by figure 3. This suggests than narrow ponds are more effective than wide ponds. Narrow ponds are considered more effective since the risk of short-circuiting is normally reduced and hydraulic characteristics are better resulting in low dispersion number, and therefore higher removal efficiency of bacteria. Because of higher efficiency, less hydraulic detention time is required for the same effluent quality and thus less pond surface area and volume is required as shown by figures 4 and 5.

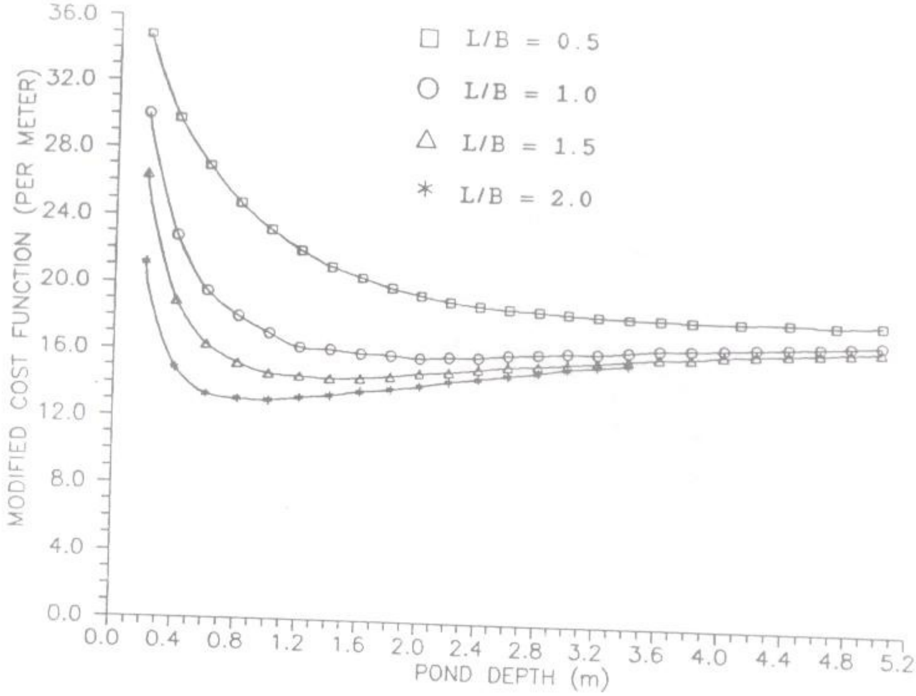

Fig. 3: Effect of Pond depth and L/B ratio on Pond cost for k_d = 0.108 d^{-1}, k_s = 0.006948 cm^2/cal, K = 12 m^{-1}, S_o = 500 cal/cm2 d, N_e/N_i = 0.1, Q = 300 m^3/d, R = 1.0 m^{-1}, E = 0.184, Visc. = 0.86 m^2/s.

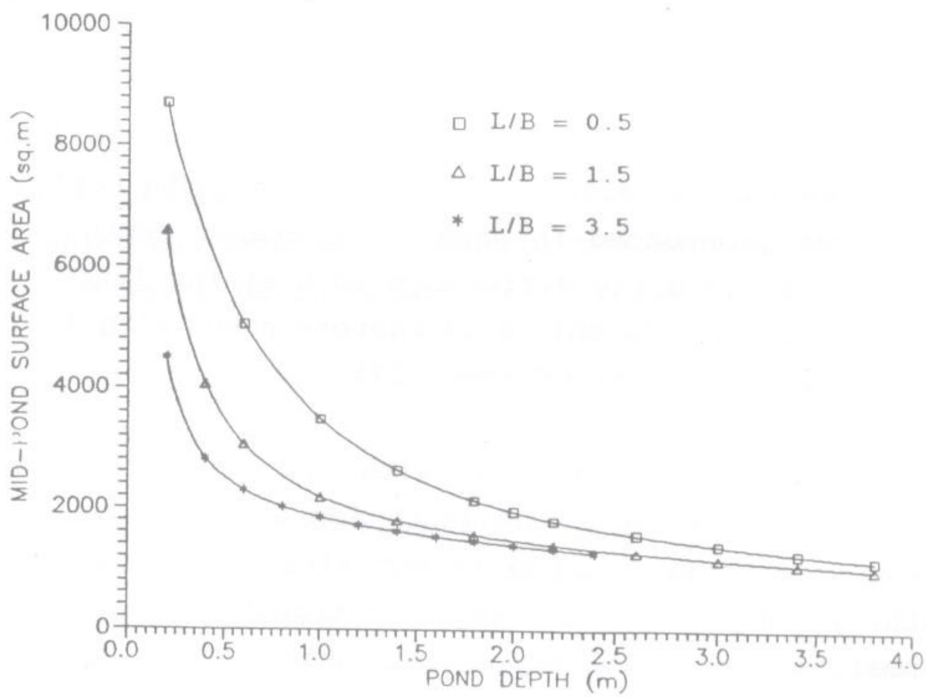

Fig. 4: Effect of Pond depth and L/B ratio on Pond surface area for k_d = 0.108 d^{-1}, k_s = 0.006948 cm^2/cal, K = 12 m^{-1}, S_o = 500 cal/cm^2 d, N_e/N_i = 0.1, R = 1.0 m^{-1}, Q = 300 m^3/d, E = 0.184, Visc. = 0.86 m^2/s.

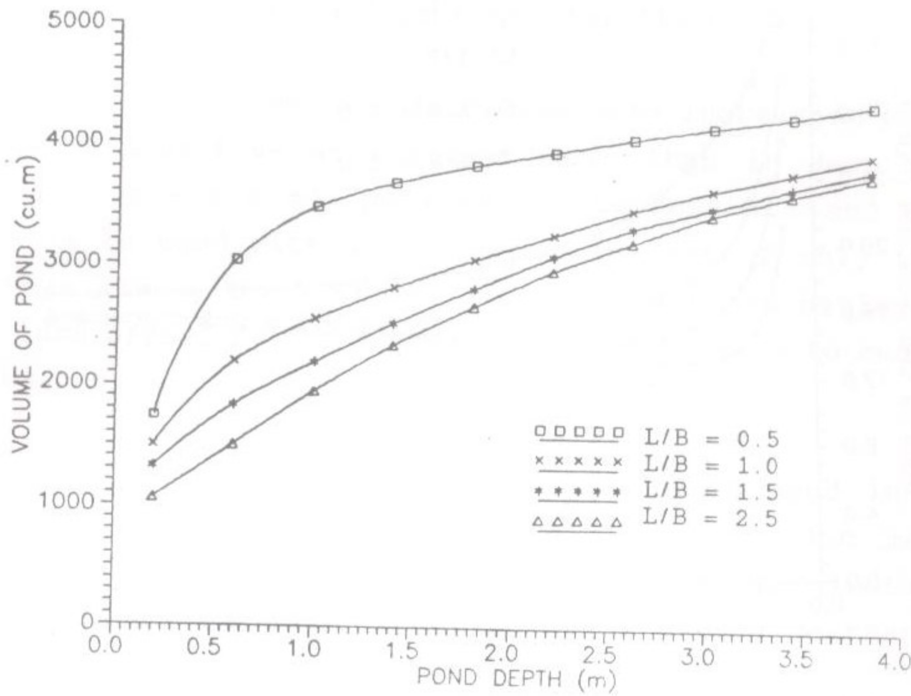

Fig. 5: Effect of Pond depth and L/B ratio on Volume of pond for $k_d = 0.108\ d^{-1}$, $k_s = 0.006948\ cm^2/cal$, $K = 12$, $S_o = 500\ cal/cm^2\ d$, $N_e/N_i = 0.1$, $R = 1.0\ m^{-1}$, $Q = 300\ m^3/d$, $E = 0.184$, Visc. $= 0.86\ m^2/s$.

The cost of pond is more sensitive at depths lower than optimum pond depth. The effect of length to breadth ratio on the pond cost is less pronounced in ponds with deeper depths. For instance, at depth of 2.4 m the difference in cost for length to breadth ratio of 1.0 and 3.5 is only 8.9% whereas ponds with depth of 1.2 m, have difference in cost of about 27%.

As depicted by figure 6, with the increasing length to breadth ratio, the optimum pond depth decreases. Figure 6 also shows that the optimum pond depth is not always the practical solution. For the given parameters, length to breadth ratio exceeding 2.5 require shallow ponds with depth less than 1.0 m, which are unfavourable due to risk of weed growth. At the same time theoretically lower detention times of about 2 to 4 days will be required which in practice may lead to premature cell washout and/or short-circuiting leading to poor effluent quality than predicted.

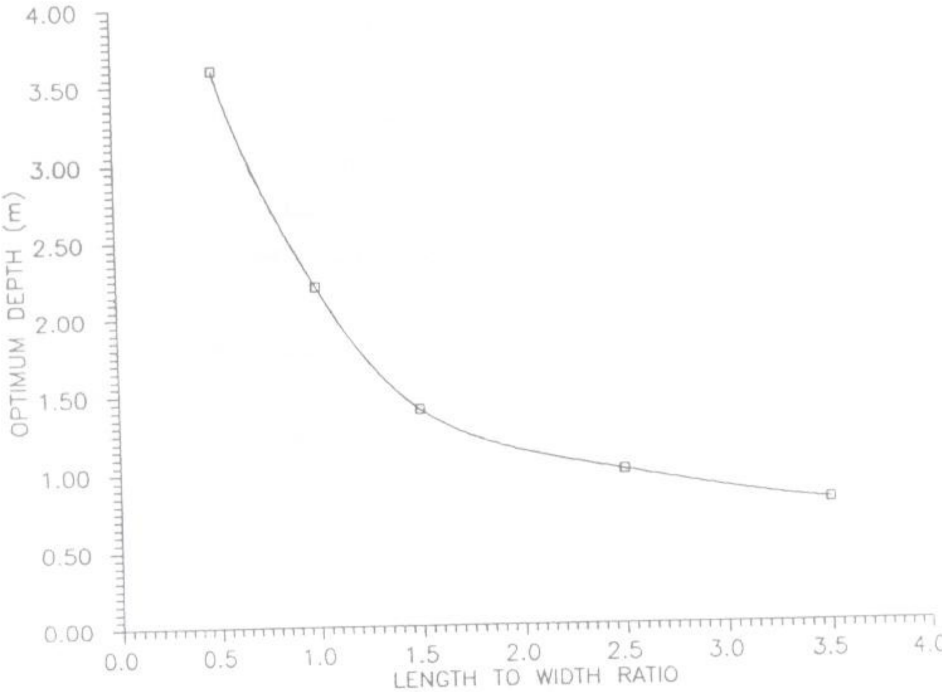

Fig. 6: Effect of L/B ratio on Optimum pond depth for $k_d = 0.108$, $k_s = 0.006948$ cm^2/cal, $K = 12$ m^{-1}, $S_o = 500$ cal/cm^2 d, $R = 1.0$ m^{-1}, $N_e/N_i = 0.1$, $Q = 300$ m^3/d, $E = 0.184$, Visc. = 0.86 m^2/s.

In practice the length to breadth ratio of 2 to 1 up to 3 to 1 are the most favourably used due to limitations of site topography, and length of fetch which may enhance erosion of the banks. Length to breadth ratio may however be increased by engineering techniques such as using baffles.

Effect of Solar Intensity

Solar intensity, the principal driving force for the photosynthetic reaction in the pond has a significant influence on the bacterial mortality rate as shown by equation 7 through 9. Inspite of its influence on bacterial mortality rate mans' ability to influence solar intensity is minimum. However, ponds may be designed at optimum conditions to decrease the cost.

Ponds with shallow depth have high bacterial mortality rate than deep ponds. Shorter hydraulic detention times will therefore be required in shallow ponds resulting into low pond volume but large area of land. Conversely, in deeper ponds light penetration will be hindered by algae layer near the surface of the pond water body, the turbidity, scum layer, and absorption of light by water body. The bacterial mortality rate in deep ponds will therefore be low thus requiring longer hydraulic detention times. Although there will be serving of land space, the increase in pond volume will lead to increase in excavation and haulage costs. The optimum pond dimensions is therefore between these two extremes where the decrease in cost due to reduction in surface area is offset by increase in excavation and haulage costs as shown in figure 7.

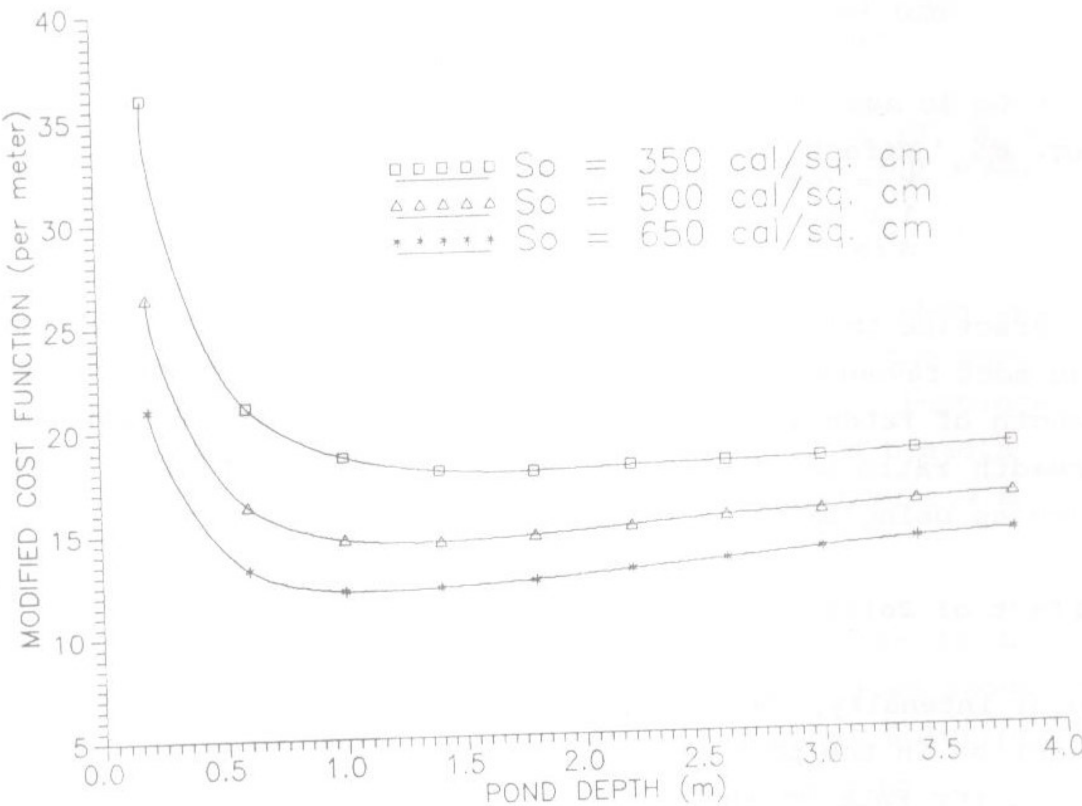

Fig. 7: Effect of Pond depth and Solar intensity on Pond cost for k_d = 0.108 d^{-1}, k_s = 0.006948 cm^2/cal, K = 12 m^{-1}, N_c/N_i = 0.1, R = 1.0 m^{-1}, Q = 300 m^3/d, L/B = 1.5, E = 0.184, Visc. = 0.86 m^2/s.

Some Design Considerations

Given that a community of 7000 people in Dar es Salaam is to be provided with ponds as the sole wastewater treatment facility. The mean solar intensity in Dar es Salaam is about 500 cal/cm^2/d and the mean water temperature is roughly 23 - 28°C.

The per capita water consumption is estimated to be 58 lcd (Meghji and Merinyo, 1989). If 75% of water is discharged as wastewater, the influent discharge, Q_{inf}, to waste stabilization ponds is

$$Q_{inf} = 0.75 \times 0.058 \times 7000 = 300 \ m^3/d$$

If the influent faecal coliform density is estimated to be 10^7 per 100 ml and that effluent is intended to be used for unrestricted irrigation, ie FC < 100/100 ml, then 5 facultative/maturation ponds in series will be required to remove bacteria by 90% in each pond. Notice that the serial effect is assumed to have no influence on bacterial mortality rate in dispersed flow ponds approaching plug flow conditions. If R = 1.0 m^{-1} and K = 12 m^{-1}, optimum conditions shown in table 2 can be considered.

Table 2: Possible Alternative Pond Dimensions

L/B ratio	Hopt (m)	Mc (per m)	Area (sq.m)	Volume (cu.m)	Detention time (d)
0.5	2.6	18.83	1569	4080	13.6
1.0	2.0	15.75	1575	3150	10.5
1.5	1.4	14.40	1800	2520	8.4
2.5	1.0	13.00	1950	1950	6.5
3.5	0.8	12.15	2025	1860	5.4

The following can be concluded from table 2:

- optimum pond depth for length to breadth ratio > 2.5 will be too shallow and thus will attract weed growth.
- low length to breadth ratio require small area of land. Thus

length to breadth ratio of 0.5 requires 24% less land than that of 2.5.
- high length to breadth ratio requires small pond volume. Thus length to breadth ratio of 0.5 requires 52% more pond volume than that of 2.5.
- low length to breadth ratio is more costly. For instance, length to breadth ratio of 0.5 is 24% more expensive than that of 2.5.

High L/B ratio appears to be more attractive option than low L/B ratio because it is cheap and requires less excavation and haulage of earth. However, if land space and site topography are the predominant selection criteria, a low L/B ratio may be chosen with improvement of hydraulic conditions. By introducing multiple inlet/outlet structures and baffles, the area characterized by dead zones in the pond will be reduced and therefore hydraulic and treatment performance of the pond is expected to improve.

It is not always necessary to consider the optimum pond depth as a selection criteria. Table 2 suggests that cheapest pond is the one with L/B = 3.5, but its optimum pond depth of 0.8 m is practically unfavourable. If the pond depth is increased to 1.0 m, the modified cost function will increase to 12.4 m^{-1} which is still cheaper than optimum pond depth for L/B = 2.5.

The designer should expect optimum depth of less than 1.0 m for high R values (eg. R = 2 m^{-1}) which are practically unsuitable and may therefore consider depths other than optimum. At the same time, for low R values (eg. R = 0.2 m^{-1}) ponds as deep as 5 m may be found optimum.

The analysis given assumes that the excavation cost is independent of the pond depth. In practice, the cost of excavation is a function of depth because deep excavations are more technically complicated and would therefore be more expensive. Whereas these theoretical consideration appear to be a useful tool in dimensioning ponds, practical aspects such as knowledge of site topography, soil conditions, project duration etc. should be taken into consideration by the designer in his final choice.

SUMMARY AND CONCLUDING REMARKS

The computer aided optimization given in this paper is a useful tool in selection of dimensions of waste stabilization ponds. The designer is advised to be careful in selection of proper design parameters as unrealistic values or assumptions may lead to serious design errors and high construction costs.

Depending on the design inputs, the optimum pond depth may either be shallow or deep. This means that the rule of thumb used in practice does not always result in optimum design. On the other hand, the optimum pond dimensions do not always represent practically suitable option. Much attention should preferably be devoted to hydraulic characteristics of ponds which influence the biochemical activities and plays a major role in treatment performance of waste stabilization ponds.

The designer's final decision is however expected to put into consideration local aspects. These include desired project completion time, site topography, soil and ground water conditions and available land space just to mention a few.

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