Design of waste stabilization ponds for dairy processing plants in Uganda

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Abstract: Waste stabilization ponds (WSP) were designed to treat 287.5 m^3 day^{-1} of wastewater generated from processing of 100 m^3 of milk per day. The design involved use of existing models including those developed by Mara to size the anaerobic, facultative and maturation ponds. The design temperature was 25^0C. The anaerobic pond was designed based on volumetric organic loading rate while facultative pond was designed based on surface loading rate. On the other hand, the maturation pond was designed based on the number of coliform bacteria removed per 100 mL of wastewater. The anaerobic pond was designed to remove 70% BOD, facultative pond-75% and maturation pond-25% BOD. In addition, the maturation pond was designed to have a coliform bacteria removal efficiency of at least 99%. The total land requirement for anaerobic pond was estimated at 945.19 m^2, facultative pond-6361.54 m^2 and three maturation ponds-2709.06 m^2. To cater for pond operation and maintenance, an additional 25% land was incorporated resulting into 1.25 hectares as the total land area required for pond construction, operation and maintenance. Besides treatment of wastewater to reduce BOD, remove pathogens and other pollutants, the use of WSP can result into high economic benefits through recycling of wastewater for agriculture and aquaculture. As a result, the payback period for the investment cost may also be shortened.

Keywords: milk, waste stabilization ponds, BOD, anaerobic, facultative, maturation

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1 Introduction

In the dairy processing industry, fresh water is used principally for cleaning equipment and work areas to maintain hygienic conditions, in cooling departments such as cooling towers and in energy production such as boilers (UNEP, 2000). Water also accounts for a large proportion of raw material in the reconstitution of milk powders for the production of liquid milk, yoghurt, ice cream, butter and cheese. According to UNEP (2000), rates of water consumption vary depending on the scale of the plant, age and type of processing, ease with which equipment can be cleaned, and operator practices. Dairy-plant maintenance is water-intensive with typical water consumption rate of 1.3 to 2.5 L of water per kilogram of milk intake (UNEP, 2000). Wastewater is generated as the clean-in-place (CIP) systems continuously wash every plant tank, pipe and drains to ensure that microorganisms from the dairy plant do not contaminate the milk. Like other industries, the wastewater from the dairy industry poses environmental problems like water and soil pollution due to the high amounts of nutrients, organic matter and pathogens (DFID, 1998).

The major pollutants in wastewater discharges from milk based food industry are organic matter, suspended solids, pH, nitrogen, phosphorus, and fats (Pooja, 2008). The organic substances in dairy wastewaters come from primarily, the milk and milk products wasted and to a much lesser degree by cleaning products, sanitizing compounds, lubricants and domestic sewage that are discharged to the waste stream (Pooja, 2008). According to World Bank (2007), the main parameter of concern for effluent permit at a dairy plant is Biological Oxygen Demand (BOD), which is one of the parameters

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that give an indication of the concentration of organic compounds in wastewater. BOD is the amount of oxygen required by microorganisms to oxidize the organic material in the wastewater (Hamzeh and Ponce, 1999). Cream, butter, cheese, and whey production are major sources of BOD in wastewater (World Bank Group, 1998). According to Wendorff (1998), the raw wastewater from dairy facilities has a typical BOD of 2,500 mg L^{-1}. Table 1 shows effluent limits of common pollutants from the dairy industry. Effluent limits represent the maximum amount of pollutants allowed to discharge from wastewater to its final destination hence, prior to design, these limits must be known since they are used as the water quality design objectives (Hamzeh and Ponce, 1999).

Table 1 Effluents from the dairy industry and their limits (World Bank, 2007)

Parameter	Maximum permissible value
pH	6-9
BOD (mg L^{-1})	50
COD (mg L^{-1})	250
TSS (mg L^{-1})	50
Oil and grease (mg L^{-1})	10
Total nitrogen (mg L^{-1})	10
Total phosphorus (mg L^{-1})	2
Temperature increase (^{0}C)	≤ 3
Coliform bacteria (MPN/100 mL)	400

Note: Effluent requirements are for direct discharge to surface waters.
MPN stands for: most probable number.

According to DFID (1998), waste stabilization ponds (WSP) are very often the most cost-effective wastewater treatment method since they can be built and repaired using locally available materials, no external energy required for operation, low in construction costs, very low operating costs, high reduction in pathogens, can treat high-strength wastewater to high quality effluent, generally reliable and function well. The effluent can also be reused in aquaculture or for irrigation in agriculture (Shuval et al. 1986). However, their major disadvantage is that availability of large areas of land far away from homes and public spaces is required (DFID, 1998). Anaerobic ponds require approximately 4 m^2 m^{-3} daily flow and facultative aerobic ponds require 25 m^2 m^{-3} daily flow (Sasse, 1998). WSPs make use of the sun, wind, gravity, and biological activity to achieve treatment (Hamzeh and Ponce, 1999). The principles behind WSP operation are simple and they place no strain on technical resources or labour (WHO, 1987). However, both the process design and the physical design of WSPs have to be carried out very carefully by competent design engineers to ensure effectiveness and efficiency (Varon, 2004).

According to Hamzeh and Ponce (1999), WSPs can be classified in respect to the type(s) of biological activity occurring in a pond. The ponds include: anaerobic, facultative and maturation ponds.

The anaerobic pond is suitable for wastewater with BOD greater than 100 mg L^{-1} day^{-1} (Hamzeh and Ponce, 1999). Methanogenic bacteria evolve to thrive in oxygen depleted conditions (i.e. anaerobic), as they break down organic material ultimately into methane and carbondioxide gas (Sperling, 2007).

The facultative pond reduces BOD by both aerobic processes at the pond surface and anaerobic processes at the bottom (Sperling, 2007). The pond is characterized by algae growth which helps to produce oxygen in the pond for the aerobic bacteria during the photosynthetic process. The soluble BOD is aerobically stabilized and suspended and colloidal BOD tends to settle and is decomposed by anaerobic bacteria (Sperling, 2007). In total, about 70% to 85% of the incoming BOD can be removed across the facultative pond (Marrais, 1987).

The maturation pond is generally shallower than other types of ponds to allow light penetration to the bottom and aerobic conditions throughout the whole depth and ensure that there are substantial amounts of treatment of the wastewater. The maturation pond removes pathogens and fecal coliform by the oxidation process. Maturation ponds are required only when stronger wastewaters (BOD_5 > 150 mg L^{-1}) are to be treated prior to surface water discharge and when the treated wastewater is to be used for unrestricted irrigation (Hamzeh and Ponce, 1999).

This research was aimed at designing waste stabilization ponds for treatment of wastewater generated from processing of 100 m^3 of milk per day. The WSP design comprised of three stabilization ponds including:

anaerobic, facultative and maturation ponds. This work is intended to serve as a basis for individuals or institutions that are interested in design of similar wastewater treatment systems particularly in Uganda.

2 Materials and methods

2.1 Design of the anaerobic pond

The design of the anaerobic ponds depends strongly on the temperature (*T*) of the system along with the volumetric organic loading rate, λ_v (mg BOD L^{-1} day^{-1}) of the effluent which is the amount of BOD that the pond can treat per volume (Norton et al., 2012). The volumetric loading, λ_v (mg BOD L^{-1} day^{-1}) and BOD_5 removal efficiency were determined according to Table 2.

Table 2 Design values of permissible volumetric BOD loadings and percentage BOD removal in anaerobic ponds at various temperatures (Alexiou and Mara, 2003)

Temperature T /^{0}C	Volumetric loading, λ_v /g m^{-3} day^{-1}	BOD removal /%
<10	100	40
10-20	20T-100	2T+20
20-25	10T+100	2T+20
>25	350	70

Input Data:

Volume of milk processed = 100 m^3 day^{-1}

According to Hibbard et al. (1996), a typical dairy plant may lose as much as 2.5% of the milk it processes from spills, rinses, and clean-in-place procedures. Therefore for this design, estimated quantity of milk lost to the wastewater stream is given by: $\frac{2.5}{100}\times 100 =$ 2.5 m^3 day^{-1}

UNEP (2000) reports that water consumption rate of 1.3 to 2.5 L of water per kilogram of milk intake is typical of dairy processing plants. Therefore, the water requirement for cleaning the plant facilities was estimated by taking the average value multiplied by the plant processing capacity: i.e. $\frac{1.3+2.5}{2}\times 150 = 285$ m^3 day^{-1}

Total wastewater generated, Q_i =285+2.5=287.5 m^3 day^{-1}

According to Wendorff (1998), the raw wastewater from dairy facilities has a typical BOD_5 of 2,500 mg L^{-1}. Therefore, influent BOD, $L_i \approx 2500$ mg L^{-1}

Design temperature, *T*, was taken as the mean temperature of the coldest month so that sufficient treatment occurs throughout the whole year (Mara, 1997). *T* was taken as 25^{0}C. However, temperature varies from location to location depending on where the ponds are to be constructed. The value of 25^{0}C may therefore not necessarily be constant for all locations in Uganda.

From Table 2 above, for *T* = 25^{0}C,

$$\lambda_v = 10T + 100 \tag{1}$$

Using Equation (1), $\lambda_v = (10\times 25)+100 =$ 350 mg BOD L^{-1} day^{-1}

$$\text{BOD removal efficiency (\%)} = 2T+20 \tag{2}$$

Using Equation (2), BOD removal efficiency in anaerobic pond = (2×25)+20=70%

BOD of the effluent entering the second anaerobic pond, $L_e = (1-0.70)\times 2500 = 750$ mg L^{-1}

McGarry and Pescod (1970) reported that for high strength industrial wastes (i.e. 1,000 mg L^{-1} BOD_5), it might be justifiable to have a series of anaerobic ponds up to three; however, the retention time in any of the ponds should never be less than one day. For this particular design, since the BOD_5 was reduced to 750 g m^{-3}, there was no need to have another anaerobic pond before discharging the effluent to the facultative pond.

The volume of the anaerobic pond was determined using Equation (3) below:

$$V_a = \frac{L_i Q}{\lambda_v} \tag{3}$$

where, L_i is influent BOD, mg L^{-1}; Q is flow rate, m^3 day^{-1}; V_a is anaerobic pond volume, m^3, $V_a = \frac{2500\times 287.5}{350} \approx 2053.37$ m^3.

Mid-depth, D of the anaerobic pond is usually 3.5-5 meters, allowing for low oxygen level conditions to prevail (Sperling, 2007). D was taken as the average value for the range mentioned above yielding: (3.5+5)/2 = 4.25 m. However, the value of D may even be lower depending on the water table and soil conditions of the site where the ponds are to be constructed. According to Hamzeh and Ponce (1999), when choosing a site to construct a pond system, an area should be selected where the water table is deep and the soil is heavy and impermeable to avoid groundwater pollution.

Using Equation (4), the mid-surface area of the pond was obtained:

$$\text{Mid-area, } A_{an} = \frac{V_a}{D} \quad (4)$$

$$\text{Mid-area, } A_{an} = \frac{2053.57}{4.25} \approx 483.19 \text{ m}^2$$

Retention time was determined using Equation (5) as follows:

$$\text{Retention time, } t_{an} = \frac{V_a}{Q_i} \quad (5)$$

where, t_{an} is retention time in the anaerobic ponds, days; Q is flow rate, m^3 day^{-1}; V_a is anaerobic pond volume, m^3.

$$\text{Retention time, } t_{an} = \frac{2053.57}{287.5} \approx 7 \text{ days}$$

A retention time of one day is only sufficient for wastewaters with a BOD_5 ≤300 mg L^{-1} at temperatures above 20^{0}C (Mara et al., 1992). In addition, FAO Natural Resources Department (1992), reports that the ponds retention time should not be less than one day; if it occurs, however, a retention time of one day should be used, and the volume of the pond should be recalculated. As the retention time increases, typically the percentage of BOD removal will increase. A retention time of more than three days is more effective (FAO Natural Resources Department, 1992). Therefore, seven days as a retention time in the anaerobic pond of wastewater with a BOD_5 of 2,500 mg L^{-1} is just ok.

For anaerobic pond, the length-to-breadth ratio should be between 2:1 to 3:1 (Alexiou and Mara, 2003). Therefore, the length would be obtained from:

$$A = L \times W \quad (6)$$

$483.19 = 3W^2$ Since $L = 3W$

Mid-width, W = 12.69 m

Mid-Length, L = 38.07 m

The dimensions at the top and bottom levels of the pond were obtained using the pond geometry shown in Figure 1.

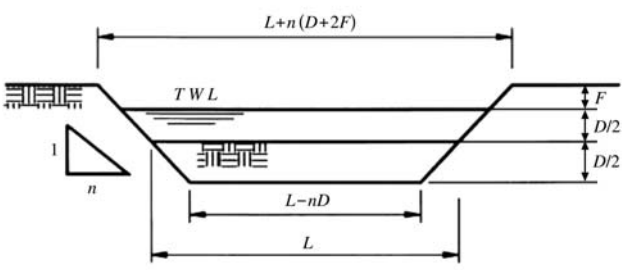

Figure 1 Pond geometry (Mara and Pearson, 1998)

Top surface dimensions

$$\text{Top length, } L = L + n(D+2F) \quad (7)$$

where, Free board, F = 50 cm; L = pond length at TWL, m; W = pond width at TWL, m; D = pond liquid depth, m; n = horizontal slope factor (i.e. a slope of 1 in n).

Using Equation (7), Top length, L = 38.07+1.5(4.25+ 2×0.5) = 45.95 m

Top width, $W = W + n(D+2F)$ (8)

= 12.69+1.5(4.25+2×0.5) =20.57 m

Using Equation (6), top surface area of anaerobic pond = $L \times W$

= 45.95×20.57 = 945.19 m^2

Bottom surface dimensions

Bottom length, $L = L - nD$ (9)

= [45.95−(1.5×4.25)]

≈39.58 m

Bottom width, $W = W - nD$ (10)

= 20.57− (1.5×4.25)

= 14.20 m

Using Equation (6), bottom surface area of anaerobic pond = $L \times W$

=39.58 × 14.20 = 561.84 m^2

It should be noted that balancing of pH in the anaerobic pond is crucial in optimizing the efficiency of anaerobic treatment. Optimal pH is between 7.0 and 7.2 (McCarty, 1964) to prevent odour problems and harming the bacteria. Therefore, pH monitoring is recommended to ensure it does not water too far from this. However, a pH range 6.6 to 7.6 is also suitable and easier to manage to help develop the methanogenic bacteria population. According to Mara and Pearson (1998), it is necessary to add lime to the pond in the first month to avoid acidification of the reactor.

2.2 Design procedure for facultative pond

Mara (1976) recommends that facultative ponds should be designed on the basis of surface loading, λ_s (kg BOD ha^{-1} day^{-1}). The selection of the permissible design value of λ_s is usually based on the temperature as shown in Equation (11) developed by Mara (1976).

$$\lambda_s = 350[1.107 - 0.002T]^{(T-25)} \quad (11)$$

$$= 350[1.107 - (0.002 \times 25)]^{(25-25)}$$

$$= 350 \text{ kg BOD ha}^{-1} \text{ day}^{-1}$$

Incoming BOD from the anaerobic pond effluent, $L_i \approx$ 750 mg L^{-1}

BOD removal efficiency in facultative pond is 70%-80% (Mara et al., 1992). Taking 75% BOD removal, the effluent BOD, L_e leaving facultative pond is given by:

$$L_e = (1-0.75)\times 750 \approx 187.5 \text{ mg L}^{-1}$$

Wastewater flow rate into the facultative pond, $Q_i = 287.5$ m^3 day^{-1}

According to FAO (2006), the mean monthly evaporation rates in Uganda are between 125 and 200 mm. Taking the average gives: (125+200)/2 = 162.5 mm $month^{-1}$

Net rate of evaporation ≈ 162.5/30 = 5.42 mm day^{-1}

Mean temperature of the coldest month, $T = 25\ ^0$C

The area of the facultative pond, A_f can be calculated from Equation (12) developed by McGarry and Pescode (1970).

$$\lambda_s = \frac{10L_iQ}{A_f} \tag{12}$$

where, L_i is the concentration of influent sewage, mg L^{-1}; A_f is the facultative pond area, m^2; Q is the influent flow rate, m^3 day^{-1};

$$A_f = \frac{10L_iQ}{\lambda_s} = \frac{10\times 750\times 287.5}{350} \approx 6160.71 \text{ m}^2$$

The wastewater flow rate, Q_e into the maturation pond can be determined using Equation (13):

$$Q_e = Q_i - 0.001A_fe \tag{13}$$

$$Q_e = 287.5 - (0.001\times 6160.71\times 5.42) \approx 254.11 \text{ m}^3 \text{ day}^{-1}$$

Retention time for the facultative pond can be determined using Equation (14)

$$\theta_f = \frac{2A_fD}{(2Q_i - 0.001A_fe)} \tag{14}$$

Marra et al. (1992) reports that the depth, D, of the facultative pond should be between 1and 2 m (commonly 1.5 m). Taking D =1.5 m, and using Equation (14), the retention time of the facultative pond is given by:

$$\theta_f = \frac{2\times 6160.71\times 1.5}{((2\times 287.5)-(0.001\times 3321\times 5.42))} \approx 33 \text{ days}$$

However, according to WSP (2007), retention time in facultative ponds should vary between 5 to 30 days. So 30 days was chosen as the retention time in the facultative pond, after which the area of the facultative pond was re-computed using Equation (15):

$$A_f = \frac{\theta_f Q_m}{D} \tag{15}$$

where, Mean flow, $Q_m = \frac{1}{2}(Q_i + Q_e) = \frac{287.5+254.11}{2}$

≈ 270.81 m^3 day^{-1}

Mid-depth, $A_f = \frac{270.81\times 30}{1.5} \approx 5416.1 \text{ m}^2$

Mid-depth volume, $V_f = A_f \times D$ (16)

$=5416.1\times 1.5 \approx 8124.15$ m^3

Mara et al. (1992) reports that the length to breadth ratio should be 2-3 to 1 if the pond receives raw wastewater. However, length to breadth ratio can be greater than 3 to 1 if pond receives anaerobic pond effluent. While selecting the length to breadth ratio, it was also important to ensure that the pond width was kept less than 36 m to cater for the reach limitations of excavator and de-sludging machinery. When designing the pond geometry, it is necessary to take into account the possibilities for the access of machinery used for de-sludging and emptying both sides of the ponds (Hamzeh and Ponce, 1999). As long as reach excavators have a maximum reach of around 18 m, adopting a maximum pond width of no more than 35 m would overcome some of the problems encountered when desludging is required. Therefore, taking length to breadth ratio of 4:1, the dimensions of length (L) and width (W) become:

From Equation (6), $A = L\times W$

$5416 = 4W^2$ Since $L = 4W$

$W \approx 36.80$ m

$L \approx 147.19$ m

The dimensions at the top and bottom levels of the pond were obtained using the pond geometry shown in Figure 1.

Top surface dimensions

From Equation (7), top length,

$$L = L + n(D+2F)$$
$$= 147.19 + 2(1.5 + 2\times 0.5)$$
$$= 152.19 \text{ m}$$

From Equation (8), top width,

$$W = W + n(D + 2F)$$

$$= 36.80 + 2(1.5 + 2 \times 0.5)$$

$$= 41.80 \text{ m}$$

From Equation (6), top surface area of facultative pond = $L \times W$

$$= 152.19 \times 41.80$$

$$= 6361.54 \text{ m}^2$$

Bottom surface dimensions

Using Equation (9), bottom length,

$$L = L - nD$$

$$= [152.19-(2 \times 1.5)]$$

$$\approx 139.19 \text{ m}$$

Using Equation (10), bottom width,

$$W = W - nD$$

$$= 41.80 - (2 \times 1.5)$$

$$\approx 38.80 \text{ m}$$

Using Equation (6), bottom surface area of facultative pond = $L \times W$

$$= 139.19 \times 38.80$$

$$= 5400.57 \text{ m}^2$$

Total land area required = 6361.54 m^2

Land requirement per flow rate = $\frac{6361.54}{287.5} \approx$ 22.13 $\text{m}^2 \text{ m}^{-3}$ flow rate. This is in agreement with Sasse (1998); the author reports that facultative aerobic ponds require 25 $\text{m}^2 \text{ m}^{-3}$ daily flow rate.

2.3 Design of maturation ponds

Input data: incoming BOD, L_i from the facultative ≈187.5 mg L^{-1} Mara and Pearson (1987) suggest 25% BOD removal in each maturation pond, for temperatures above 25^{0}C when the BOD is based on filtered BOD values. Thus the BOD of the effluent leaving the first maturation pond is given by:

$$L_e = 0.75 \times 187.5 \approx 140.63 \text{ mg L}^{-1}$$

Since the BOD leaving the maturation pond is greater than the permissible discharge limit of 50 mg L^{-1} (The National Environment Regulations, S.I. No 5/1999 and World Bank, 2007), the effluent requires further treatment to reduce the BOD load. The BOD load can be reduced by planting aquatic plants that filter the wastewater to BOD levels that are acceptable for discharge. This may not require construction of a secondary maturation pond as long as the desired pathogen removal is achieved in the first maturation pond.

Mean temperature, T of the coldest month = 25^{0}C

Depth, D, of maturation pond is usually between 1and1.5m (Mara, 1989). Therefore $D = 1.25$ m

Effluent flow from facultative into the maturation pond, $Q_i = 254.11 \text{ m}^3 \text{ day}^{-1}$

According to FAO (2006), the mean monthly evaporation rates in Uganda are between 125 and 200 mm. Taking the average gives: (125+200)/2 = 162.5 mm month^{-1}.

Net rate of evaporation ≈ 162.5/30 = 5.42 mm day^{-1}. However, the net rate of evaporation varies from location to location depending on the prevailing weather conditions. Therefore 5.42 mm day^{-1} may not be applicable to all locations in Uganda.

The number of maturation ponds was determined by examining Equation (17) which contains two unknowns (θ_m and n), as θ_a and θ_f were already known.

$$\theta_m = \left\{ \left[\frac{N_i}{N_e(1 + K_B\theta_a) \times (1 + K_B\theta_f)} \right]^{\frac{1}{n}} - 1 \right\} \times \frac{1}{K_B} \qquad (17)$$

where, N_e and N_i are the number of coliform/100 ml in the effluent and influent; K_T is the first-order rate constant for coliform removal (per day); θ is a retention time (day).

The best approach to solving Equation (17) was 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 namely:

a) θ_m should not be greater than θ_f

b) θ_m should not be less than $\theta_{\min}$

Where $\theta_{\min}$ is the minimum acceptable retention time in the maturation pond.

The remaining pairs of θ_m and n, together with the pair and $\tilde{n}$, where $\tilde{n}$ is the first value of n for which θ_m is less than $\theta_{\min}$, were then compared, and the one with the least product selected, since this would identify the least land area requirements.

The value of the first order rate constant, K_T is highly temperature-dependent and can be found using Equation

(18).

$$K_T = 2.6(1.19)^{T-20} \tag{18}$$

$$K_T = 2.6(1.19)^{25-20} = 6.2 \text{ day}^{-1}$$

George et al. (2002) reported that dairy wastewater contains total coliforms in the order of 10^8-10^{10} coliform units per Litre. N_i was estimated as the average for the range:

$$\frac{10^8 + 10^{10}}{2} \approx 5.05\times10^9 \text{ coliform units per Litre}$$

N_e was taken as 400 MPN/100 mL (World Bank, 2007)

Retention time in the anaerobic pond, θ_{an1}=7 days

Retention time in facultative pond, θ_{fac}=30 days

Therefore, varying the number of maturation ponds, n in the Equation (17), yields various values of retention time, θ_m for the maturation pond.

For $n = 1$, retention time in the maturation pond, $\theta_m \approx$ 1809 days

For $n = 2$, retention time in the maturation pond, $\theta_m =$ 17 days

For $n = 3$, retention time in the maturation pond, $\theta_m =$ 3 days

Therefore, since $n = 1$ and $n = 2$ result in a retention time well above the recommended minimum retention time of three days (Marais, 1974), three maturation ponds would be incorporated in the treatment system to be able to treat the wastewater to the desired quality. According to Hamzeh and Ponce (1999), short-circuiting (when water enters and leaves the pond in a very short time) results in a large reduction in the discharge quality.

The mid-area of the maturation pond was determined from Equation (19):

$$A_m = \frac{2Q_i\theta_m}{(2D + 0.001\theta_m e)} \tag{19}$$

$$= \frac{2\times254.11\times3}{\left((2\times1.25)+\left(0.001\times3\times5.42\right)\right)} \approx 605.92 \text{ m}^2$$

Volume of the maturation pond = $A_m \times D$ (20)

= 605.92 × 1.25

≈ 757.40 m^3

Using the length to breadth ratio of 4:1 the dimensions at the mid-area were obtained by using Equation (6):

$A_m = L\times W$

$605.92 = 4W^2$ since $L = 4W$

Mid-width, W = 12.31 m

Mid-length, L = 49.23 m

The dimensions at the top and bottom levels of the pond were obtained using the pond geometry shown in Figure 1.

Top surface dimensions:

From Equation (7), top length,

$L = L+n(D+2F)$

= 49.23+2(1.25+2×0.5)

= 53.73 m

From Equation (8), top width,

$W = W+n(D+2F)$

= 12.31+2(1.25+2×0.5)

= 16.81 m

From Equation (6), top surface area of maturation pond = $L\times W$

= 53.73 ×16.81

= 903.20 m^2

Bottom surface dimensions:

Using Equation (9), bottom length,

$L = L-nD$

= [53.73− (2×1.25)]

≈ 51.23 m

From Equation (10), bottom width,

$W = W-nD$

= 16.81−2(2×1.25)

≈ 14.31 m

From Equation (6), bottom surface area of maturation pond = $L\times W$

= 51.23×14.31

= 733.10 m^2

Total land area required = (903.20 ×3) = 2709.6 m^2

Effluent flow rate discharged from the maturation pond,

$$Q_e = Q_i - 0.001A_m e \tag{21}$$

$$Q_e = 254.11 - \left(0.001\times605.92\times5.42\right) \approx 250.83 \text{ m}^3 \text{ day}^{-1}$$

Mean flow rate, $Q_m = \dfrac{254.11+250.83}{2} \approx 252.47$ m^3 day^{-1}

Land requirement per flow rate = $\dfrac{2709.6}{\text{mean flow rate, } Q_m}$

$= \frac{2709.6}{252.47} \approx 10.73 \text{ m}^2 \text{ m}^{-3}$ flow rate

Total area required to construct the ponds, A_{tot} = sum of top surface areas of anaerobic, facultative and maturation ponds.

$A_{tot} = 945.19 + 6361.54 + (903.20 \times 3) \approx 10016.33 \text{ m}^2$ or 1.002 hectares

However, to take into account the overall land area required for pond operation and maintenance, the total area calculated was multiplied by a factor of 1.25-1.3 (i.e. additional 25% to 30% land). A factor of 1.25 is suitable for large systems while a factor of 1.3 is more suitable for small systems (Mara et al., 1998). Therefore, total land requirement for pond construction, operation and maintenance:

$= 1.25 (10016.33) = 12520.41 \text{ m}^2$

= 1.25 hectares

2.4 Pond maintenance

The maintenance of the WSPs requires withdrawal of sludge and the control of odours through the recirculation process of pond effluent from final ponds. Alexiou and Mara (2003) found that the volume of the sludge needs to be disposed every two to three years. Once the WSP operated, it is necessary to carry out the maintenance work. The maintenance of the waste stabilization ponds are simple and easy to manage. According to Mara and Pearson (1998), the preliminary treatment involves removal of screening and grit retained in the inlet work. The mosquito breeding habitats can be prevented by cutting, pruning, and removing the vegetation that grows in the pond. On the other hand, floating scum should be removed from facultative and maturation ponds to maximize photosynthesis and surface re-aeration.

3 Results and discussion

The total land area required for anaerobic pond was obtained as 945.19 m^2. The land requirement by flow rate was obtained as 3.92 m^2 m^{-3} flow rate. This is in close agreement with Sasse (1998); the author reports that anaerobic ponds require approximately 4 m^2 m^{-3} daily wastewater flow.

The total land area required for facultative pond was obtained as 6,361.54 m^2. The land requirement by flow rate was obtained as 22.13 m^2 m^{-3} flow rate. This is in close agreement with Sasse (1998); the author reports that anaerobic ponds require approximately 25 m^2 m^{-3} daily wastewater flow. The variation in the rates may be attributed to variation in climatic conditions. On the other hand, the total land area required for the maturation pond was 2,709.06 m^2. The land requirement by flow rate was obtained as 10.73 m^2 m^{-3} flow rate.

Table 3 shows a summary of the dimensions for the anaerobic, facultative and maturation ponds obtained in the design process.

Table 3 Summary of pond dimensions

Parameters	Anaerobic	Facultative	Maturation
Volume (m^3) at D	2053.57	8124.15	757.40
Number of Ponds	1	1	3
Area of water surface (m^2) at D	483.19	5416.1	605.92
Length: Width ratio 1:X	0.33	0.25	0.25
Dimensions			
Freeboard, F (m)	0.5	0.5	0.5
Slope of Embankment, n	1.5	2	2
Depth of Water Body, D (m)	4.25	1.5	1.25
Mid- depth length, L(m) at D	38.07	147.19	49.23
Mid-depth width, W (m) at D	12.69	36.80	12.31
Top length (m) at F	45.95	152.19	53.73
Top width (m) at F	20.57	41.80	16.81
Surface area at top (m^2) at F	945.19	6361.54	903.20
Base length (m) at $D=0$	39.58	139.19	51.23
Base width (m) at $D=0$	14.20	38.80	14.31
Surface area at base at $D=0$	561.84	5400.57	733.10

Besides treatment of wastewater to reduce BOD, remove pathogens and other pollutants, there are other several benefits associated with the operation of waste stabilization ponds. Other than discharging the effluent to the environment, it can be recycled for use in agriculture and aquaculture. According to WHO (2006), WSP are effective in removing nematodes (worms) and helminth eggs while preserving some nutrients. The use of wastewater in agriculture is a possible strategy for addressing water scarcity and nutrient deficiency in agricultural systems in the face of climate change (Kanyoka, 2011). On the other hand, the ponds can be combined with aquaculture to locally produce animal feed (e.g. duckweed) or fish (e.g. fishponds) (Varon, 2004). Biogas may also be recovered for use when anaerobic ponds are covered with a floating plastic membrane (Varon, 2004). The recycling of wastewater for agriculture and aquaculture may result into high economic benefits (Kanyoka, 2011) that can offset the operation and maintenance costs of the ponds. As a result, the payback period of the investment cost may also be shortened. However, there are also negative aspects related to wastewater reuse which include soil salinity, health of farmers and consumers, public acceptability, marketability of produce, economic feasibility and sustainability of wastewater irrigation (IWMI, 2006; WHO, 2006).

A cost benefit analysis needs to be done to ascertain the feasibility of the technology which is partly influenced by availability of land and its price among others. According to IRC (2004), WSPs are especially appropriate for rural communities that have large, open and unused lands, away from homes and public spaces where it is feasible to develop a local collection system. In comparison with other wastewater treatment technologies, waste stabilization ponds have the advantage of very low operating costs since they use no energy and also use low-tech infrastructure (IRC, 2004). This makes them particularly suitable for developing countries where the conventional wastewater treatment plants may fail because water and sewer utilities may not generate sufficient revenue to pay the electricity bills for the plant (IRC, 2004). However, pond systems are not recommended for use if appropriate mosquito control measures are not guaranteed (WHO, 2005; Morel & Diener, 2006).

4 Conclusion

When the design of the ponds is done carefully by competent design engineers, the system is expected to function well with high efficiency. In addition, the use of waste stabilization ponds for treatment of dairy wastewater can result in significant economic and environmental benefits if recycled for agriculture and aquaculture.

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