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"Design Criteria of the oxidation ponds in Egypt and world wide"

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ABSTRACT

"Design Criteria of the oxidation ponds in Egypt and world wide"

Oxidation ponds, known as waste stabilization ponds, have become one of the world's most used methods of treating waste water in areas where there is large space for their construction. In addition, they are one of the most economical and environmentally friendly ways of treating wastewater and producing a highly purified effluent. They create a natural environment and utilize natural processes to treat wastewater contaminants and can include other systems such as constructed wetlands, septic tanks, lagoons and others.

In this research present how to access to the best ways to design different oxidation ponds that are consistent with the Egyptian conditions of temperature, wind direction, the amount of treated water and study the nature of groundwater, soil characteristics and components, and the sun shining, the characteristics of the wastewater ponds, suitable form of ponds and method of operation optimization, construction and operating costs and the price of land, the fields of use of treated water and taking into account the characteristics of wastewater in Egypt, and which can be known by COD, BOD, TSS of the water where the properties of wastewater in Egypt is different from the rest of the world and present the principal considerations are taken during the design.

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Chapter 1

Introduction

Chapter 1 – Introduction

1-1 Background

Impurities, mainly suspended and dissolved solids are resulted from typical domestic sewage. There are also gases, microorganisms and other materials exist. To reduce the organic pollutants which involve the wastewater treatment by the separation of the solid fraction from the liquid phase (treating the solids) removing pathogens without bad effects into the environment by different methods of treatment involve nature, chemical and biological treatment. In this research it will be used nature treatment by oxidation ponds because it is economical method and can be achieved all the processing operations and the treatment in the same pond or can use a set of similar ponds in all types of the oxidation ponds. (1)

Eutrophication can be caused at high quantities of organic pollutants (a significant environmental problem) in natural water systems. Wastewater treatment ponds are designed to oxidize wastewater before it is released into a natural water body or recycled. (3)

The oxidation ponds involve biological and chemical processes where the effluent’s organic pollutants (the organic load) are broken down primarily through bacteria’s of the organic matter as a source of its food. Treatment ponds are capable of removing heavy metals, harmful pathogenic organisms and nutrients like nitrogen and phosphorous. (1)

If there is small amount of pollutants in the wastewater, so oxidation ponds can be used individually. Whereas in other cases the ponds are used in conjunction with other ponds types or treatment process, if there is big amount of pollutants in the wastewater. It is depending on the nature of the use of treated effluent from the treatment. Climate (particularly temperature) in the region of the treatment ponds represent the key in the possible treatment and in design the oxidation ponds. (3)

Control the environmental factors in the wastewater help many of the microorganisms in using organic materials as a source of food for the reproduction and supply of energy. The rate of activity of these organisms have a relation with the component of sewage and the rates of oxygen consumption (or in other words) rates of change in the value of Biochemical oxygen demand (BOD) with time .Because of the presence of algal cells and not to absorb sudden changes in hydraulic and organic loads and temperature variation with depth change are not taken into account in current design methods which depending on the surface loading rate and volumetric loading rate and is reflected in the difficulty of reaching the limits of concentrations of suspended solids required. (1)

1-2 Advantages of Oxidation ponds

Wastewater treatment process is simple, and it represents the simplest process to stabilize the biodegradable matter contained in wastewater by creating conditions favorable for the natural processes of purification. The forces of nature are allowed to act upon the wastewater (such as sunshine, wind, temperature, spontaneous plant and animal life).The Designer has a limited role in design the oxidation ponds and even less than the operator can do to steer or modify the conditions in the pond if they are not as predicted. In additional to the previous advantages of oxidation ponds it doesn't need much more maintenance and supervision. (1)

1-3 Disadvantages of Oxidation ponds

Ponds need a large area of land, and the difficulty of reaching the limits of concentrations of suspended solids required, when the percentage of flow is high amounts. So the efficiency of the pond is reduced. (1)

1-4 Principals are taken during the design process

- Adequate protection for the public health (removal of pathogens).
- Level of operator skills available.
- Minimization of operating costs (energy, spare parts, maintenance).
- Taking advantages of local resources (labor, equipment materials).
- Capital cost. (1)

1-5 Definitions

1-5-1 Wastewater Oxidation pond

It is a treatment unit which wastewater is allowed to stand for a time, under the influence of the forces of nature and microorganisms so that it is converted into an effluent that meets the quality standards established for final disposal or reuse and design the oxidation ponds is different according to type. (1)

1-5-2 Types of Oxidation ponds

1-5-2-1 Anaerobic pond

It is depending on anaerobic bacteria and break down the complex organic matter into the initial components. Anaerobic pond is devoid of dissolved oxygen. (1)

1-5-2-2 Facultative pond

It consists of aerobic layer containing dissolved oxygen which algae and both facultative and aerobic bacteria coexist are found and anaerobic layer devoid of oxygen and an middle layer is also between them. (1)

1-5-2-3 Maturation pond

It is an aerobic pond secondary or tertiary wastewater stabilization pond and it removes from a treated effluent "the pathogenic agents". It also removes some suspended matter, and it reduces the concentration of biodegradable organic matter. (1)

1-5-2-4 Facultative Aerated Lagoons

It is more compact than facultative ponds or anaerobic facultative ponds. It used to have a predominantly aerobic system, and when land space is limited so is therefore more appropriate for wastewater treatment than the use of facultative ponds. The main difference with relation to the conventional facultative pond regards the form of oxygen supply. While in facultative ponds the oxygen is obtained from algal photosynthesis, in the case of facultative aerated lagoons the oxygen is supplied by aerators. (3)

1-5-2-5 Complete - Mix Aerated Lagoons - Sedimentation Ponds

Complete - Mix aerated used for treatment of water with aerobic mechanisms. The aerators serve to introduce high levels of oxygen into the system and allowing for the suspension of all solids in the liquid column. (3)

1-5-2-6 Partial mix aerated lagoons

Partial mix lagoons are commonly used in United States for at least 40 years to treat municipal and industrial wastewater by using the aeration that provided by either mechanical surface aerators or submerged diffused aeration systems.(3)

1-5-3 Biochemical oxygen demand (BOD)

It is the total amount of oxygen required for the biological breakdown of carbon-containing and oxidizable-nitrogen containing material. It is an important factor to indicate the efficiency of the pond because it defines the amount of oxygen needed by this pond. (1)

1-5-4 Biochemical Oxygen Demand for 5- days (BOD_5)

It represents the amount of oxygen need to keep biological activity a period of 5 days at 20^0C. (1)

1-6 Pond location and orientation

Allowance must be made for sewerage expansion plans and urban master plans. Oxidation ponds should be located so that the direction of the prevailing wind is away from the nearest community. If possible, the ponds should be oriented so that their longest dimension is parallel to the wind direction. And eliminate the need for pumping by design the discharge water level in the first stabilization pond below the invert level of the final manhole in the incoming sewer. Anaerobic ponds should be located at least 1000 meters from the nearest dwelling, while facultative ponds should preferably be at least 500 meters away. (1)

1-7 Configurations of the oxidation ponds

The common arrangements are often used in series of the ponds are anaerobic, facultative and maturation ponds. Anaerobic ponds are frequently used ahead of facultative ponds in order to reduce the land area required. To have a high-grade effluent is necessary(especially with regard to pathogenic organisms) it will be used Facultative ponds followed by one or more maturation ponds .This is the case when effluent is reused for agricultural purposes and for aquaculture.
Some typical pond configurations in parallel and in series are shown in Fig 1.

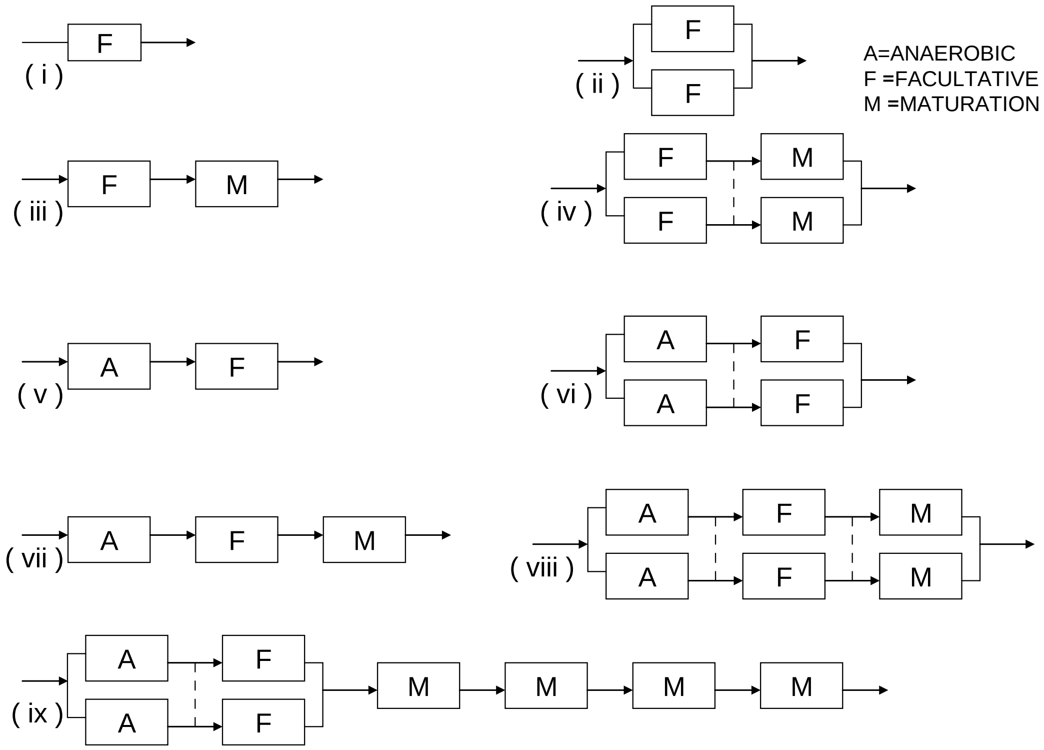


Figure 1: A selection of possible pond arrangements in parallel and series .

1-8 Pond Geometry

1-8-1 Pond shapes

1-8-1-1 Square

A convenient shape for the oxidation ponds is the square. (1)

1-8-1-2 Rectangular

In rectangular shaped ponds the ratio of length to width should not be greater than 2:1. Multiple inlets should be used if a greater ratio is unavoidable. Some authors favor very long rectangular ponds ratio length to width 3:1 or more to foster wind action. (1)

1-8-1-3 Irregular

Sometimes the available area of level or only slightly sloping land is limited, and thus square or rectangular ponds may be uneconomical because of loss of useful pond area at the edges. In such cases irregular-shaped ponds which fit into the available land may be used, but no peninsular or re-entrant angles or curves should exist. (1)

1-8-2 Pond Corners

To prevent effective water movement and to avoid dead spaces so vertices should be rounded off and smoothed out. At the intersection of embankment slopes should be provided without sharp corners. (1)

1-9 Appurtenances

1-9-1 Pipes

To avoid cutting and filling newly constructed work, with the risk of creating weak points so pipes going through the embankment and it should be installed prior to construction. (1)

1-9-2 Inlets

In square-shaped ponds, the inlet usually ends at the center. In rectangular ponds the inlet is frequently located as shown in Fig.2, which prevents raw sewage from reaching the edges of the pond. (1)

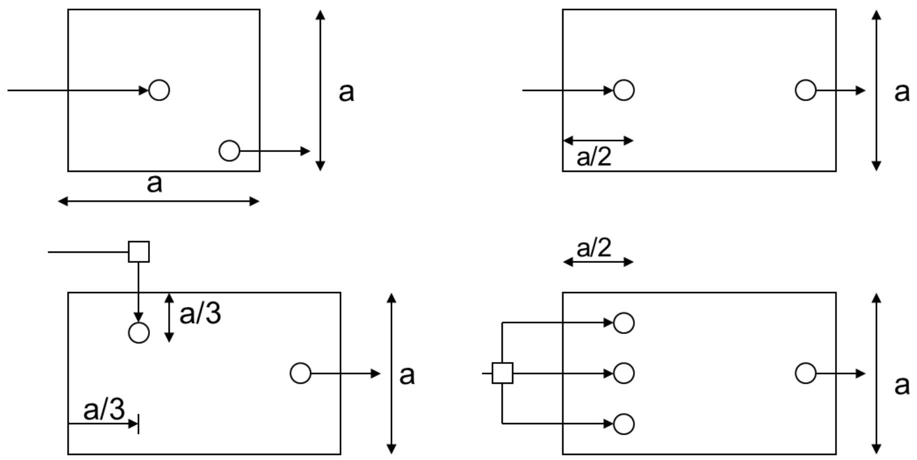


Figure 2: Typical placement of inlets in square and rectangular ponds .

1-9-3 Outlets

There are many types of outlet, but many of them are attached to a discharge pipe at bottom level going through the embankment. The outlet of a pond may be placed at any point on its edge, but is normally located at the base (toe) of the embankment at the opposite end to the inlet. (1)

1-9-4 Flow measurement

In each pond should be installed at least two flowmeters, one on the inlet and the other on the outlet of the pond. The inlet flowmeter may best be installed on the top of the embankment, just above the upper end of the inlet pipe. And the outlet flowmeter may be the outlet control device itself if it is designed as a rectangular weir. Otherwise, a suitable flowmeter can be installed on the discharge pipe where it emerges from the outer side of the embankment. The magnitude of evaporation (seepage or infiltration) as well as of the diluting effect of rainfall an indication of inlet and outlet flows to evaluate the pond performance. (1)

1-9-5 Interconnecting pipes between the oxidation ponds

Interconnecting pipes are used to convey the effluent from one pond to another where two or more units are operated in series. In many cases, a pipe laid through the embankment below water level will serve as a suitable interconnection, and the difference in water levels between the ponds will (at least) equal the head loss in the interconnecting pipe. If it is intended that the two ponds should have specific water levels, the outlet from the first pond must be fitted with a device which assures the required level in that pond (e.g. Fig. 3), and so on.

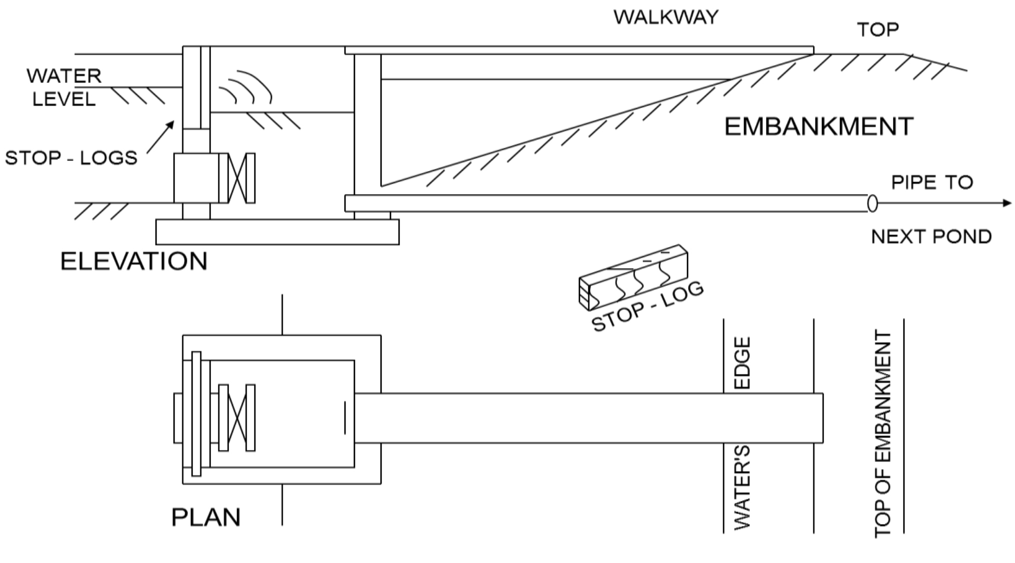


Figure 3: Variable level outlet that serves as a weir

No special protection is needed to keep floating material on the surface of the first pond from entering the second, if the interconnecting pipe is under water at ends (say 30 cm or more). Some designers add at the inlet end an elbow-shaped bend, in order to be drawn from a deeper layer and to be used as a variable level draw-off to transfer the contents of the first pond from the most advantageous layer to the second.

Frequently, the inlet pipe to the second pond is extended down along the slope until it reaches the toe of the embankment. A flowmeter is installed sometimes on the transfer pipe. Its best location is on the inlet side, placed in a box penetrating slightly into the embankment. Interconnecting pipes between successive ponds (for example an anaerobic and a facultative pond or a facultative and a maturation pond) must always be protected against floating materials entering to the interconnecting pipe. Interconnecting pipes should enable individual ponds to be isolated. To accomplish this while continuing to operate the pond installation requires a by-pass system for each isolated pond. (1)

1-9-6 Revising the design of the incoming sewer to the oxidation ponds

The balance between the fill against the excavation allowing for compaction is the most economical design. If the elevation of the pond bottom so designed is at a level below that of the influent sewer (though the surface of the pond is above) it is possible that the incoming sewer could be relaid rather than resorting to pumping. This situation often arises when the sewer has been designed by a different designer, responsible only for the collection system. In this case, it might be reduce the slope of a length of the sewer so that it arrives at the pond embankment at an elevation above the pond surface level. If not, the cost of deeping the excavation for the pond must be compared with the capital and operating costs of a pumping unit. (1)

1-10 The required data for designing the oxidation ponds

When designing the oxidation ponds put in consideration the following:

1- Period of design, target date for start of operation;
2- Present and future populations to be served (estimates of population growth);
3- Industrial wastewaters (population equivalents, acceptability of industrial wastes, recommended pre-treatment);
4- Analysis of sewage;
5- Flows for design;
5- Loadings for design;
7- Treatment requirements;
8- Insulation conditions;
9- Temperature conditions;
10- Direction of prevailing wind(s);
11- Groundwater infiltration into the sewer system;
12- Storm water intrusion;
13- Suitability of pond treatment, combinations of ponds;
14- Possible sites (distance from nearest dwellings, communities etc.) with selection of the best alternative;
15- Shape and orientation of the ponds, positions of inlets and outlets;
15- Area requirements, net and gross;
17- Sludge accumulation and removal. (1)

1-11 The objective of this research

The research present how to access to the best ways to design oxidation ponds that are consistent with the Egyptian conditions of temperature, wind direction, the amount of treated water and study the nature of groundwater, soil characteristics and components, the sun shining, the characteristics of the wastewater ponds, suitable form of ponds and method of operation optimization, construction and operating costs and the price of land, the fields of use of treated water and taking into account the characteristics of wastewater in Egypt, and which can be known by the COD, BOD, TSS of the water where the properties of waste water in Egypt is different from the rest of the world.

Chapter 2
Literature review

Chapter 2- Literature review

2-1 Anaerobic ponds

2-1-1 Design Criteria

"Anaerobic ponds are similar to septic tanks and unheated anaerobic digesters". (1)
"Typically an anaerobic treatment pond will be used as the first biological treatment step and followed by further treatment, such as a facultative pond". (See figure 4). (3)

It is used to remove BOD_5 from the waste water and it is various from 50 - 70%. It is depending on anaerobic bacteria and break down the complex organic matter into the initial components. (1)

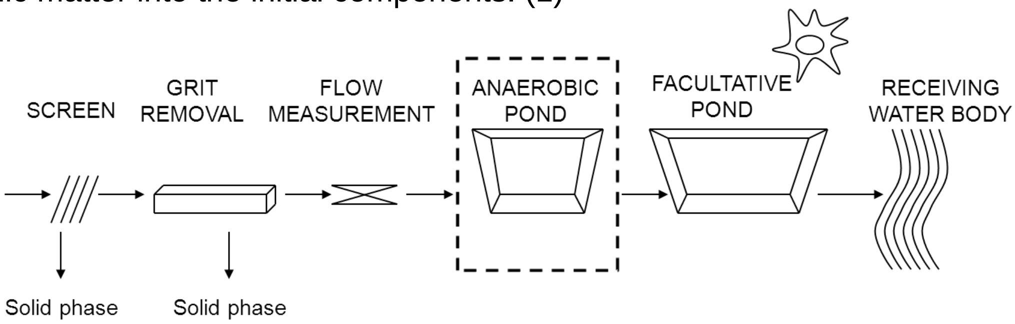


Figure 4: Typical Anaerobic Treatment Pond in series with a Facultative Pond

One of the most important factors influencing BOD removal efficiency is Temperature, so retention time requirements are linked with ambient temperature. (1)

In Anaerobic ponds the oxygen production rates less than the oxygen consumption rate ,which creates the anaerobic conditions .Particular bacteria have evolved to thrive in oxygen depleted conditions, as they break down organic material ultimately into methane and carbon dioxide gas .(3)
There are three criteria as a basis in design procedures:
Surface loading rate, volumetric loading rate and hydraulic retention time. (1)

2-1-1-1 Surface loading rate

It is defined in terms of kg BOD_5 / ha. d. (1 ha = 10000 m^2). It is useful to assess land requirements and to check whether there is a risk of the pond becoming facultative at some time in the year.
It is incorrect to refer to an anaerobic pond in terms of surface loading.
The surface area of anaerobic pond does not influence its performance, whereas the volume does. This is a logical rationalization unless the sludge/liquid interface (if it exists) is the site of significant anaerobic activity in the liquid phase. Sometimes the deep of the anaerobic ponds is five meters, keeping the surface area as small as possible in order to lose less heat and absorb less oxygen from the atmosphere. (1)

2-1-1-2 Volumetric loading rate

"Volumetric loading is expressed in terms of grams BOD_5 per cubic meter per day (gm $BOD_5/m^3.d$)". (1)

$V = L / L_v$ (2.1)

Where:

V = Volume required for the pond (m^3)

L = total (soluble + particulate) influent BOD load (Kg BOD_5 / day)

L_v = volumetric organic loading rate which is the amount of BOD that the pond can be treat per volume (Kg BOD_5 /m^3. day)

Volumetric loading rate is influence on the performance of the pond. Anaerobic sludge digesters and anaerobic dispersed-growth processes examples similar to the volumetric loading rate. (1)

2-1-1-3 Hydraulic retention time

The most commonly used parameter in anaerobic pond design is retention time. (2)

It is range varies from author to author. Gloyna reports retention times as low as 18 hours and recommends a maximum of 5 days in tropical areas so as not to turn ponds to facultative ponds.(1)

Eckenfelder makes reference to retention times between 5 and 15 days, but the high values should be attributed to cold climates and there isn't anaerobic activity under 10°C). (2)

2-1-2 Critique of design criteria

Some species and strains are believed to metabolize organic matter more slowly than others (predators may be present) and mixing in the anaerobic ponds resulted from the rising gas/ sludge mixture is never complete (kinetics are probably not of the first order) the complex biota are of different composition and efficiency from place to place. Also the difference temperature with difference the depth in present design approaches. None of these significant factors are taken in the design.

The influent flow and the retention time not to be taken in the design criteria that based on areal or volumetric organic loading .The design criteria based on hydraulic retention time imply that not only the BOD of the solids, but also some soluble BOD is removed. If this were not, so reductions in influent sewage BOD_5 of the order of 60% or more could not be achieved.

When measured the anaerobic biodegradation, it generated carbon dioxide and methane per unit time, so an increase in temperature of 5°C speeds up four-fold. From anaerobic ponds the Scum, mats and other floating material should not be removed, even if they become unsightly, because this floating cover keeps the pond contents warmer and isolates the surface from air contact, thus minimizing objectionable odours evolving from the surface.(1)

2-2 Facultative ponds

2-2-1 Design Criteria

A facultative pond efficiency of about 80%, if the preceding ponds to the facultative ponds will be anaerobic and succeeding ones will be maturation.(1)
The algal cells is very important in facultative ponds, which supplies the upper layers in the water on the pond of oxygen using sunshine is termed as Photosynthesis

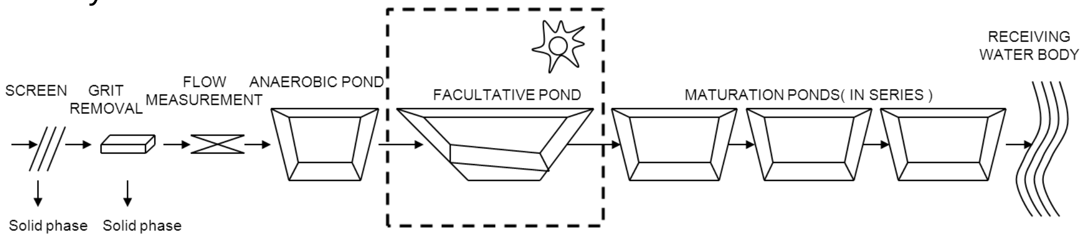


Figure 5: Facultative Pond

2-2-2 Surface loading rate method

There is an empirical method for the design; it is the most frequently used and it is the adoption of a figure for pond surface (or areal) organic loading based on local experience. Surface loading is a useful and appropriate parameter for application to design facultative ponds.
But in the absence of local data, the problems appear in decide what this loading should be. The San Juan experience suggests that may result in occasional slight odour problems; at loadings between 200 and 400 kg BOD_5/ha·d and there would be frequent light odours at values between 400 and 700 kg BOD5/ha.d. And light or strong odours would always be present at higher loadings. During the survey, pond temperature varied between 19°C and 27.5°C. There was rainfall and it ranged between 1.0 and 1.9 mm per month. (1)
For warm climates, a suitable areal loading lies between 200 and 400 kg BOD_5/ha.d . At pond temperatures 30°C favour loadings around 300 to 400 kg BOD_5/ha·d, whereas at lower temperatures (in the range 20°C - 25°C) lower loadings would be needed from 200 to 250 kg BOD_5/ha.d. To establish appropriate surface loadings for facultative ponds, sometimes pilot plant studies are sometimes carried out. But studies might give misleading results unless the pilot-scale ponds are large enough to allow the full effects of wind-mixing to be established. (1)

2-2-3 Empirical models

2-2-3-1 Arceivala equation (Indian method)

The following relationship between admissible surface load Ls,o and local latitude (lat) between 8°N and 36°N suggested by Arceivala:
" $L_{s,o}$(kg BOD_5 / ha.d) = 375 - 6.25 (lat) "   (2.2)

Thus at the southern areas the extreme values of Ls,o work out to be 150 and 325 kg BOD_5/ha.d, and less at in the northern areas , respectively. This approach depends on the local climatic conditions such as sunlight and temperature that vary with latitude. (1; 2)

2-2-3-2 McGarry and Pescod regression equation

McGarry and Pescod found that hydraulic retention time, influent BOD_5 concentration and pond depth had little influence on percentage or areal BOD_5 removal under the normal operating range of loadings for facultative ponds. But the temperatures were not evenly distributed over the range of loadings. So the effect of temperature on percentage BOD_5 removal appeared to be minimal. McGarry and Pescod performed regression analyses of performance data on a wide range and found a relation between maximum applied areal or surface loading $L_{s,o}$ and the minimum ambient mean monthly air temperature, T_a (^{0}C). The relationship is:

" $L_{s,o}$ (Kg BOD_5 / ha. d) = $60.3 * 1.0993^{Ta}$ " (2.3)

The loadings below 112 kg BOD_5/ha.d can not apply the McGarry and Pescod formula. About 80% for an influent wastewater BOD_5 concentration removed at application of the McGarry and Pescod equation for the amount of BOD_5 concentration is 500mg/L for an influent .Unfiltered effluent BOD_5 can be expected to be between 50 and 80 mg/L reflecting the algal content. (1)

In the case of increasing the loadings derived from Eq. (2.3), a facultative pond can be expected to operate completely anaerobically at certain periods.
A linear approximation of this equation suggested by Mara:
" $L_{s.0}$ (Kg BOD_5 / ha. d.) = $20\ T_a - 120$ " (2.4)

Arthur presented an adjustment of the linear approximation of McGarry and Pescod's equation, so Eq. (2.4) became unnecessarily for use in design.
So Arthur suggested using the following form for design:
$L_{s.0}$ (Kg BOD_5 / ha. d.) = $20\ T_a - 60$ (2.5)

This is an appropriate modification which suggested by Mara's data in north-eastern Brazil.

2-2-3-3 Gloyna equation

To achieve a predetermined BOD removal and to prevent aesthetic nuisances, Gloyna's approach to oxidation pond design to provide factor of safety in the model and form of the equation is:

" $V = 3.5 * 10^{-5}\ Q\ L_u\ \Theta^{(35-T)}\ f\ f`$ " (2.6)

Where:

V = pond volume (m^3);
Q = wastewater flow (L/d);
L_u = ultimate influent BOD (or COD) (mg/L)
Θ = temperature reaction coefficient (assumed to be 1.085 for facultative

ponds treating sewage comprising domestic and industrial wastewater);
T = pond water temperature (^{0}C);
f = algal inhibition factor (f = 1 for sewage and many industrial wastewaters);
f ' = sulphide or other immediate chemical oxygen demand (f ' = 1 for So_4 equivalent ion concentration of less than 500 mg/L).(1)

2-3 Maturation ponds

2-3-1 Design Criteria

Maturation ponds are primarily designed for tertiary treatment. Maturation ponds are generally used in other types of the oxidation ponds such as anaerobic and facultative ponds. They are also known as polishing ponds because they are the last step in the process of treatment. Maturation ponds are very important to ensure the removal of pathogens, excess nutrients and algae. (3)

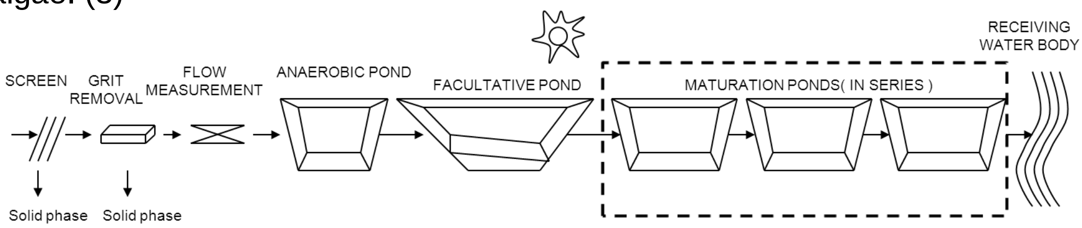


Figure 6: Anaerobic Pond - Facultative Pond - Maturation Ponds

Maturation ponds used to removal of pathogenic agents, such as some species of bacteria(fungi) protozoa and viruses. Maturation ponds are often considered indispensable, when agricultural reuse of the effluent is planned.
It constructed when land cost is low and sufficient area is available. (1)
To get a specific output of concentration pathogens usually determined Size and number of maturation ponds required in the system by retention time required. (3)
Retention time is the main parameter to be considered in bacterial die-off in ponds. (1; 2)
Experimental work and practical experience have shown that a minimum retention period of 5 days in a single maturation pond or 3 days per pond in a series of two or more ponds following a facultative pond is usually adequate.
Retention times can extend to 10 days or more. Maturation ponds are usually 1.0 to 1.5 meters in depth. (1)
Marais pointed out, the disappearance of fecal bacteria (Escherichia coli, streptococcus faecalis and others) in oxidation pond may be estimated using the following equation:

$$(N_R / N_0) = 1/(K` R + 1) \qquad (2.7)$$

Where:
N_0 = bacterial population in the influent;
N_R = bacterial population after R days;
K' = die-off constant (d^{-1}). This varies from micro-organism to micro-organism and between different strains of the same one;
R = retention time (d). (2)

Chapter 3
Research Methodology

Chapter 3 - Research Methodology

3-1 Research Methodology

1) Showing different designs for the oxidation ponds in different areas and projects for oxidation ponds plants carried out by methods of a variety of design.
2) Analysis of the design results and their compatibility with circumstances Egyptian conditions.
3) Review of codes for design the oxidation ponds and illustrate the reasons for selecting specific design methods for the oxidation ponds.
4) Highlight the advantages of choosing method of design for another approach and appear the extent efficiency of oxidation ponds in wastewater treatment.
5) Proposal for work of the design for the oxidation ponds under Egyptian conditions.

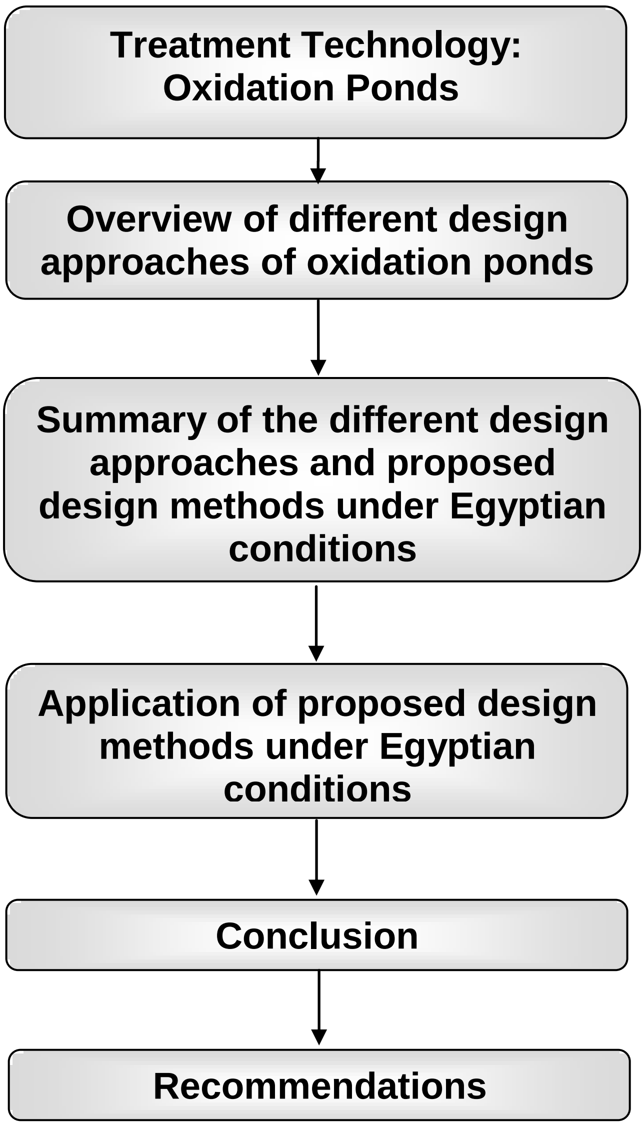


Figure 7: Research Methodology

3-2 The procedures for the design

The procedures for the design of the wastewater treatment ponds are:

a - Estimate the influent flow;
b - Calculate BOD concentration of the waste water;
c - Survey the mean temperature situations;
d - Local environment investigation for meteorological and hydrological characteristics;
e - Explore the soil conditions and the locations of waterways and ground water;
f - Determine the combination of ponds and the number of series;
g - Anaerobic ponds design;
h - Facultative ponds design;
j - Aerobic ponds/Maturation ponds design. (25)

Chapter 4
Overview of different design approaches of oxidation ponds

Chapter 4
Overview of different design approaches of oxidation ponds

4-1 Design of the oxidation ponds in Egypt

4-1-1 Anaerobic ponds

Anaerobic ponds constructed prior to the facultative ponds. It does not depend on algae (see figure 4-chapter 2).

4-1-1-1 Design criteria

The water depth varies between 2.5 - 5 m at temperature higher than 20ºC.(2) The organic load BOD_5 is ranging from 0.125 kg per cubic meter per day to 0.3 kg per cubic meter per day. Appear bad odors if increasing organic load BOD_5 to 0.4 kg per cubic meter per day.Collect sediments rate ranging from 0.03 cubic meters per person per year to 0.04 cubic meters per person per year.When the depth of the deposits about half the depth of the pond, it must be unload the anaerobic pond of sediments. The length to the breadth of pond ratio must to be range of 2:1 to 3:1. Volumetric loading rate influence on the performance of the anaerobic pond and for design use the equation (2.1).

$V = L / L v$

Where:

V = Volume required for the pond (m^3)

L = total (soluble + particulate) influent BOD load (Kg BOD_5 / day)

L v = volumetric organic loading rate (Kg BOD5 /m^3. day)

Retention time in the anaerobic ponds is 3-5 days. And taken 3 days and the retention time at 1 day, the rate of removal BOD_5 is 50%;and at retention time 2.5 day, the rate of removal BOD_5 is 60% and at retention time 5 days, the rate of removal BOD_5 is 70%,And these percentages of removal taken at average temperatures of the water in the pond. (2)

4-1-1-2 Advantages and disadvantages of the anaerobic ponds

Advantages

The rate of removal BOD_5 is various from 50 -70 %.

Disadvantages

Appear the bad odour and the flies in the surface area because of a floated layer on the surface of the pond.

4-1-2 Facultative ponds

The algal cells is very important in facultative ponds, which supplies the upper layers in the water on the pond of oxygen using sunshine is termed as Photosynthesis (see figure 5 -chapter 2). And organic materials are oxidized by bacteria to inorganic oxidized materials and algal cells appear in waste water treatment.

4-1-2-1 Design criteria

The organic load is ranged from 200 kg/ha.day – 300kg/ha.day. In facultative ponds, the length to breadth ranges from 2:1 to 3:1.The depth of the water ranges from 1.5 m to 2.00 m and use the equations (4-1) to (4-6) for design. (2)

4-1-2-2 The different methods for design the facultative ponds

"$\text{Area} = \dfrac{Q * t}{D}$" (4.1)

Where:
Q = flow in m^3/day
t = Retention time in days
D = Depth of water in Pond (m)

4-1-2-2-1 Asian Institute of Technology Method

"$L_s = 8 * 1.054^{T}$" (4.2)

Where: L_s = permissible load of BOD_5 Kg / hectare/day
T = temperature in (^{0}F).(2)

"$T = 10 * D \dfrac{L_i}{L_s}$" (4.3)

Where: t = detention time in days.
Li = influent BOD mg / liter.
Ls = allowed BOD load Kg / hectare.
D = depth of water in pond in meters.(2)

Another Modified Formula:

"$T = \dfrac{Li - 60}{18 * 1.05^{T-20}}$" (4.4)

Where: Li = Influent B.O.D. mg / liter.
T = temperature in (^{0}C).
60 = 60 mg/liter, the allowed BOD in effluent. (2)

4-1-2-2-2 Design Based on Mean Temperature of Coldest Month

"$A = \dfrac{Q\,(Li - Le)}{18D\,(1.05)^{T-20}}$" (4.5)

Where: A = Area in m^2.
Q = flow m^3 per day.
Li = influent BOD_5 mg/liter.
Le = effluent BOD_5 mg/liter.
T = Mean temperature of coldest month in (^0C).
D = depth of water in pond in meters.(2)

Empirical Method:

$$"A = \frac{Li * Q}{2T - 12}" \quad (4.6)$$

Where: A = Area m^2.
Li = influent BOD_5 mg/liter.
Q = Daily flow m^3 per day.
T = Temperature in (^0F) (average of coldest month).(2)

4-1-2-3 Advantages and disadvantages of the facultative ponds

Advantages

The sunlight reaches to the layers in the pond, so the algae are growth and the algae are considered the source of the dissolved oxygen in this pond. facultative ponds don't need to the electrical energy so this type is economical.

Disadvantages

Facultative ponds need large areas.

4-1-3 Maturation ponds

Maturation ponds are used to improve the properties of the waste water from the bacteriological and chemical side (see figure 6-chapter 2).

4-1-3-1 Design criteria

The depth of water is 1-1.5 m for maturation ponds .Retention time various from 3 and 10 days and the retention time not less for 5 days in the case of one pond. (2)

4-1-3-2 Bacterial reduction models

Determine the required retention time and number of ponds in series at bacterial reduction models without final disinfection using ponds.
The disappearance of fecal bacteria estimated by using the following equation:
$(N_R / N_0) = 1 / (K` R + 1)$ (4.7)
Where: N_0 = bacterial population in the influent;

N_R = bacterial population after R days;
$K^`$ = die-off constant (d^{-1}). This varies from micro-organism to other and between different strains of the same type;
R = retention time (d). (2)

Another formula from the equation is:

$(N_1/N_E) = 1/[(K^` R + 1)]^n$ (4.8)

Where: N_1 = bacterial population in the influent;
N_E = bacterial population after R days;
$K^`$ = die-off constant (d^{-1}).
R = retention time (d).
n = number of ponds in series. (2)

N_1 assumed the fecal coliforms numbers in design maturation ponds in the influent by $4.2 * 10^8$ E – Coli / 100 ml and remove the fecal coliforms in anaerobic ponds are low and in the facultative ponds are 99% and the rate of evaporation is 10% from the volume of water in the facultative ponds, still 90% so the influent of the fecal coliforms to the first pond of maturation ponds is:

$$N_1 = 4.2 * 10^8 * \frac{(1 - 0.99)}{0.9} = 4.7 * 10^6 \text{ E – Coli / 100 ml}$$

Take $N_1 = 4.7 * 10^6$ E – Coli / 100 ml
Take $N_E < 2 * 10^3$ E – Coli / 100 ml
Where N_E based on the re-use resulting of the ponds.
$k^`$ shall be take n= 2 at $T = 20^0$ When the fecal coliforms group is based on the design constant disappearing and this factor depends on the temperature and this value taken when designing the ponds and when the temperature changed will applied the following equation:

$$\frac{K^`_T}{K^`_{20}} = \Theta\ (T - 20) \qquad (4.9)$$

Where: $K^`_T$ = die-off constant at temperature T^0.
$K^`_{20}$ = die-off constant at temperature $20\,^0C$.
Θ = temperature factor and will taken 1.07.

Retention time will take between 3 and 10 days. Suggested values for the number of ponds respectively after selecting the values of the previous design factors and choose the number of ponds that check number in bacterial behaviour emerging after a period less than $2 * 10^3$ colonic bacteria / 100 ml. And calculates the volume of the pond by:

$V = Q * T$ (4.10)

Where :
Q = flow m^3 per day.
T = Retention time (day).

It is repeated the number of ponds respectively according to the number of ponds. (2)

In the influent flow = $7.4 * 10^6$ bacteria coli / 100 ml and the number of bacteria in the effluent flow $<2 \times 10^3$ bacteria coli / 100 ml,so it will selected the retention time (R) and the number of ponds (n) according to the table (1) follows: (2)

Table 1. The relation between retention time (R) and the number of ponds (n).

No of ponds (n)	1	2	3	4	5
Retention time (R) day	1175	23.7	6.15	2.98	1.96

4-1-3-3 Advantages and disadvantages of the maturation ponds

Advantages

Rate on the removal of harmful bacteria is greater in smaller depth.

Disadvantages

In the design, do not depend on the maturation ponds and must be preceded by anaerobic - facultative ponds to get a high efficiency.

4-2 Design of the oxidation ponds in Sweden

4-2-1 Anaerobic ponds

4-2-1-1 Design criteria

The use of anaerobic ponds is primarily for treatment the wastewater with high biological oxygen demand loads and 50 - 70% is typical biological oxygen demand removal efficiency (see figure 4 –chapter 2).(3)
"They are deep ponds, 3-5 meters, allowing for low oxygen level conditions to prevail". (3;4)
The length/breadth ratio should be around 1 to 3, roughly square or rectangular. The Volumetric Organic Loading (VOL) rate (kg BOD/m^3.d) is big at high temperature in small ponds. (See Table 2)
Typical Volumetric Organic Loading rates are between 0.1 and 0.4 kg BOD /m^3.d and to prevent odor problems and prevent the pond from possibly becoming more of a facultative pond so 0.1 kg BOD/m^3.d value has been selected. To prevent organic overloading and to prevent odor problems 0.4 kg BOD/ m^3.d is selected.(3)
Volume required by anaerobic treatment pond, given the volumetric organic loading rate in table 2:

$$V = L / L_v \qquad (4.11)$$

Where:
V = Volume required for the pond (m^3)
L = total (soluble + particulate) influent BOD load (Kg BOD_5 / day)
L_v = volumetric organic loading rate which is the amount of BOD that the pond can be treat per volume (Kg BOD_5 /m^3. day)

Table 2.Permissible volumetric loading rates for anaerobic ponds as a function of temperature in Sweden.

Mean air temperature in the coldest month (T ^{0}c)	Permissible volumetric loading rate, Lv (Kg BOD_5/m^3.day)(5)
<10	Up to 40% BOD_5 removal
10 to 20	0,02*T - 0,10
20 to 25	0,02*T+ 0,10
> 25	0,35

The detention time of the anaerobic treatment pond depends strongly on case temperature and volumetric organic loading rates. (3)

"According to FAO the ponds detention time should not be less than 1 day and as the retention time increases, typically the percentage of BOD removal will increase. A detention time of more than 3 days is more effective". (3; 28)

$t = V / Q$ (4.12)

Where:

t = detention time (days).

V = volume of the pond (m^3).

Q = average influent flow rate (m^3/day).

"Other design requirements include the consideration of temperature. Warmer temperatures are preferred, as optimum temperatures are above25^{o}C. But as there have been limited studies performed for anaerobic treatment in temperatures less than 10^{o}C, accurate data on the level of treatment for these temperatures is unknown, but from Gambrill a value of up to 40% BOD_5 removal can be assumed" .(3;5)

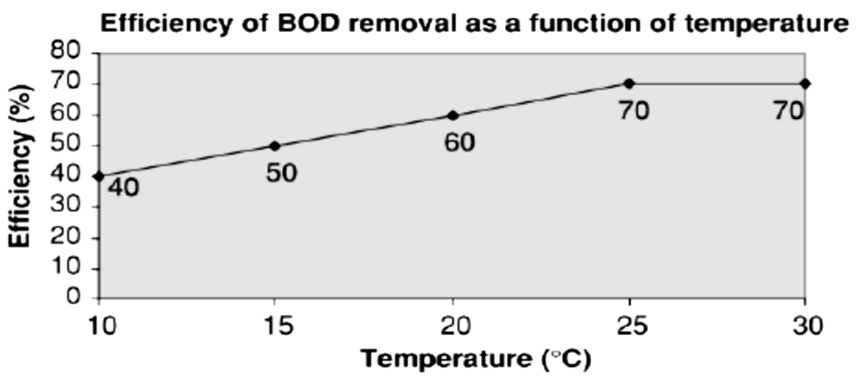


Figure 8: Relationship between BOD removal efficiency and temperature in anaerobic ponds.

"Chemical requirements of the system include the balance of pH, which is crucial in optimizing the efficiency of anaerobic treatment. Optimal pH is between 7.0 and 7.2 to prevent from odor problems and harm to bacteria". (6) PH range 6.6 to 7.6 is suitable and also easier to manage.(3)

4-2-1-2 Advantages and disadvantages of the anaerobic ponds

In Sweden or other mild-temperate climate regions the use of anaerobic treatment ponds can be limited due to the consistent below-zero temperatures in winter, which may cause the ponds to become too cold and therefore relatively ineffective.(3)

To ensure sufficient treatment occurs throughout the whole year, Therefore the design anaerobic treatment ponds by take in calculations the lowest average monthly temperature. (3)

Advantages include

"Production of very little excess sludge".
"Small pond volume needed".
"Low operating costs as there is no power required".
"Methane is a useful by – product". (3,4)

Disadvantages include

"Long start up period".
"No significant nutrient removal (nitrogen and phosphorous)".
"Post treatment always required, to remove remaining organic matter".
"Can produce an unpleasant odor".
"Requires sludge removal more often than facultative due to the smaller pond volume".
"Operates optimally at warmer temperatures, >25^{o}C". (3,6)

4-2-2 Facultative ponds

4-2-2-1 Design criteria

A facultative pond is a treatment pond that has BOD reduction both by anaerobic processes at the bottom and at the pond surface aerobic processes (See figure 5-chapter 2).(3)
"The soluble BOD is aerobically stabilized and suspended and colloidal BOD tends to settle and is decomposed by anaerobic bacteria". (3,4)
BOD can be removed across the facultative pond in total about 70 to 85 percent of the incoming (see table 3).
Different suggested detention times are presented in Table 2, and nutrient removals per areal unit in Swedish conditions are also presented in Table 3.

Cost of constructed the facultative ponds 245000 SEK / ha, the maintenance 3000 SEK / ha divided over 30 years. (3)
"An example of a facultative pond is a constructed wetland, which have been commonly created in Skåne in recent years. Wetlands have been stated as an important tool towards the Swedish 16 national environmental goals to decrease the nitrogen and phosphorous load in lakes and oceans .However the nutrient reduction efficiency in a wetland is questioned. One reason to the

varying nutrient efficiency is strongly dependent on environmental factors such as temperature, soil type and precipitation". (3,8)

Table 3. Suggested loading criteria of BOD for municipal ponds in Sweden. (7)

Environmental conditions	Suggested loading (Kg BOD / ha .day)	Detention time (days)
Cold seasonal climate	10 - 50	100 - 150
Temperature to warm climate	50 - 150	40 - 100
Hot or tropical temperature	150 - 350	20 - 40

Many studies are carried out in southern Sweden to evaluate what and how efficient the Swedish constructed wetlands are. The results show that the nitrogen reduction is between 600 – 1400 kg / ha*yr, and phosphorous reduction of 100 kg / ha * yr , and the cost of nitrogen reduction is 38 SEK per Kg reduced nitrogen. (3)

"Therefore the cost of nitrogen reduction in the conventional wastewater treatment plant is approximately 35-50 Kg nitrogen". (3,9)
The efficiency of the pond is low when the nutrients increase in the pond. To achieve a positive performance in the pond is the length to breadth ratio (minimum of 2:1),minimum of 500 mm freeboard and the depth of approximate 3.00 m. (3)

4-2-2-2 Advantages and disadvantages of the facultative ponds

Natural nutrient and BOD removal is the main goals with a wetland or other facultative pond to treatment wastewater, but there are also other positive effects connected to a well-maintained wetland such as aesthetic and recreational factors and increased biodiversity. The wetlands primarily attract many types of birds, frogs and other reptiles.(3)

Advantages

- "Efficient BOD reduction".
- "Nutrient reduction, by aerobic and anaerobic bacterial processes and also via uptake by other plants".
- "Natural aeration of the upper layer via movement of air and natural wave action".
- " Low energy consumption compared to conventional systems".
- " Solar induced disinfection". (7; 8)

Disadvantages

- Significant space requirements (big footprint).
- Size of the wetland also affects the positive side effects of the pond, such as function as bird habitat and aesthetically appealing free water surface.

- Efficiency is strongly affected by environmental factors.
- Demand continuous maintenance. (7; 8)

4-2-3 Facultative Aerated Lagoons

4-2-3-1 Design Criteria

Facultative Aerated Lagoons preferred on using of the facultative ponds in waste water treatment when the land is limited. This method named Facultative Aerated Lagoons because it used the aerobic and anaerobic process on lagoons so it is considered facultative, and provide the oxygen for a greater proportion of organic material by Mechanical aerators (see figure 9).(3)
The treatment method inFacultative Aerated Lagoons is different from facultative ponds in the major oxygen source, is that in a facultative aerated lagoon is the aerator, in contrast to photosynthesis by algae in facultative ponds.

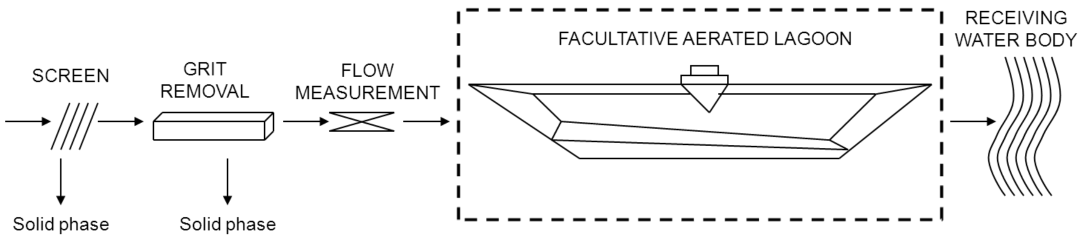


Figure 9: FACULTATIVE AERATED LAGOON

Settle in the bottom of the lagoon a proportion of suspended solids and organic colloidal material (BOD) are to form a sludge layer, and undergo anaerobic decomposition.The aerobic decomposition occurs above the sludge layer for the remaining suspended BOD and fine particulate BODnearer to the lagoon surface.And the hydraulic detention time reduces in a facultative aerated lagoon because of greater oxygen content.(3)

Some factors take into the design process are amount of water which need treatment,BOD_5 (and information the how much of it is soluble, fine particulate and in bigger particles which will settle more readily), Temperature, Environmental conditions, Land availability and Local or national regulation of effluent discharge quality (see table 4). (3)

Table 4. Design Criteria for Facultative Aerated Lagoons in Sweden.

Criteria	**Guidelines**
Detention Time	5 to 10 days
Depth of pond	2,5 to 4 m
Power level	0,75 to 1,50 W / m^3
Oxygen Requirement (OR)	OR= [a * Q * (S_0 * S)] / 1000 Where : OR = Oxygen requirement (Kg O_2/d) a = coefficient , varying from 0,8 to 1,2 Kg O_2 / Kg BOD_5 Q = influent flow (m^3 / d) S_0 = total (soluble + particulate) influent BOD_5 concentration (g / m^3) S = soluble effluent BOD_5 concentration (g / m^3) 1000 = conversion from g to Kg (since 1000g /1Kg)

Data from reference (3;4).

4-2-3-2 Advantages and disadvantages of the Facultative Aerated Lagoons

Advantages

-Not required large areas of land.
-The detention time is low.

Disadvantages

-This method is not economy because it need mechanical aerators.

4-2-4 Complete - Mix Aerated Lagoons – Sedimentation ponds

4-2-4-1 Design Criteria

The aerators used to treatment wastewaterwith Complete-Mix and the aerators used to introduce high levels of oxygen into the system and allow for the suspension of all solids in the liquid column. The detention time for such ponds is 2 to 4 days; the depth about 2.5 m to 4m in Complete- Mix Aerated Lagoons (see figure 10).

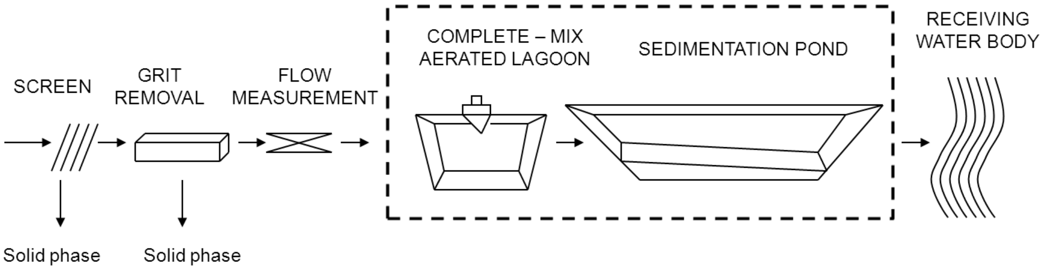


Figure 10: Complete-mixed Aerated Lagoon in series with Sedimentation Pond.

To calculate the volume of the lagoon (V) from equation (4.13) and to calculate Oxygen Requirement (OR) from equation (4.14).

$V = t_d * Q$ (Note: can then get the surface area required). (4.13)
Where:
t_d = Detention time in days.
Q = flow rate of effluent in m^3/day

$OR = a. Q. (S_o – S) / 1000$ (4.14)
Where:
OR = Oxygen requirement (KgO_2/d)
a = coefficient, varying from 1.1 to 1.4 KgO_2 /Kg BOD_5
Q = influent flow (m^3/day)
S_o = total (soluble + particulate) influent BOD_5 concentration (g / m^3)
S = soluble effluent BOD_5concentration (g / m^3)
1000 = conversion from kg to g (g / Kg)

Sedimentation ponds used after the Complete- Mix Aerated Lagoons because the effluent of complete-mix aerated lagoon contains high amounts of suspended solids; it cannot be freely discharged into nature.For the sedimentation pond the detention time is about 2 days or longer and the depth ≥ 1.5 m.

4-2-4-2 Advantages and disadvantages of Complete - Mix Aerated Lagoons - Sedimentation ponds

Advantages

- High quality for the discharged effluent.
- The effluent from the sedimentation pond contains lower solids amounts.
- Therefore it can be discharged into some receiving body directly. (3)

Disadvantages

- Consume a large energy. (3)

4-2-5 Maturation ponds

4-2-5-1 Design Criteria

Maturation ponds are known as polishing ponds and it generally shallower than the other types of ponds and the range in depths from 0.9 – 1 m (see figure 6 - chapter 2). To allow thelight to penetrate bottom and traffic air conditions through the full depth and to ensure that treatment the large amounts of wastewater. Maturation pond is safety pond in case of facultative pond failure so it is represent as a buffer and Table 5 shows the Design Criteria of Maturation ponds in Sweden. (3)

Table 5. Design Criteria of Maturation ponds in Sweden

	Climate	**Surface loading (Kg BOD/ha .day)**	**Detention time (days)**
Municipal criteria	Cold seasonal climate	10 - 20	20 - 30
Industrial criteria	Not applicable		

Data from reference (7).

Detention time must be long enough to allow for sufficient settling and a detention time at least 3 days is recommended at temperatures of 20°C. At temperatures less 20°C detention time varies from 4-5 days or 20-30 days to ensure no pollutants are allowed to be discharged from the wastewater treatment system into the natural water body. It is removed 70% of BOD during primary and secondary treatment. (3)

4-2-5-2 Advantages and disadvantages of the maturation ponds

Advantages include

- Removal of excess nutrients.
- Removal of pathogenic organisms.
- Removal of fecal coliforms. (3)

Disadvantages include

- Small removal of BOD.
- Additional costs.
- Additional land requirement. (3)

4-3 Design of the oxidation ponds in Illinois State

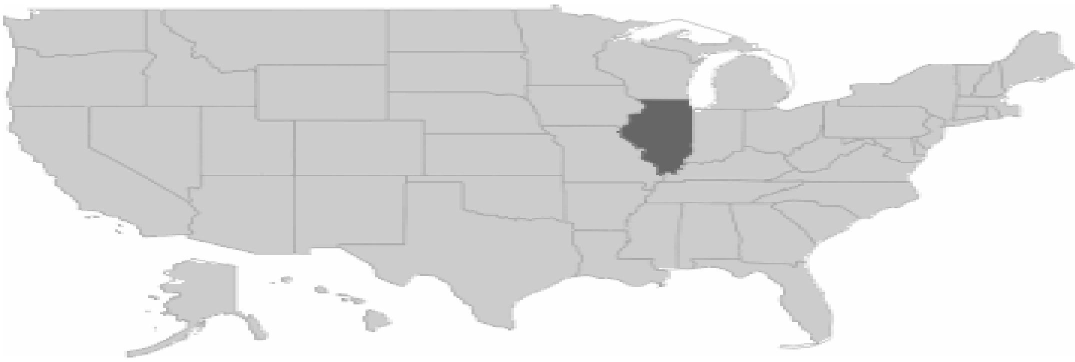


Figure 11: Illinois State in the map of the United States

For over 300 years oxidation ponds have been employed for treatment of wastewater in Illinois State in the United States (see figure 11).(13)

4-3-1 Anaerobic Ponds

4-3-1-1 Design Criteria

There is no aerobic zone in an anaerobic pond (see figure 4-chapter 2). Anaerobic ponds are usually deep and are subjected to heavy organic loading and anaerobic bacteria decompose organic matter to carbon dioxide and methane. The detention times are 20-50 day and the depths of anaerobic ponds usually range from 2.5 to 5 m (8 to 16 ft). (13)
"Normal operation, to achieve a BOD removal efficiency of at least 75 percent, entails a loading rate of 0.32 kg BOD/(m^3.d) (20 lb/(1000 ft^3.d)), a minimum detention time of four days, and a minimum operating temperature of 24^0C (75^0F)". (10;13)
For strong industrial wastewater and for rural communities with high organic load they have been used as pretreatment to facultative or aerobic ponds.
For municipal wastewater treatment they are not in wide application.

4-3-1-2 Advantages and Disadvantages of Anaerobic Ponds

Low production of waste biological sludge and no need for aeration equipment is the advantages of anaerobic ponds compared with an aerobic treatment process. The generation of odorous compoundsis an important disadvantage of the anaerobic pond represent in organic acids and hydrogen sulfide. Its incomplete oxidation of wastes requires a second-stage aerobic process. It requires a relatively high temperature for anaerobic decomposition of wastes.(22)

4-3-2 Facultative Ponds

It is called the wastewater lagoon and it is the most common type of oxidation ponds (see figure 5-chapter 2). It can also be used to follow trickling filters, aerated ponds, or anaerobic ponds. Facultative Ponds is a symbiotic relationship between bacteria and algae. The organic materials present in the aerobic zoneand it decompose into oxidized end products by the organisms and temperature is a major factor for the biological symbiotic activities.(27)

4-3-2-1 Design Criteria

In the manual design, several design formulas with operational data were presented. Facultative ponds are usually 1.2-2.5 m (4-8 ft) in depth often containing sludge deposits and the detention time is usually 5-30 days. (27)

"Typical organic loading rates are 22-67 kg BOD/(ha • d) (20-60 Ib BOD/(a.d)). Typical detention times range from 25 to 180 days. Typical dimensions are 1.2-2.5 m (4-8 ft) deep with 4-60 ha (10-150 acres) of surface area". (13;27)

"Facultative ponds are commonly designed to reduce BOD to about 30 mg/L; but, in practice, due to algae it ranges from 30 to 40 mg/L or greater. Volatile organic removal is between 77 and 96 percent. Nitrogen removal achieves 40-95 percent. Less phosphorus removal is observed, being less than 40 percent. Effluent TSS levels range from 40 to 100 mg/L, contributed by algae". (12;13)

The ponds are effective in removal of fecal coliform (FC) and it remove 200 FC/100 ml due to their dying off.Calculations of the volume of a facultative pond are also illustrated for the areal loading rate procedure, Gloyna equation, Marais-Shaw equation, plug-flow model, and Wehner-Wilhelm equation. The design methods of the areal loading rate method and Wehner and Wilhelm model are discussed. (13)

4-3-2-1-1 Areal loading rate method

"The design procedure is usually based on organic loading rate and hydraulic residence time. The recommended BOD loading rates based on average winter air temperature are given in Table 6". (13;14)

Table 6. Recommended BOD_5 Loading Rates for Facultative Ponds Illinois State. (13)

Average winter air temperature , ^{0}C	Water Depth		BOD loading rate	
	m	ft	Kg / (ha. D)	Ib / (acre. D)
< 0	1,5-2,1	3-7	11-22	10-20
0 - 15^{0}C (59^{0}F)	1,2-1,8	4-6	22-45	20-40
> 15^{0}C	1,1	3,7	45-90	40-80

The surface area required for the facultative pond can be expressed as the following equation (as can other processes):

"$\text{Area} = \dfrac{Q * BOD}{LR * 1000}$" (SI units) (4.15)

"$\text{Area} = \dfrac{Q * BOD * 8.34}{LR * 1000}$" (British units) (4.16)

Where:A = area required for facultative pond, ha or acre.

BOD = BOD concentration in influent, mg/L.
Q = flow of influent, m^3 / d or Mgal/d.
LR = BOD loading rate for average winter air temperature, Kg/(ha.d) or Ib/(a.d).
1000 = conversion factor 1000 g = 1 Kg.
8.34 = conversion factor, 1(Mgal/d) (mg/ L) = 8.34 Ib.(22)

For warm climates,BOD loading rate at the first cell in a series of cells (ponds) should not exceed 100 kg/ (ha.d) (90 lb/(a.d)), at average winter air temperature greater than 15^0C (59^0F), and 40 kg/(ha.d) (36 lb/(a.d)) for average winter air temperature less than 0^0C (32^0F). (13)

EXAMPLE 1

"The design flow of facultative ponds for a small town is 1100 m^3/d (0.29 Mgal/d). The expected influent BOD is 210 mg/L. The average winter temperature is 10^0C (50^0F).Design a three-cell system with organic loading less than 80 kg/(ha . d) (72 lb/(a . d)) in the primary cell. Also estimate the hydraulic detention time when average sludge depth is 0.5 m and there are seepage and evaporation losses of 2.0 mm of water per day". (13)

Solution

Step 1. Determine area required for the total ponds.
From Table 6, choose BOD loading rate
LR = 38 kg/(ha . d) (35 lb/(a . d)), **because mean air temperature is 10^0C**.
Using Eq. (4.15)

$$A = \frac{Q * BOD}{LR*1000} = \frac{1100m^3/dx210g/m^3}{38\ kg/(ha\ .d)\ x\ 1000\ g/kg}$$

$$= 6.08\ ha\ (16.6a) = 60,800\ m^2$$

Step2.Determine the area required for the primary cell (use LR = 80 kg/ ha .d).

$$Area = \frac{1100x210}{80\ x\ 1000} = 2.88\ ha$$

Step 3.Sizing for 3 cells.
Referring to Table 6, choose water depth of 1.5 m (5 ft) for all cells.

(a) Primary cell:
Area = 2.88 ha = 28,800 m^2
Use 100 m (328 ft) wide, 288 m (945 ft) long and 1.5 m (5 ft) deep pond.
(b) Two other cells:
Area for each = (60,800 - 28,800) m^2/2= 16,000 m^2

Choose 144 m in length, then the width= 111 m
The pond arrangements are as shown below.

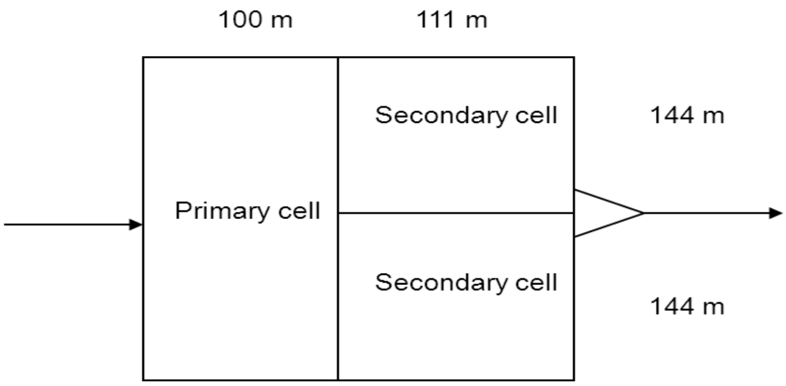


Figure 12 : The pond arrangements

Step 4. Estimate the hydraulic retention time.
(a) Calculate the storage volume V (when average sludge depth = 0.5 m)
$V = (1.5\ m - 0.5\ m) \times 60{,}800\ m^2 = 60{,}800\ m^3$

(b) Calculate the water loss V '
$V' = 0.002\ m/d \times 60{,}800\ m^2 = 122\ m^3/d$

(c) Calculate the HRT storage time

$$HRT = \frac{V}{Q - V'} = \frac{60{,}800\ m^2}{1100 m^3/d - 122 m^3/d} = 62\ days$$

4-3-2-1-2 Wehner and Wilhelm equation

"Wehner and Wilhelm (1958) used for a pond with an arbitrary flow-through pattern the first-order substrate removal rate equation, which is between a plug-flow pattern and a complete-mix pattern".(22;57)
Their proposed equation is:

$$"\frac{C}{C_0} = \frac{4a * \exp(1/2D)}{(1+a)^2 \exp(a/2D) - (1-a)^2 \exp(-a/2D)}" \qquad (4.17)$$

Where:
C = effluent substrate concentration, mg/L.
C_0 = influent substrate concentration, mg/L.
$a = (l + 4ktD)^{0.5}$.
k = first-order reaction constant, 1/h.
t = detention time, h.
D = dispersion factor = H / u * L.
H = axial dispersion coefficient, m^2/h or ft^2/h.
u = fluid velocity, m/h or ft/h.
L = length of travel path of a typical particle, m or ft.(22)

"The Wehner-Wilhelm equation for arbitrary flow was proposed by Thirumurthi 1969 as a method of designing facultative pond systems. Thirumurthi developed a graph, shown in Fig.13, to facilitate the use of the equation".(13;26;29)

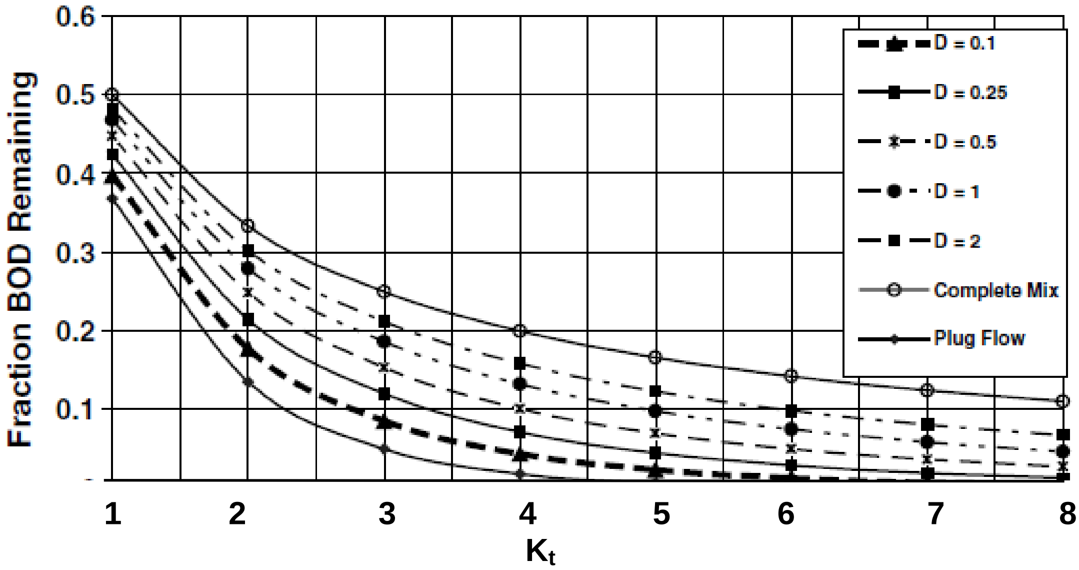


Figure 13. Relationship between k_t values and percent BOD remaining for various dispersions factors, by the Wehner-Wilhelm equation.

In Fig.13, the term k_t is plotted against the percent BOD_5 remaining (C/C_0) in the effluent for dispersion factors varying from zero for an ideal plug-flow pond to infinity for a complete-mix pond. For oxidation ponds the dispersion factors range from 0.1 to 2.0, with most values not exceeding 1.0 due to the mixing requirement.(13)

"Typical values for the overall first-order BOD_5 removal rate constant k vary from 0.05 to 1.0 per day; depending on the operating and hydraulic characteristics of the pond. The use of the arbitrary flow equation is complicated in selecting k and D values. A value of 0.15 per day is recommended for K_{20}". (13;15)

Temperature adjustment for k can be determined by:

"$K_T = K_{20} (1.09)^{T-20}$"   (4.18)

Where:
K_T = reaction rate at minimum operating water temperature T, per day.
K_{20} = reaction rate at 20^0C, per day.
T = minimum operating temperature, ^{0}C.

EXAMPLE 2

"Design a facultative pond system using the Wehner-Wilhelm model and Thirumurthi application with the following given data.
Design flow rate Q = 1100 m^3/d (0.29 Mgal/d); Influent TSS = 220 mg/L; Influent BOD_5 = 210 mg/L; Effluent BOD_5 = 30 mg/L; Overall first-order k at 20^0C = 0.22 per day; Pond dispersion factor; D = 0.5; Water temperature at critical period = 1^0C; Pond depth = 2 m (6.6 ft); Effective depth = 1.5 m (5 ft)".(13)

Solution

Step 1. Calculate the percentage of BOD remaining in the effluent.
C/C_0 = 30 mg/L x 100 %(210 mg/L) = 14.3%

Step 2. Calculate the temperature adjustment for K_{20} .
$k_T = K_{20} (1.09)^{T-20} = 0.22 (1.09)^{1-20} = 0.043$ per day

Step 3. Determine the value of k_{Tt} from Fig. 13.
At C/C_0 = 14.3% and D = 0.5
So k_{Tt} = 3.1

Step 4. Calculate the detention time for the critical period of year.
$t = 3.1/(0.043d^{-1}) = 72d$

Step 5. Determine the pond volume and surface area requirements.
Volume = Q*t = 1100 m^3/d x 72 d= 79,200 m^3

Area = volume / effective depth = 79,200 m^3/1.5 m
= 52,800 m^2= 5.28 ha = 13.0 acres

Step 6. Check BOD_5 loading rate.

$$\text{Loading} = \frac{1100m^3/d \times 210\ g/m^3}{5.28\ ha \times 1000\ g/kg}$$

= 43.8 kg/(ha . d)
= 40.0 lb/(a . d)

4-3-2-2 Advantages and disadvantages of the facultative ponds

Advantages

The ponds commonly receive no more pretreatment than screening (few with primary effluent). In the first or primary ponds they store grit and heavy solids to form an anaerobic layer. The ponds are effective for removal of fecal coliforms.(13)

Disadvantages

The presence of algae in the effluent is one of the most serious performance problems associated with facultative ponds. (13)

4-3-3 Tertiary ponds

4-3-3-1 Design Criteria

Tertiary ponds, also called polishing or maturation ponds. Itserves as the third stage process for effluent from activated-sludge or trickling filter secondary clarifier effluent. The water depth of the tertiary pond is usually 1-1.5 m (3-4.5

ft); BOD loading rates are less than 17 kg/(ha.d) (15 lb/(a.d)) and the detention times are vary from 4 to 15 days. (13)

4-3-3-2 Advantages and disadvantages of the Tertiary ponds

Advantages

They are used as a second stage following facultative ponds and are aerobic throughout their depth.Detention times are short.

Disadvantages

It can't used at high values of BOD.

4-3-4 Aerobic Ponds

Aerobic Ponds is containing aerobic bacteria and the aerobic bacteria used to oxidize the organic waste (see figure 6-chapter 2). (13)

4-3-4-1 Design Criteria

Aerobic ponds, also referred to as high-rate aerobic ponds, are relatively shallow to allowing the light to penetrate the full depth and it ranging from 0.3 to 0.6 m (1 to 2 ft).The ponds maintain dissolved oxygen throughout their entire depth.Hydraulic retention time (HRT) in the ponds is 3 to 5 days. To expose all algae to sunlight and to prevent deposition and subsequent anaerobic conditions,so the mixing is often provided. And dissolved oxygen is supplied by algal photosynthesis and surface reaeration. (13)

4-3-4-2 Advantages and disadvantages of the Aerobic ponds

Advantages

The use of aerobic ponds is limited because it need a warm and sunny climates, especially where a high degree of BOD removal is required.(13)

Disadvantages

The land area is not limited.
Very little coliform die-off occurs. (13)

4-4 Design of the oxidation ponds in BRAZIL

4-4-1 Anaerobic ponds

"Anaerobic ponds constitute an alternative form of treatment, in which the existence of strictly anaerobic conditions is essential (see figure 4-chapter 2).This is reached through the application of a high BOD load per unit of volume of the pond, which causes the oxygen consumption rate to be several times greater than the oxygen production rate. In the oxygen balance, the production by photosynthesis and atmospheric reaeration are, in this case, negligible ".(22)

The conditions should be met in the Anaerobic ponds are absence of dissolved oxygen ,adequate temperature of the liquid above 15°C and adequate pH close to or above 7. (22)

The anaerobic conversion takes place in two stages the first is Liquefaction and formation of acids (through the acid-forming bacteria, or acidogenic bacteria) and the second is formation of methane (through the methane-forming organisms, or methanogenic archaea). (22)

4-4-1-1 Design Criteria

Volumetric loading rates (L_v) usually are within the following range of 0.1 kgBOD$_5$/ m^3 .d to 0.3 kgBOD$_5$/ m^3 .d .The hydraulic detention time (t) is usually within the range of 3.0 d to 6.0 d, with detention times greater than 6 days for domestic sewage, the anaerobic pond can behave occasionally as a facultative pond. (22)

Values of the depth usually are in the range of 3.5 m to 5.0 m. But In the fact, the deeper pond is the better. However, deep excavations tend to be more expensive.Anaerobic ponds are square or slightly rectangular, with typical length / breadth (L/B) ratios 1 to 3. (22)

4-4-1-2 The Effluent BOD Concentration from the Anaerobic Pond

"The BOD removal efficiencies as a function of the temperature presented in Table 7 and illustrated in Figure 8 ". (22,30)

Table 7. BOD removal efficiencies in anaerobic ponds as a function of the temperature.

Mean air temperature of the coldest month – T (^{0}C)	BOD removal efficiency E (%)
10 to 25	2T+ 20
> 25	70

Once the removal efficiency (E) has been estimated, the effluent concentration (BOD_{eff}) of the anaerobic pond is calculated using the formulas:
"$E = (S_o – BOD_{effl}).100/ S_o$" (4.19)
Or
"$BOD_{eff}= (1 – E/100).S_o$" (4.20)
Where:
E = BOD removal removal efficiency.
S_o = influent total BOD concentration (mg/L)
BOD_{eff} = effluent total BOD concentration (mg/L)
The effluent BOD considered is the total BOD(different from the calculations of facultative ponds) in which the effluent BOD is split in terms of soluble BOD and particulate BOD. (22)

4-4-1-3 Design of Facultative Ponds Following Anaerobic Ponds

"The estimation of the effluent BOD concentration from the facultative pond can be done according to the limited methodology(see figure 5-chapter 2). The removalcoefficient K will be in this case lower than in primary facultative ponds, due tothe previous removal of the more easily degradable organic matter in the anaerobic pond. The remainder of the organic matter is harder to degrade, implying slower conversion rates. The following values of K have been suggested for secondary facultative ponds, using the complete-mix model:
K = 0.25 to 0.32 d^{-1}(20°C, secondary facultative ponds, complete-mix model)". (22)

4-4-1-4 Sludge Accumulation in Anaerobic Ponds

"The accumulation rate is in the order of 0.03 to 0.10 m^3/inhab.year". (31; 33;22).

"And the lower range is more usual in warm-climate areas. Other data available for accumulation rates are 2 to 8 cm/year". (32; 33; 34;50)

These values of yearly increases in the thickness of the sludge layer due to accumulation rates lower than 0.03 m^3/ inhab.year.So the system can operate for several years without needing to remove sludge, because there is a good grit removal in the preliminary treatment.(22)

The anaerobic ponds should be cleaned when the sludge layer reaches approximately 1/3 of the liquid depth or annual removal of a certain volume, in a pre-determined month, to includethe cleaning stage in a systematic way in the operational strategy of thepond. If the removal is not by emptying and drying inside the pond, the whole sludgemass should not be removed, it lead to a total loss of the biomass, requiring the anaerobic pond to start up again. (22)

4-4-2 Facultative ponds

4-4-2-1 Design Criteria

The main factors for the design of facultative ponds are surface organic loading rate; depth; detention time and geometry (length / breadth ratio). (22)

4-4-2-1-1 Surface organic loading rate

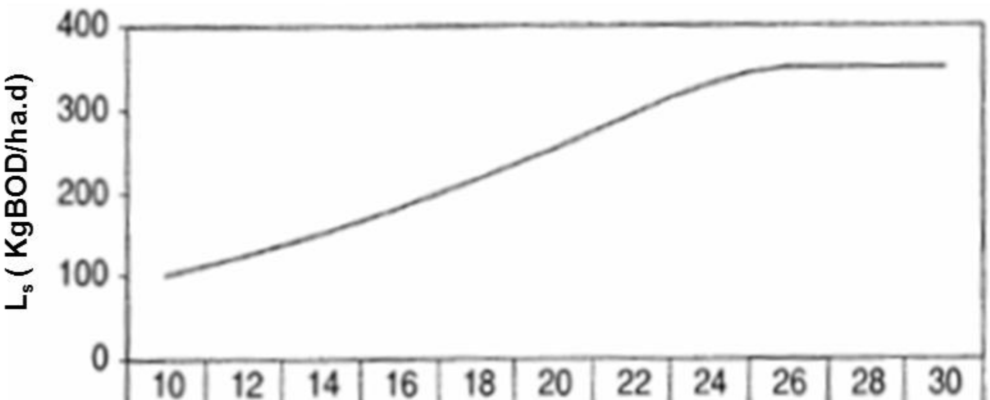
For thepond, the area required is calculated as a function of the surface loading rate Ls. The rate is expressed in terms of BODload that can be treated per unit surface area of the pond and it uses the mean temperature of the air in the coldest month.

"A = L / Ls" (4.21)

Where:
A = area required for the pond (ha)
L = influent total (soluble + particulate) BOD ($kgBOD_5/d$)
L_s = surface loading rate ($kgBOD_5/ha.d$) and to estimate L_s it will be consider the air temperature the same as the liquid temperature.(22)

So in tropical and subtropical climate regions, the following rates are:

• Regions with warm winter and high sunshine: L_s = 240 to 350 $kgBOD_5/ha.d$
• Regions with moderate winter and sunshine: L_s = 120 to 240 $kgBOD_5/ha.d$
• Regions with cold winter and low sunshine: L_s = 100 to 180 $kgBOD_5/ha.d$
Equation (4.21) used to initial estimate of the surface loading rate.
At temperatures above 25°C the Equation (4.21) leads to very high values of L_s. So it preferred at maximum value of 350 kgBOD /ha.d, the surfaceloading rate is limited for design purposes. (22)



T(°C)	10	12	14	16	18	20	22	24	26	28	30
—L_s	100	124	152	183	217	253	291	331	350	350	350

T(^{0}C)

Figure 14. Values of the surface loading rate as a function of the mean air temperature in the coldest month (according to Equation 4.22, ref.31)

$L_S = 350 \times (1.107 - 0.002 \times T)^{(T-25)}$ (4.22)

Where:
T =mean air temperature in the coldest month (^0C).
L_s= The surface loading rate (kg BOD /ha.d)presented in Figure 14.

There is no absolute maximum value for the surface area, beyond which the facultative ponds become unfeasible.So that the division of a single pond into ponds in parallel are depends on topography and the desirability to have more flexibility and improved hydraulics. (22)

4-4-2-1-2 Depth

The aerobic zone of the facultative pond depends on the penetration of the sun light to occur the photosynthetic process. In the water body the intensity of light reduce with the large depths.The depth range to be adopted in the design of facultative ponds lies between 1.5 to 3.0 m, although the following range is more usual 1.5 m to 2.0 m. (22)

Table 8. Aspects related to the pond depth

Shallow Depth	Deep Depth
- The depths lower than 1.0 m because of aerobic reactions. - Need large area to achieve the detention time requirement. - The penetration of light is practically complete through the depth but the lightenergy change with depth, even in clean water. - Due to the photosynthesis, the production of algae is maximized and the pH is usually high causing removal of phosphates and nutrients. - There can be the development of emergent vegetation, which is a potential shelter for mosquito larvae at a depth around 0.60 m or less at ponds. - Shallow ponds are more affected by ambient temperature variations along the day, and can reach anaerobic conditions in warm periods (Increase of the decomposition rate of the organic matter and a larger influence of the resolubilisation of by-products from the anaerobic decomposition of the sludge at the bottom).	- The higher depths provide a larger detention time for the oxidation of the organic matterand There is a larger storage volume for the sludge. - The performance of the pond is less affected bythe environmental conditions and throughout the year.It produces an effluent with a more uniformquality; and The bottom layer stays in anaerobic conditions, in which BOD removal rates and the pathogenic death rates are slower. - In the medium pond ,The anaerobic decomposition does not consume the dissolved oxygen .Thus, in the calculation of the DO balance, the fraction of the organic matter subject to the anaerobic decomposition can be taken in the calculations in the design. - The by-products of the anaerobic decomposition are released to the upper layers lead to appear bad odours but it reduces because in the aerobic layer the sulphide generated in the anaerobic decomposition is oxidized chemicallyand biochemically. - The deeper ponds allow future expansion for the inclusion of aerators becoming aerated lagoons.

"Based on the area and volume criteria, the depth H of the pond is a compromise between the required volume V and the required area A, considering that H = V/A.However, other aspects influence the selection of the depth of the pond as listed in Table 8 ". (17;22)

4-4-2-1-3 Detention time

The required detention time for the pond is a function of the kinetics of BOD removal and the hydraulic regime of the pond. (50)
"The detention time of the pond is associated with the volume and the design flow".(50)
"$T = V / Q$" (4.23)

Where:
t = detention time (d)

V = pond volume (m^3)
Q = average influent flow (m^3/d) and the average flow calculated by

Q average = (Q infl – Q effl)/2 (4.24)

Q effl = Q infl+ Q precipitation _ Q evaporation _ Q infiltration (4.25)

The detention time varies with the local conditions, especially the temperature. In **primary** facultative ponds, the detention times usually vary between15 to 45 days.In areas where the liquid temperature is higher, the detention times are lower and the volume reduction required is achieved. At case of low per capita sewage flow and a high BOD concentration, the detention time is to be high. (50)

4-4-2-1-4 Geometry of the pond

More frequently, the Length / breadth (L/B ratio) is situated within the range 2 to 4. (22)

4-4-2-2 Estimation of the Effluent BOD Concentration

The total effluent BOD is associated with soluble BOD where mostly remaining BOD from the influent wastewater after treatment and particulate BOD where BOD caused by the suspended solids in the effluent.From facultative ponds, the suspended solids are about 60 to 90% algae andeach 1 mg of algae generates 0.3 to 0.4 mgBOD_5/L.(22)

Monitoring of some ponds in Brazil leads to 1mg SS/L = 1.0 to 1.5 mg COD/L
Owing to the uncertainty regarding these aspects, a practical approach can be the one of not considering BOD from the algae or from the suspended solids in the effluent. As a result, BOD of the effluent from facultative ponds can be considered as being just the soluble BOD. In fact, the European Community established the standards for the effluents from theoxidation ponds.(22)
• Soluble (filtered) BOD_5 ≤ 25 mg/L;
• Soluble (filtered COD) ≤ 125 mg/L;
• Suspended solids ≤ 150 mg/L. (50)
For design purposes, the estimation of the particulate BOD may be based on effluent SS and the range ofSS effluent is 60 to 100 mg/L. (22)

4-4-2-3 Influence of the hydraulic regime

"BOD removal follows a first-order reaction (in which the reaction rate is directly proportional to the substrate concentration).Under these conditions, the hydraulic regime of the pond influences the efficiency of the system. Although the kinetics of BOD removal is the same in the different hydraulicregimes, the effluent BOD concentration varies. According to the first-order kinetics, BOD removal rate is higher the greater is BOD concentration in the medium".(22)

This aspect has a great implication in the performance of the pond, as seen below:
If there is a high BOD concentrationinpondsand the rate of removal is higher. Thisis the case, for instance, of predominantly longitudinal ponds, such as "the plug-flow ponds"(see fig.15).And the concentration close to the pond inlet is differentfrom the effluent concentration.(22)

Ponds that allow an immediate dispersion ofthe pollutant as a result of the homogenization of the entire tank cause the influent concentration to rapidly equal the low effluent concentration. The low concentrations prevailing in the pond lead to a lower BOD removal. This is the case of predominantly square ponds, such as "complete-mix ponds" (See fig. 16). And the concentration in the pond, close to the inlet, is equal to the concentration at the outlet.These two types of idealized ponds characterize an envelope, inside which all the existing ponds are placed in practice. (22)

4-4-2-4 The hydraulic models

The hydraulic models used in design the oxidation ponds more frequently such as plug flow, complete mix, and complete-mix pond in series and dispersed flow.it will be presented the Characteristics of the hydraulic models and performance the evaluation of the oxidation ponds .(22)

4-4-2-4-1 Plug flow

Plug-flow ponds are idealized ponds, since complete absence of longitudinal dispersion is difficult to obtain in practice. The fluid particles enter to the tank continuously in one end, pass through the pond, and then discharged at the other end, in the same sequence in which they entered the pond. The fluid particles move as a plug, without any longitudinal mixing. The particles stay in the tank for a period equal to the theoretical hydraulic detention time. Plug flow is reproduced in long tanks with a large length-to-breadth ratio, in which longitudinal dispersion is minimal. The particles in the tank are immediately dispersed in the entire pond and the influent and effluent flows are continuous. (22)

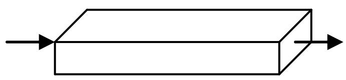


Figure15. Plug flow

4-4-2-4-2 Complete mix

Complete-mix ponds are known as CSTR or CFSTR (continuous-flow stirred tank ponds).Complete mixing can be obtained in tanks in which the contents arecontinuously and uniformly distributed. Complete-mix ponds are idealized ponds, since total and identical dispersion is difficult in practice to occur. (22)

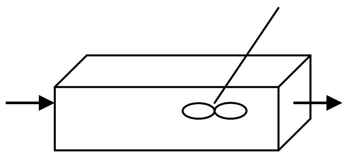


Figure16. Complete mix

4-4-2-4-3 Complete-mix pond in series

Complete-mix ponds in series are used to have model for the hydraulic regime of ponds in series that exists between the idealized plug flow and complete mix. If the series is composed of only one pond, the system reproduces a complete-mix pond. If the system has an infinite number of ponds in series, plug flow is reproduced. Influent and effluent flows are continuous. Ponds in series are commonly applied to maturation ponds. (22)

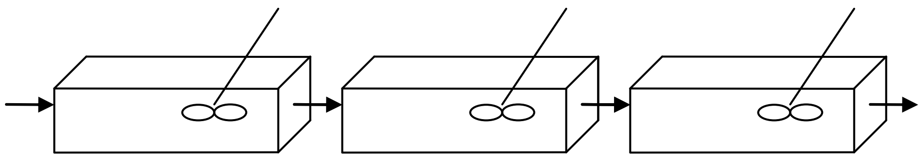


Figure17. Complete-mix pond in series

4-4-2-4-4 Dispersed flow

Dispersed flow is occurring in any pond with an intermediate degree of mixing between the two idealized extremes of plug flow and complete mix. In reality, most ponds present dispersed-flow conditions. However, because of the greater difficulty in their modeling, the flow pattern is frequently represented by one of the two idealized hydraulic models. The influent and effluent flows are continuous.The formulas used for determination the calculation of the effluent soluble BOD concentration (S) presents in Table (9).(22)

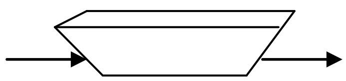


Figure18. Dispersed flow

Table 9.Formulas for the calculation of the effluent soluble BOD_5 concentration (S) for the various hydraulic regimes.

Hydraulic regime Scheme	Formula for the soluble effluent BOD_5 concentration
Plug flow	$S = S_o.e^{-K.t}$
Complete mix (1 cell)	$S = \frac{S_o}{1 + K.t}$
Complete mix (equal cells in series)	$S = \frac{S_o}{(1 + K.t/n)^n}$
Dispersed flow	$S = S_o.\frac{4ae^{1/2d}}{(1+a)^2 e^{a/2d} - (1-a)^2 e^{-a/2d}}$ $a = (1 + 4K.t.d)^{0.5}$

S_o = total influent BOD concentration (mg/L)
S = soluble effluent BOD concentration (mg/L)
K = BOD removal coefficient (d^{-1})
t = total detention time in the system (d)
n = number of ponds in series (__)
d =dispersion number (dimensionless)

4-4-2-5 BOD removal according to the complete-mix model

However, in the design of facultative pondsfor one or more ponds the complete-mix model has been more frequently adopted due to the following reasons:
- The calculations are simpler with the complete-mix model.
- BOD removal coefficients for thecomplete-mix modelare available in literature.
-There is no need to determine the dispersion number of the pond.
- Facultative ponds are not especially elongated, and deviations from a complete-mix pond are not substantial.

"For the most frequent case of the design according to the complete-mix model, in the following table". (11;17;18;19)

Table 10.Range of K values may be used for design facultative ponds

Pond	K value (20°C)
Primary ponds (receiving raw wastewater)	0.30 to 0.40 d^{-1}
Secondary ponds (receiving effluent from a previous pond or reactor)	0.25 to 0.32 d^{-1}

Because the raw wastewater contains more easily biodegradable organic matter, soin the primary facultative pondsthe BOD removal coefficient is higher.And the effluent from anaerobic ponds or anaerobic reactors has a

more slowly biodegradable organic matter, since the more easily degradable fraction has been already removed in them. So lower K values are in the secondary facultative ponds, maturation ponds or polishing ponds. The value of K can be corrected at various temperatures using the followingequation.(50)
"$K_T = K_{20}.\ \Theta^{(T-20)}$" (4.26)
Where:
K_T = BOD removal coefficient at a temperature T (d^{-1})
K_{20}= BOD removal coefficient at a temperature of 20°C (d^{-1})
T = liquid temperature (^{0}C)
Θ = temperature coefficient (-)
It should be noted that different values of Θ are proposed in the literature.
"ForK = 0.35 d^{-1}, the temperature coefficient is Θ = 1.085". (11;22)
"For K = 0.30 d^{-1}, the reported value is Θ = 1.05". (19;22)

4-4-3 Maturation Ponds

4-4-3-1 Design Criteria For Maturation Ponds

The maturation ponds should reach high coliform removal efficiencies (E >99.9 or 99.99%), so it can be used the effluent for direct uses, such as irrigation (see figure 6 -chapter 2). The overall removal efficiency in a system comprised by a series of ponds with different dimensions and characteristics is given by:
"$E = 1 - [(1 – E_1) \times (1 - E_2) \times ... \times (1 - E_n)]$ " (4.27)
where:E = Overall removal coliform efficiency.
E_1= Removal efficiency in pond 1.
E_2= Removal efficiency in pond 2.
E_n = Removal efficiency in pond n. (22)

In this equation, all removal efficiencies should be expressed as a fraction, and not as percentage (e.g. 0.9, and not 90%).In case the ponds have the same dimensions and characteristics, the formula is simplified to:

" $E = 1 – (1 – E_n)^n$ " (4.28)
Where:E = overall removal coliform efficiency.
E_n = removal efficiency in any pond of the series.
n =number of ponds in the series.
In this equation, all removal efficiencies should be expressed as a fraction, and not as percentage (e.g. 0.9, and not 90%).

If the removal efficiencies are expressed in terms of log units removed, the overall removal is given by the sum of the individual efficiencies in each pond, irrespective of the dimensions and characteristics being the same or not:
"Log units = (log units pond 1) + (log units pond 2) + ……….. + (log units pond n)" (4.29)
Where:log units = log units removed in the overall system.
log units pond 1 = log units removed in pond 1.
log units pond 2 = log units removed in pond 2.
log units pond n = log units removed in pond n.

Maturation ponds are usually designed with shallow depths 0.8 - 1.0 m for increasing photosynthesis and the bactericidal effect of the UV radiation.
3 days is the minimum detention time to avoid short circuits and thewashing-out of the algae. Average removal efficiency of helminth eggs from the wastewater used to represent average operation conditions:

$$E = 100 \times [1 - 0.14 \times e^{(-.49 \times t + 0.0085\, t^{2})}] \qquad (4.30)$$

Removal efficiency of helminth eggs from the wastewater according to the lower confidence limit of 95% (to be used for design, as a safety measure):

$$E = 100 \times [1 - 0.41.e^{(-0.4 \times t + 0.0085 \times t^{2})}] \qquad (4.31)$$

where: E = removal efficiency of helminth eggs (%).
t = hydraulic detention time in each pond of the series (d).

4-4-3-2 The Effluent Coliform Concentrations

The decay of the pathogenic organisms such as bacteria and viruses areindicators of faecal coliforms, follows first-order kinetics. The die-off rate of pathogens is proportional to the pathogen concentration at any time according with the first-order reactions. If the pathogenic concentrations are great, the die-off is big rate.In the coliform removal efficiencyfrom the ponds, the hydraulic regime has a great influence.The decreasing order of efficiency is:

- Plug-flow pond — greater efficiency
- Complete-mix ponds in series
- Single complete-mix pond — lower efficiency

The dispersed-flow regimerepresents well pondsthat approach both plug-flow and complete-mix conditions so it is not listed above. The determination of the coliform count in the effluent from ponds by the formulas, as a function of the different hydraulic regimes presents at Table (11).

Table 11. Formulas for the calculation of the effluent coliform concentration (N) from ponds.

Hydraulic regime Scheme	**Formula for the effluent coliform concentration (N)**
Plug flow	$N = N_o.e^{-Kb.t}$
Complete mix (1 cell)	$N = \dfrac{N_o}{1 + K_b.t}$
Complete mix (equal cells in series)	$N = \dfrac{N_o}{(1 + 4K_b.t./n)^{n}}$
Dispersed flow	$N = N_o.\dfrac{4ae^{1/2d}}{(1+a)^2 e^{a/2d} - (1-a)^2 e^{-a/2d}}$, $a = (1 + 4K_b.t.d)^{0.5}$

N_o = coliform concentration in the influent (org/100mL).

N = coliform concentration in the effluent (org/100mL).
K_b =bacterial die - off coefficient (d^{-1}).
t = detention time (d).
n = number of ponds in series (-).
d = dispersion number (dimensionless).

4-4-3-3 Idealized hydraulic regimes

Assumeponds in series or a pond approaching plug flow isrequiredto have high efficiencies on removal of the fecal coliforms.So the theoretical relative pond volumes required presents in table (12), as a function of the number of cells. All the values are expressed as a function of the dimensionless product (K_b.t).If the value of K_b is known, the table can be used for the direct calculation of thetotal volume required (calculation of t, followed by the calculation of V, knowing that V = t.Q).

Table12. Theoretical relative volumes necessary to reach a certain removal efficiency,as a function of the number of complete-mix ponds in series.

Number of ponds in series	Relative volume (dimensionless product K_b.t)			
	E=90%	E=99%	E=99.9%	E = 99.99%
1	9.0	99	999	9999
2	4.3	18	61	198
3	3.5	11	27	62
4	3.1	8.6	18	36
5	2.9	7.6	15	27
∞ (plug flow)	2.3	4.6	6.9	9.2

The interpretation of Table 12 leads to the following comments:

• The ideal plug-flow pondneed small volumes in comparison to the different systems. In practice plug-flow conditions are seldom achieved

• Forhigh removal coliforms with only one ideal complete-mix pond, it is necessaryhigh volumes.

• High reduction of volume of ponds in series is occurs if it more than 3 ponds.

• These comments are valid assuming the ponds to be ideal ponds.

The efficiencies and the number of logarithmic unitsremoved illustrates for different values of the dimensionless pair K_b.t and the number of ideal complete-mix ponds in series present in Figure 19.

An efficiency of E = 90% corresponds to theremoval of one logarithmic unit ; E = 99% ⟶ 2log units; E = 99.9% ⟶ 3 log units; E = 99.99% ⟶ 4 log units; E = 99.999% ⟶5 log units, according to the formula:

log units removed = - $\log_{10}$ [(100 - E)/100]   (4.32)

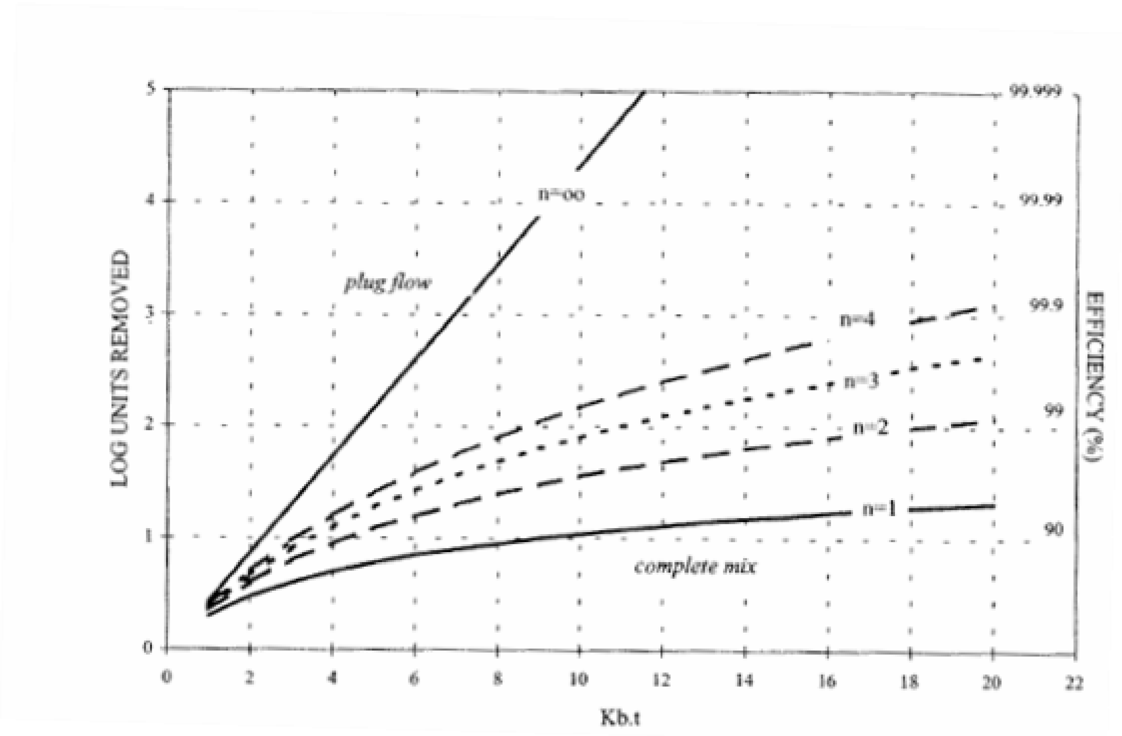


Figure 19. Coliform removal efficiencies, for different values of K_b.t and number of cells in series, assuming the complete-mix hydraulic regime.

The ideal plug-flow pond in figure 19 has big efficiency, so plug flow is an idealized hydraulic regime. Removal efficiencies above 99.9% without excessively large detention times can only be reached with a number of cells in series greater than four or preferably with a plug-flow regime.

The ideal plug-flow pond, Zero dispersion is hardly achievable in a pondin practice. It can be only approached (but not reached) through the adoption of a low dispersion, induced by baffles.

The behavior of ponds follows the dispersed-flow hydraulic regime, and not the idealized regimes of complete mix and plug flow, in reality.The values of the efficiency E and the number of logarithmic units removed as a function of the dimensionless pair K_b.t and the dispersion number d presents in Figure 20. In the dispersed-flow regime usually the coefficient $K_b$is different from the value adopted for the complete-mix regime. In the case of a single pond, figure 22 shows clearly the importance of having a pond with a low dispersion number, tending to the plug-flow regime, in order to increase the removal efficiency.Dispersion number is lower than 0.3 is needed to get efficiencies greater than 99.9% (3-log removal) without excessive detention times. Thesedispersion numbers are only obtained at length/breadth (L/B) ratio greater than 5 or 10 in ponds.

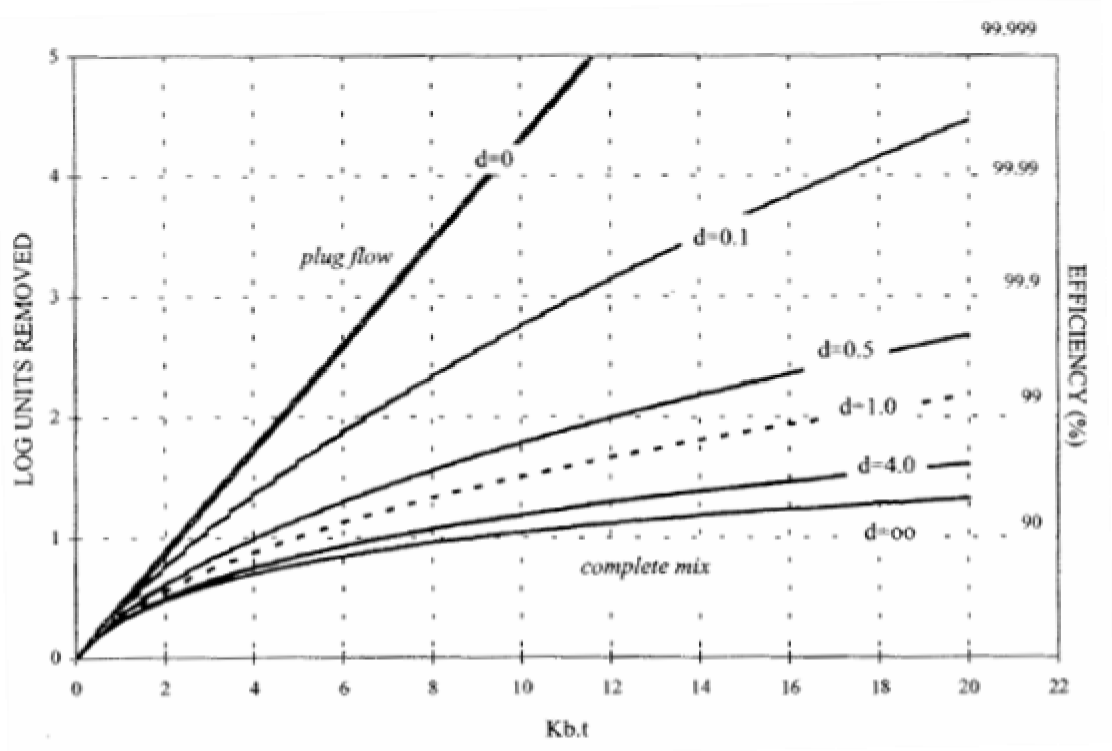


Figure 20. Coliform removal efficiency and number of log units removed in a single pond, for different values of K_b.t and d, assuming the dispersed-flow hydraulic regime

The next Figure presents the number of logarithmic units removed and the removal efficiency in maturation ponds, expressed as a function of the Length / breadth (L/B) ratio.The relationship between the L/B ratio and the dispersion number d was calculated in figure 21, using the equation:

d =1 / (L/B)   (4.33)

In a pond, the calculation of the L/B ratio with internal divisions can be approximated by:

• Divisions parallel to the breadth B:

" $L/B = \frac{B}{L} (n + 1)^2$ "   (4.34)

• Divisions parallel to the length L:

" $L/B = \frac{L}{B} (n + 1)^2$ "   (4.35)

Where:

L/B = resultant internal length / breadth ratio in the pond

L = length of the pond (m)

B =breadth of the pond (m)

n = number of internal divisions

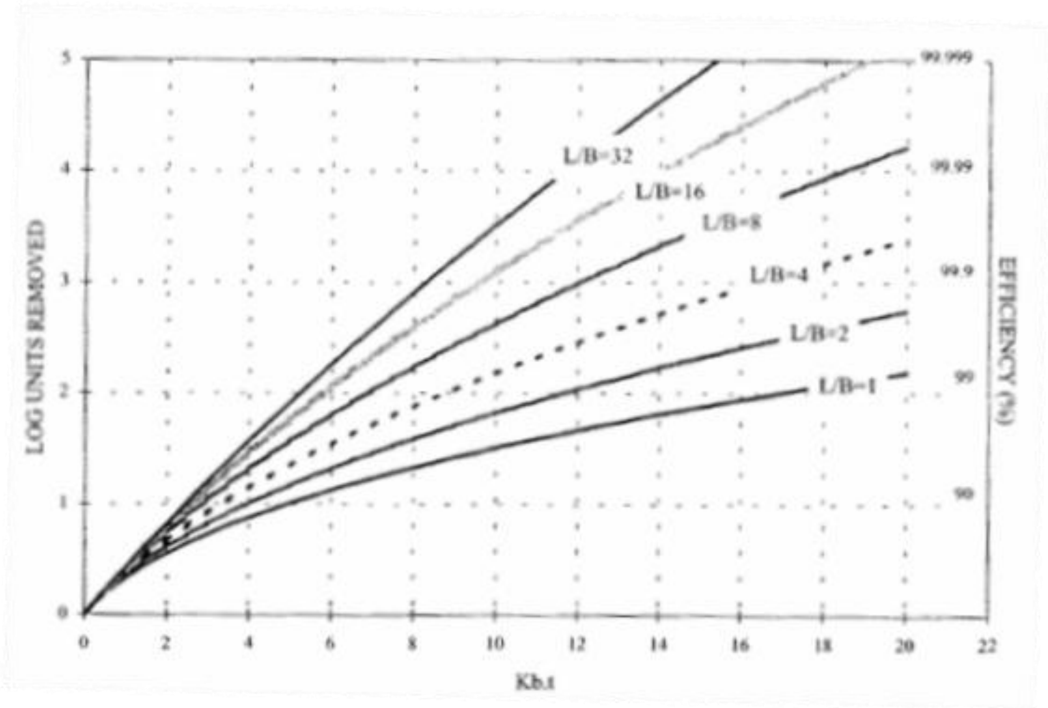


Figure 21. Coliform removal efficiency and number of log units removed for different values of K_b.t and L/B ratio, assuming dispersed flow.

The relationship between L/B and d was calculated according to equation (4.33).

4-4-3-4 The coliform die-off coefficient K_b according to the dispersed - flow regime

"An equation correlating K_b (dispersed flow) with the depth and the hydraulic detention time was determined through non-linear regression analysis with the available data (von Sperling, 1999):
"K_b (dispersed) = $0.917.H^{-0.877}.t^{-0.329}$" (33 ponds in Brazil) (4.36)
"The Coefficient of Determination was very high ($R^2 = 0.847$), indicating a good fitting of the proposed model to the experimental data". (22;23)

Dispersed - flow regime have simple structure, Despite ofthere was the advantage of depending only on variables which, in a design application, are known beforehand (H and t). But in practice it is difficult in reproducing the wide diversity of situations.(22)

Dispersion number assumed as d = 1/(L/B) and in the world from 82 facultative and maturation ponds (n) are 140 results in Figure 22 from the application for equation(4.37)and $R^2 = 0.500$. (22)

"K_b (dispersed) = $0.542.H^{-1.259}$" (82 ponds in the world) (4.37)

Table 13. Values of K_b (dispersed flow), obtained from Equation 4.37 **($K_b = 0.542.H^{-1.259}$) for facultative and maturation ponds.**

H (m)	0.6	0.8	1.0	1.2	1.4	1.6	1.8	2.0	2.4
$K_b(d^{-1})$	1.03	0.72	0.54	0.43	0.35	0.30	0.26	0.23	0.18

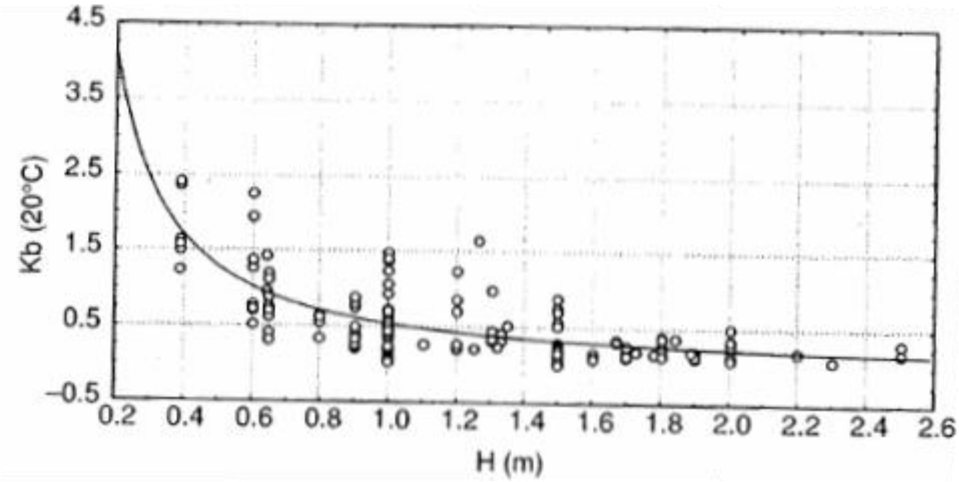


Figure 22. Regression analysis between K_b (20°C, dispersed flow) and the depth H of the ponds.

Less practical, because the variables that are not known at the design stage. In spite of the limitations, the model lead to a very good prediction of the logarithm of the effluent coliform concentrations from the 33 ponds (R^2=0.959).Then the databaseenlarged to 82 ponds and 140 mean datain Brazil and in other countries (Argentina, Colombia, Chile, Venezuela, Mexico, Spain, Belgium, Morocco and Palestine). Although the coefficient of determination was reduced to R^2= 0.505, Equation 4.36 was still shown to be valid and there is smaller influence from the hydraulic detention time and it could be removed from the equation, without significantly affecting the performance of the model.

The prediction of the log of the effluent coliform concentration was still entirely satisfactory from equation (4.37), Figure 22 and Table 13, showing the values of K_b and the best-fit curve. For low values of the hydraulic detention time and depths lower than 1.0 m, Equation 4.37 approaches Equation 4.36. (22)

In maturation ponds in series occur Low values of H and t simultaneously, which justifies that the simpler model keeps its practical applicability.

According to the dispersed-flow and complete-mix hydraulic regimes for facultative and maturation ponds, the typical range of resultant values of the coefficient K_b calculated using the last methodologies presents in Table 13.

The ranges of K_b for dispersed flow are much narrower than those for complete mix; it can be observed.K_b can be correctedfor various temperatures, by the formula:

"$K_{bT} = K_{b20} . \Theta^{(T-20)}$"   (4.38)

Where:Θ =temperature coefficient, the values of Θ vary andvery high values (Θ = 1.19).

For an increase of 1°C in the temperature, K_b increase 7%.However these values are overestimated and the values of Θ to be assumed should be in the range of **1.07**.(22)

Table 14. Summary of the ranges of typical values of K_b (20°C) for facultative and maturation ponds, according to the dispersed-flow and complete-mix models.(22)

Pond type	Detention time t (d)	Depth H (m)	L/B ratio	K_b dispersed flow (d^ -1)	K_b complete mix (d^-1)
Facultative	10 to 20	1.5 to 2.0	2 to 4	0.2 to 0.3	0.4 to 1.6
	20 to 40				1.6 to 5.0
Maturation (unbaffled , in series)	3 to 5 (in each pond)	0.8 to 1.0	1 to 3	0.4 to 0.7	0.6 to 1.2
Maturation (baffled, single pond)	10 to 20	0.8 to 1.0	6 to 12	0.4 to 0.7	(*)
Maturation (baffled, in series)	3 to 5 (in each pond)	0.8 to 1.0	6 to 12	0.4 to 0.7	(*)

Larger values of K_b associated to smaller values of t, smaller values of H and larger values of L/B.

(*) Baffled maturation ponds: adoption of the dispersed-flow model isrecommended.

4-4-3-5 Quality Requirements for the Effluent

If the effluent for unrestricted irrigation is used, the recommended values are faecal coliforms≤ 1,000 faecal coliforms/ 100 mL and helminthes eggs ≤ 1 egg / L .There is no limits for coliforms and a limit for only helminthes eggs ≤1 egg / L for restricted irrigation. So the coliform counts in the effluent should be very lowforagricultural reuse or the receiving body. The faecal coliform rates in the raw sewage are ranged from 10^6 to 10^9 org / 100 mL,so the removal efficiencies should be extremely high to be 3 - 6 log units (99.9 to 99.9999%) in terms of the geometric mean.(22)

It is easy to use the geometric mean instead of the arithmetic mean, fordifferent values of magnitude. This is the case in the monitoring of coliforms which vary within a very wide range, for instance, from 10^6 to 10^9FC / 100 mL in raw wastewater. The higher values have a great weight on the arithmetic mean,distorting the concept of the mean as a measure of central tendency. The geometric mean is given by the n root of the product of the n terms:

" Geometric mean = $(X1.\ X2\ ...\ Xn)^{1/n}$ " (4.39)

The geometric mean can be also calculated by:

" Geometric mean = $10^{\text{(arithmetic mean of the logarithms)}}$" (4.40)

The following statement is also important, and easily obtainable from the considerations above:
"Log_{10} of the geometric mean = arithmetic mean of the $\log_{10}$" (4.41)

4-4-4 Facultative aerated lagoons

4-4-4-1 Design criteria

The following criteria should be considered:

4-4-4-1-1 Detention time

To allow a high removal of BOD, The detention time (t) should be assumed. Usually, the values of the detention time vary in the range of 5 to 10 d (see figure 9).

4-4-4-1-2 Depth

The depth of the pond (H) should be selected in order to satisfy the compatibility with the aeration system and need of an aerobic layer to oxidize the gases from the anaerobic decomposition of the bottom sludge so the depth varies in the range of 2.5 to 4.0 m.

4-4-4-2 The Effluent BOD Concentration

The evaluation of the effluent BOD concentration follows a similar procedure to that used for the facultative ponds similarly to the facultative lagoons, the effluent from facultative aerated lagoons is constituted of soluble BOD and particulate BOD. However, the latter is not anymore associated predominantly to algae.
$BOD_{tot} = BOD_{sol} + BOD_{part}$ (4.42)

Where:
BOD_{tot} =total BOD_5 of the effluent (mg/L)
BOD_{sol}= soluble BOD_5 of the effluent (mg/L)
BOD_{part} = particulate BOD_5 of the effluent (mg/L)

4-4-4-2-1 Soluble effluent BOD

The value of BOD removal coefficient K is higher at the facultative aerated lagoons. Typical values for the complete-mix regime are in the range of K = 0.6

to 0.8 d^{-1} for the liquid temperature of 20 ^{0}C. For other temperatures, another Equation can be used, with Ѳ = 1.035.

$L_S = 350 \times (1.107 - 0.002 \times T)^{(T-25)}$ (4.43)

The following aspects should be taken in design for the design of the aeration system:

Facultative aerated lagoons allow to the particulate organic matter of the raw sewage to sediment, which occur anaerobic decomposition in the bottom sludge.

The influent BOD value (S_o) available for aerobic oxidation is lower than the total value in the raw sewage. And the value of S_o to be assumed in the calculations depends on the anaerobic activity, which is a function of the temperature in the liquid.

Consequently, the following two conditions can happen regarding the organic matter in the bottom sludge:

• Anaerobic decay with hydrolysis and acidification, but without methanogens is at cold weather and S_o= 100% of total influent BOD.

Comment:

There are regions with cold periods in which the removal of BOD stage does not fully occur. Which exert an oxygen demand in the aerobic layer. To occur aerobic oxidation,BODcan be considered as being equal to S_o.

• Anaerobic decay with hydrolysis, acidification, and methanogens at warm weather and S_o = 40% to 70% of total influent BOD.

Comment:

Under conditions in which the liquid temperature is > 15°C, the anaerobic conversion is complete, including all the stages. Because a fraction of BOD is oxidized anaerobically in theBottom, the value of S_o considered for the estimation of the effluent BOD and it is ranged from40 to 70%of the influent BOD.

However BOD load to be aerobically oxidized can be considered
as being equal to the total influent load (S_o = BOD of the influent)for design purposes.(22)
"S_o = 100% of total (soluble + particulate) BOD_5 of the influent". (4.44)

4-4-4-2-2 Particulate effluent BOD

The power level represents the energy introduced by the aerators per unit volume of the pond, being obtained from the formula
$\Phi = P/V$ (4.45)

Where: Φ= power level introduced by the aerators (W/m^3)
P = power for aeration (W)
V = pond volume (m^3)

The power level is great if the quantity of suspended solids is big. That can remain dispersed in the liquid medium. The values presented in table 15 are only estimates. The mixing intensity depends on the number and distribution of aerators (in the case of mechanical aeration) and on the volume and geometry of the pond. (36)

Table 15. Suspended solids concentrations that can be maintained dispersed in the liquid as a function of the power level.

Power level (Φ) (W/m^3)	SS (mg/L)
0.75	50
1.75	175
2.75	300

As a result, Facultative aerated lagoons work with low power levels in the range of 0.75 to 1.50 W/m^3, since one of their objectives is exactly to facilitate the sedimentation of the solids. In the effluent from the lagoon,the SS concentrations would be in the rangeof 50 to 140 mg/L.The expected value for the effluent particulate BOD can be assumedfrom 0.3 to 0.4 $mgBOD_5$/mgSS.
Thus, each 1 mg/L of SS produces a particulate BOD between 0.3 and 0.4 mg/L.In order to improve the settling conditions and the effluent quality, sothe outlet zone of the lagoon may be left without aerators.

4-4-4-3 Oxygen Requirements

The amount of oxygen to be supplied by the aerators for the aerobic oxidation of the organic matter should usually be equal to the total ultimate influent BOD.BOD_u is reached at the end of a long period, in the order of 20 days. BOD_u is therefore higher than BOD_5, since the latter is exertedonly until the fifth day.BOD_u /BOD_5ratio is frequently adopted in the range of 1.2 and 1.5.
So the amount of oxygen to be supplied can be calculated as:
$OR = a. Q. (S_o - S) /1000$ (4.46)
Where:OR = oxygen requirement (kgO_2/d).
a = coefficient, varying from 0.8 to 1.2 $kgO_2/kgBOD_5$.
Q = influent flow (m^3/d).
S_o = total (soluble + particulate) influent BOD_5 concentration (g/m^3).
S = soluble effluent BOD_5 concentration (g/m^3).
1000 = conversion from kg to g (g/kg).

4-4-4-4 Aeration System

The aspects should be taken in the designwhile designing the Aeration System are:

*There should be a minimum of two aerators in small ponds andin the aerated zone of the lagoon; the aerators should be distributed homogeneously.

*In close to the inlet zone can placed a larger number of aerators or more powerful aerators, where the oxygen demand is higher. And at adjacent aerators should have opposite rotation directions, that is,one should be clockwise and the other anti-clockwise.

*If the lower effluent SS concentrations are required, the final area of the lagoon can be without aerators, in order to provide better settling conditions.

*The manufacturers' data should be consulted with relation to the recomm-ended lagoon depth, influence zone of each aerator, oxygenation efficiency.

The usual power of aerators are 1; 2; 3; 5; 7.5; 10; 15; 20; 25; 30; 40 and 50HP.There are high-speed aerators with greater powers, but these aerators are less efficient. In the bottom pond, Anti-erosion plate situated underneath the aerator. The oxygenation efficiency ($OE_{standard}$) of the aerators is ranged from1.2 to 2.0 kgO_2/kWh, at standard conditions.

The types of the influence area of a mechanical aerator are **Mixing zone** (see Figure 23).That area with a smaller diameter, having the aerator in the center andthe area in which mixing of the liquid allow the solids in suspension.
Oxygenation zone is another type of the influence area of a mechanical aerator. That area has a larger diameter, encircling the mixing zone.The diffusion of oxygen in the liquid is guaranteed, but not the mixing zone. The area of influence of each aerator for oxygenation is much higher than that for mixing.Approximate values for the operating ranges of mechanical aerators as a function of their power presents in Table 16.

Table 16. Usual operation ranges of high-speed aerators.

Power (HP)	Normal operating depth (m)	Influence diameter (m)		Diameter of the anti erosion plate (m)
		Oxygenation	Mixing	
5-10	2.0-3.6	45-50	14-16	2.6-3.4
15-25	3.0-4.3	60-80	19-24	3.4-4.8
30-50	3.8-5.2	85-100	27-32	4.8-6.0

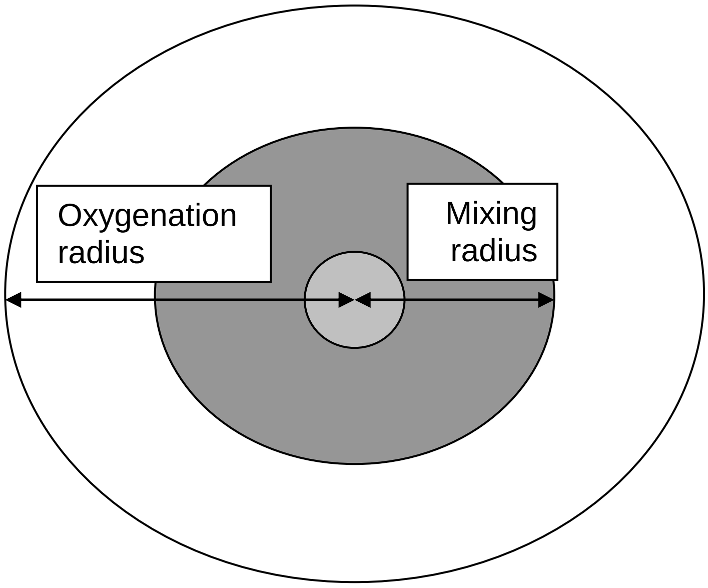


Figure 23. Mixing radius and Oxygenation radius in a mechanical aerator

The oxygenation efficiency OE_{field} is smaller and ranged from 0.55 to 0.65 $OE_{standard}$ in the treatment plant, under field operating conditions.

The power requirements are finally given by the following formula:

$$p = \frac{OR}{24 \times OE_{field}} \qquad (4.47)$$

Where:
P = power required (kW).
24 =conversion from days to hours (24 h/d).
The power of each aerator is specified based on the manufacturers' data (or Table 16). KW needs to be converted into HP (multiply kW by 1.34 to obtain HP).

4-4-4-5 Sludge Accumulation

The sludge should be removed whenusually the sludge reaches 1/3 of the pond depth. The inclusion of grit removal upstream of aerated lagoons is very important.The sludge accumulation rate is 0.03 to 0.08 m^3/inhab .year.

4-4-5 Complete-mix aerated lagoons followed by sedimentation ponds

The aerators distribute the oxygenation in the medium and dispersedthe suspended solids in the liquid medium (see figure 10). So Complete-mix aerated lagoons are essentially aerobic.The effluent from a complete-mix aerated lagoon has high amounts of suspended solids, so it is not enoughfor direct discharge. Complete-mix aerated lagoons must followed by sedimentation ponds, where occur settling and oxidation of the settled solids. The sludge removal is every1 to 5 years by systems with continuous sludge removal, using pumps coupled to rafts. (50)

4-4-5-1 Design Criteria for the Complete-Mix Aerated Lagoons

4-4-5-1-1 Detention time

The hydraulic detention time (t) in the complete-mix aerated lagoons of the liquid is equal to the solids retention time (Θ_c). The hydraulic detention time (t) is the average residence time of the liquid molecules in the pond.The solids retention time, or sludge age(Θ_c) is the average residence time of the bacterial cells in the pond.The molecules of the liquid in complete-mix aerated lagoonsand the bacterial cells remain the same time in the pond($t = \Theta_c$),because it is not occur the sludge recirculation or any form of solids retention.The detention time varies in the range 2 to 4 d.If more than one pond in series is adopted, the detention time in each one is 2 days lead to reduce in the growth of algae.(22)

4-4-5-1-2 Depth

Depths (H) are in the range of 2.5 to 4.0 m.The depth should be selected to satisfy the requirements of the aeration equipment in Complete-Mix Aerated Lagoons for mixing and oxygenation.(22)

4-4-5-2 Evaluation of The Effluent BOD Concentration From The Aerated Lagoon

The effluent from the aerated lagoons is containing of dissolved organic matter (soluble BOD) and suspended organic matter (particulate BOD) (see equation 4.42)

BOD tot = BOD sol + BOD part (4.42)

4-4-5-2-1 Soluble effluent BOD

From the aerated lagoon, thecalculation of the effluent soluble BOD can be done using the same formulas presented for facultative pondsand facultative aerated lagoons, which are a function of the hydraulic regime adopted for the pond.Because of the larger biomass concentration in the pond, the value of the removal coefficient K is higher than in the other ponds systems. The value of K incorporates the effect of the concentration of the volatile suspended solids, which represent the biomass (see equation 4.48). And the typical values of K for a liquid temperature of 20^0Care 1.0 to 1.5 d^{-1}.

" $K = K` . X_v$" (4.48)

Where:

K' = BOD removal coefficient $(mg/L)^{-1}(d)^{-1}$. The value of K' is in the range of 0.01 to 0.03 $(mg/L)^{-1}(d)^{-1}$for a liquid temperature of 20^0C.

X_v = Concentration of volatile suspended solids (mg/L).

K =The removal coefficient.

When K' is constant, the biomass concentration (X_v) is big at the higher the coefficient K and, consequently, the rate of BOD removal efficiency increase. The effluent soluble BOD concentration from the aerated lagoon formula is:

$$S = \frac{S_0}{1 + K^{`} . X_v . t} \quad (4.49)$$

Where: S_o= the influent total (soluble + particulate) BOD.
S = the effluent soluble BOD.
X_v = the biomass concentration.

When S is constant, S_o and the biomass concentration (X_v) increases because of the big amounts from food. The concentration of the biomass (X_v) is a result of the gross growth (positive factor) and the bacterial decay (negative factor).Yield coefficient (Y) represent the amount of biomass (mg X_v) that is produced per unit substrate used (mg BOD_5) and bacterial decay coefficient(K_d) represent the decay rate of the biomass during endogenousmetabolism. The typical values of Y and K_d are given in Table 17.
The calculation of X_v is given by:

$$X_v = \frac{Y.(S_0 - S)}{1+ K_d.t} \quad (4.50)$$

Where: Y =yield coefficient ($mgX_v/mgBOD_5$).
K_d=bacterial decay coefficient or endogenous respiration coefficient (d^{-1}).

Table 17. The typical values of Y and K_d.

Coefficient	Unit	Range	Typical value
Y	mgVSS/mgBOD5	0.4 - 0.8	0.6
K_d	d^{-1}	0.03 - 0.08	0.06

4-4-5-2-2 Particulate effluent BOD

The particulate BOD varies from 0.4 to 0.8 $mgBOD_5/mgVSS$ based on the relationship with the volatile suspended solids. In the effluent from the complete-mix aerated lagoon, it is important to calculate the concentration of suspended solids. That effluent particulate BOD is caused by the suspended solids. In the effluent of the aeratedlagoon, the concentration of volatile suspended solids is given by equation (4.50).The rate between the volatile suspended solids (VSS or X_v) and the total suspended solids (SS or X) is 0.7 to 0.8.Thus, the particulate BOD (BOD_{part}) ranged from 0.3 to 0.6 $mgBOD_5/mgSS$.

From the sedimentation pond, the particulate BOD is a function of the effluent SS in the final effluent. There are no accepted models that allow calculating the effluent concentration.

4-4-5-3 Oxygen amounts in the Aerated Lagoon

The ratio BOD_u/BOD_5 in the raw wastewater is varies from 1.2 to 1.5.For the aerobic oxidation, the amount of oxygen to be suppliedby the aerators of the organic matter should usually be equal to the total ultimate BOD (BOD_u) removed. The calculation of the oxygen amounts by:

$$" OR = \frac{a \,.\, Q \,.(S_0 - S)}{1000} " \qquad (4.51)$$

Where:OR = oxygen requirement (kgO_2/d)
a = coefficient of oxygen consumption (1.1 to 1.4 $kgO_2/kgBOD_5$ removed)
Q = influent flow (m^3/d)
S_o = influent total (soluble +particulate) BOD concentration (g/m^3)
S = effluent soluble BOD concentration (g/m^3)
1000 = conversion of g to kg (g/kg). (22)

4-4-5-4 Power Requirements in the Aerated Lagoon

To keep the suspended solids distributed in the liquid medium, the mixing energy should be fulfilled. The power of the aerators was calculated from equation (4.45).The power level (Φ) should be ≥ 3.0W/m^3 to ensure complete dispersion of the suspended solids in the aerated lagoon. The concepts of Oxygen Requirement (OR) and Oxygenation Efficiency (OE) are used for determine the required power for oxygenation. The required power must comply with both requirements.(22)

4-4-5-5 Design of the Sedimentation Pond

The required volumes for design of the sedimentation pond are volume for sedimentation and volume for the storage and digestion of the sludge. The detention time (t) is ≥1 d and the depth (H) is ≥1.5 m for the volume required for sedimentation. The total volume of the pond, the detention time of the pond (t) is ≤ 2.0 d to avoid algal growth and the depth (H) is ≥ 3.0 m to allow an aerobic layer above the sludge. For the sludge accumulation volume can be calculated from the equation (4.52).

$$" V_t = \frac{\frac{M_V}{K_V} .(1 - e^{-K_V .t}) + t.M_F}{1000. \text{(Dry solids fraction)}} " \qquad (4.52)$$

Where:
V_t = volume of sludge accumulated after a period of t years (m^3)
M_v = mass of volatile suspended solids retained in the pond per unit time (kg VSS/year)
M_F = mass of fixed suspended solids retained in the pond per unit time (Kg SS_F/year)
K_v = decay coefficient of the volatile suspended solids in the sludge in anaerobic conditions (year $^{-1}$). $K_v$varies from 0.4 to 0.6 year $^{-1}$,with an average value of 0.5 year $^{-1}$.
t = time (year)
Dry solids = fraction of dry solids in the sludge = 1 - water content fraction in the sludge.(17)

To calculate V_t adopt the VSS/SS ratio in the influent solids to the settling pond is0.7-0.8 .And the Volatile solids reduction rate in the sludge(K_v) is 0.5 year^{-1} (50% removal per year).

4-5 An Application of modern design methods in Visual Basic program

To design the oxidation ponds by modern design methods, it requires a Visual Basic programming language. The computer program is run in an excel spreadsheet so that the design output data can be analyzed statistically.This program is available for different design conditions.

4-5-1 Monte Carlo simulations for design of the oxidation ponds

The design calculations for anaerobic, facultative and first maturation ponds are achieved by the computer program with a number of simulations. By the computer program using Monte Carlo simulations, new sets of random design values are selected at every run of a simulation within the defined input design range until the final simulation run. The random design values of the hydraulic retention time selected from the minimum range of 3-5 days for design of the second and subsequent maturation ponds in the empirical equation to model faecal coliform removal.The Visual Basic computer program that has been developed is based on the proceduresfor the Monte Carlo simulation presents in Figure 24. (24)

In anaerobic and facultative ponds,the volumetric organic loading rates and BOD removal are determined by the logical part of the computer program. The logical part of the program is based on the application of the random design values of temperature for every run of a simulation. When designing the first maturation pond, the computer program determines the random design value of overall BOD removal in anaerobic and facultative pondsdepending on the selected random temperature. The program uses four temperature conditions are (T) <10^{o}C; between 10 and 20^{o}C; between 20 and 25^{o}C; above 25^{o}C, which must at least be satisfied for every selected random temperature. Thisvalues in a logical selection of volumetric organic loading and BOD removal ratesin anaerobic and facultative ponds. So this enables the design based on random values of the influent BOD for the facultative and first maturation ponds. The computer program selects randomly the temperature, at every run of a simulation, from the proposed range and the selected temperature is compared with the four temperature conditions. (24)

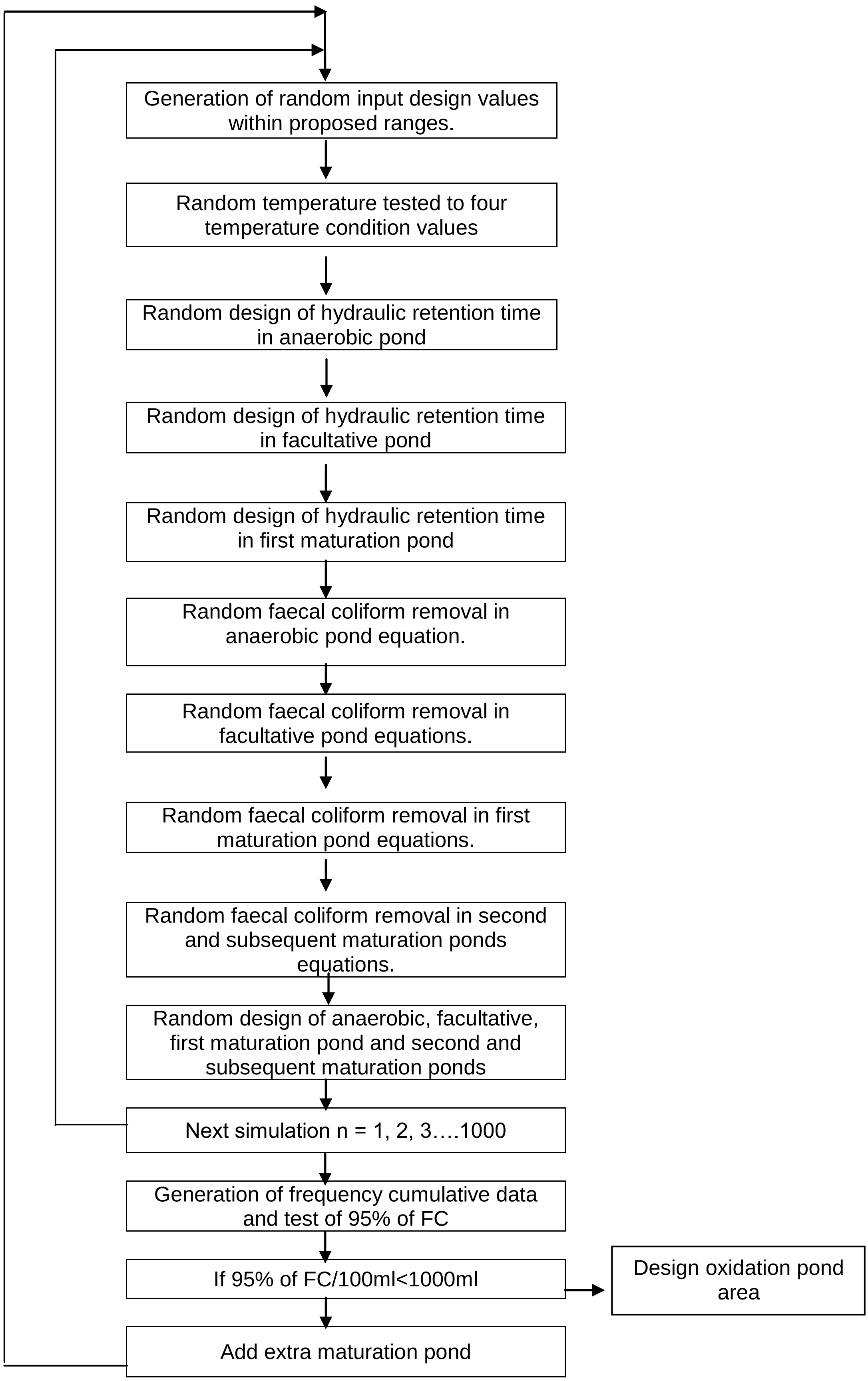


Figure 24: Procedure for the Monte Carlo simulation.

The first temperature condition is satisfied.the computer program calculates equations 4.53 and 4.54,when the selected random temperature (T)<10^oC.The computed random values assigns to these design variables

$\lambda_v = 100$

$(L_i)_f=0.6\ (L_i)_a$ (4.53)

$(L_i)_{m1}=0.3\ (L_i)_a$ (4.54)

Where:

λ_v= volumetric organic loading rate (g/m^3 day)

$(L_i)_a$ = random design value of influent BOD in anaerobic pond (mg/l)

$(L_i)_f$ = influent BOD in facultative pond (mg/l)

$(L_i)_{m1}$= influent BOD into first maturation pond (mg/l)

"The selected random temperature, the volumetric organic loading rate and the random influent BOD concentration in the facultative and first maturation ponds are then used in designing the random hydraulic retention time in the anaerobic, facultative and first maturation ponds".(24)

The second temperature condition is satisfied. the computer program calculates equations 4.55, 4.56, 4.57,when the selected random temperature is between 10 and 20^oC.The computed random values assignsto these design variables.

$\lambda_v = 20T-100$ (4.55)

$(Li)_f = \frac{100-(2T+20)}{100} * (Li)_a$ (4.56)

$(Li)_{ml} = 0.3(Li)_a$ (4.57)

Where:

T and $(Li)_a$ are random design parameters

The selected random temperature, random volumetric organic loading rate and the random influent BOD concentration in the facultative and first maturation ponds are then used todesign the random hydraulic retention time in the anaerobic, facultative and first maturation ponds. (24)

The third temperature condition is satisfied. The computer program calculates equations 4.58, 4.59 and 4.60, when the selected random temperature is between 20 and 25^oC. The computed random values assigns to these design variables:

$\lambda_v = 10T+100$ (4.58)

$(Li)_f = \frac{100-(2T+20)}{100} * (Li)_a$ (4.59)

$(Li)_{ml} = 0.2(Li)_a$ (4.60)

Where:

T and $(Li)_a$ are random design parameters

The selected random temperature, random volumetric organic loading rate and the random influent BOD concentration in the facultative and first maturation ponds are then used in designing the random hydraulic retention time in the anaerobic, facultative and first maturation ponds.

The fourth temperature condition is satisfied. The computer program calculates equations 4.61 and 4.62, when the selected random temperature is above 25^oC.The computed random values assigns to these design variables:

$\lambda_v = 350$
$(Li)_f = 0.3\ (Li)_a$ (4.61)
$(Li)_{ml} = 0.2\ (Li)_a$ (4.62)

Where:
$(Li)_a$ is a random design parameter.
The selected random temperature, the volumetric organic loading rate and the random influent BOD concentration in the facultative and first maturation ponds are then used in designing the random hydraulic retention time in the anaerobic, facultative and first maturation ponds. (24)

4-5-1-1 Random design computation of the hydraulic retention time in anaerobic ponds

In anaerobic pond, Monte Carlo simulation is used by the computer program for designing the random hydraulic retention time for every run of a simulation. In equation 4.63 calculates the random design of the hydraulic retention time is givenby:

$$\theta_a = \frac{(L_i)_a}{\lambda_v} \quad (4.63)$$

Where:
θ_a = hydraulic retention time (days)
$(L_i)_a$ = influent BOD concentration in anaerobic pond (mg/l)
λ_v = volumetric BOD loading (g/m^3.day)
From the proposed uniform distribution range,the influent BOD concentration $(L_i)_a$, is randomly selected. The volumetric organic loading rate (λ_v) is determined from any of the four defined temperature conditions which satisfy the selected random temperature. For the random design of the hydraulic retention time (θ_a), a new set of the random values of the influent BOD concentration $(L_i)_a$and volumetric organic loading (λ_v) are selected to run the next simulation. The computer program compares between the computed random design values of the hydraulic retention time (θ_a) with the minimum hydraulic retention time of 1 day. If the calculated random retention time is less than 1 day, the computer program assigns a value of 1 day as the random hydraulic retention time for that simulation. (24)

4-5-1-2 Random design computation of the hydraulic retention time in facultative pond

In the facultative pond, the developed program uses Monte Carlo simulations to calculate the random design value of the hydraulic retention time. The computer program chooses randomly the input design parameters of temperature, design flow, net evaporation rate from their proposed uniform distribution range. In equations 4.64, 4.65 and 4.66 calculate the random design value of the hydraulic retention time isgiven by:

$\lambda_{sf} = 350(1.107-0.002T)^{T-25}$ (4.64)
$A_f = (10(L_i)_f\ Q_f)/\ \lambda_{sf}$ (4.65)

$\theta_f = (2\ A_f\ H_f)/\ (2\ Q_f - 0.001e\ A_f)$ (4.66)

Where:
f =The subscript refers to facultative pond
The parameters T, $(L_i)_f$, Q_f, λ_{sf}, A_f and e are random design values
θ_f= hydraulic retention time in facultative pond (days)
λ_{sf}= surface BOD loading (kg/ha day)
T = temperature (^oC)
A_f= facultative pond area (m^2)
$(L_i)_f$= influent BOD concentration in the facultative pond (mg/l)
Q_f= mean flow (m^3/day)
H_f = pond depth (m)
e = net evaporation (mm/day)

The computed random design value of the hydraulic retention time (θ_f)is compared with the recommended minimum hydraulic retention facultative pond and it is 4 days for the facultative pond. If the computed random retention time is less than 4 days, the computer program assigns a random design value of 4 days as the random hydraulic retention time for that simulation. Newsets of the random design value of temperature T, design flow Q_f, net evaporation e, and influent BOD concentration $(L_i)_f$ are selected from their proposed uniform distribution range to run the next simulation and to compute the next random design value of the hydraulic retention time.(24)

4-5-1-3 Random design value of the hydraulic retention time in first maturation ponds

In first maturation ponds,the developed program uses Monte Carlo simulations to calculate the random design value of the hydraulic retention time. The random design value of the influent BOD concentration is calculated from the four defined temperature conditions andthe random design hydraulic retention time of the first maturation pond is carried out by using the design equations .(24)

In equations 4.64 and 4.67 presented the random design value of the hydraulic retention time in the first maturation pond is given by:
$\theta_{m1} = (10(L_i)_{m1}.H_{m1})/\ (0.75\ \lambda_{sf})$ (4.67)

Where:
θ_{m1} = The subscript refers to first maturation pond.
The parameters T, $(L_i)_{m1}$ and λ_{sf}, are random design values
θ_{m1} = minimum hydraulic retention time in first maturation pond (days)
H_{m1}= design depth of the first maturation pond (m)
$(L_i)_{m1}$ = influent BOD concentration in first maturation pond (mg/l)
λ_{sf}= surface BOD loading in facultative pond (kg/ha day)
The computed random design value of the hydraulic retention time (θ_{m1}) is compared with the minimum hydraulic retention time of 3 days. If the computed

random value is less than 3 daysin the first maturation pond, the computer program assigns a value of 3 days as the random hydraulic retention time. Infirst maturationpond; the computed random value of the hydraulic retention time is compared with the hydraulic retention time in the facultative pond. The computer program assigns the computed random value of the first maturation pond as that of the facultative pond, if the random design value of the hydraulic retention time in the first maturation pond is more than the computed value of the hydraulic retention time in the facultative pond. To run the next simulation, new sets of the random design values of temperature (T) and influent BOD concentration $(L_i)_{m1}$, are selected from the proposed uniform distribution range.These new selected sets of random design values are used to calculate the next random design of the hydraulic retention time (θ_{m1}) in the first maturation pond. (24)

4-5-1-4 Random design value of the hydraulic retention time in the second and subsequent maturation ponds

The random design values in the second and subsequent maturation ponds are selected from the minimum hydraulic retention time range of 3-5 days.(53)
In the second and subsequent maturation ponds, the developed program uses Monte Carlo simulations to calculate the random design value of the hydraulic retention time. These random values of the hydraulic retention timeare used in the empirical equation of Von Sperling for modeling faecal coliform removal. (24)
To run the next simulation, the computer program selects a new set of random design values for the hydraulic retention time used to model faecal coliform removal in the second and subsequent maturation ponds. (24)

4-5-1-5 Random design value of the effluent in anaerobic ponds

In anaerobic ponds, Monte Carlo simulation is used by the computer programto compute the random design value of the effluent faecal coliform concentration. The computer program selects randomly the input design variables of temperature T, influent faecal coliform concentration $(N_i)_a$, the first-order rate constant for faecal coliform removal, and the temperature coefficient of faecal coliform removalK_{FCT} ,φ, from their proposed uniform distribution range.

The random design values of the hydraulic retention time θ_a, and the influent BOD concentration $(L_i)_a$ is determined from equation (4.63) .In anaerobic ponds, the computer program determines the random design value of the effluent faecal coliform concentration by employing the empirical equation, presented as follows:

$$(N_e)_a = (N_i)_a / (1 + K_{FCT}\, \varphi^{(T-20)}\, \theta_a) \qquad (4.68)$$

Where:
a =The subscript refers to anaerobic pond.
$(N_e)_a$ = effluent faecal coliform concentration (per 100 ml)
N_i = influent faecal coliform concentration (per 100 ml)

K_{FCT}= first order rate constant for faecal coliform removal (day^{-1})
Φ= temperature coefficient of faecal coliform removal
θ_a= hydraulic retention time (days)
T = air temperature (^oC)
The next run of the simulation selects new sets of random design values of temperature T , influent faecal coliform concentration $(N_i)_a$, the first-order rate constant for faecal coliform removal K_{FCT} , temperature coefficient of faecal coliform removal φ and the hydraulic retention time θ_a , from the uniform distribution range. (24)

4-5-1-6 Random design computation of the effluent in facultative pond

The computer program uses Monte Carlo simulations in facultative pond to compute the random design value of the effluent faecal coliform concentration. The program selects the input design variables randomly of temperature T, influent faecal coliform concentration $(N_i)_f$,the temperature coefficient of faecal coliform removal φ and dispersion numbers d_f ,from their proposed uniform distribution range. The random design value of the hydraulic retention time θ_f , is determined from equation (3.74) .In equations 4.69, 4.70, 4.71, 4.72 and 4.73 presented the random design value of the effluent faecal coliform concentration bythe computer program from the empirical equations is given by:

$d_f = 1/ (L/W)_f$ (4.69)

$K_{FC20f} = 0.917. (H_f)^{-0.877}. (\theta_f)^{-0.329}$ (4.70)

$K_{FCTf} = K_{FC20f} . \varphi^{(T-20)}$ (4.71)

$(N_e)_f = (N_e)_a * [(4a_f e^{(1/2d f)})/ (1+ a_f)^2 . e^{a f /2d f} – (1 - a_f)^2 . e^{-a f /2d f})]$ (4.72)

$a_f = (1+ 4 K_{FCT f} . \theta_f . d_f)^{0.5}$ (4.73)

Where:
f = The subscript refers to facultative pond
$(N_e)_f$ = effluent faecal coliform concentration (per 100 ml)
$(N_e)_a$= influent faecal coliform concentration (per 100 ml)
K_{FCTf}= faecal coliform die-off rate at temperature T ^oC
K_{FC20f} = faecal coliform die-off rate at 20^oC
H_f= pond depth (m)
Φ= temperature coefficient for faecal coliform removal = 1.07
d_f= dispersion numbers
L = pond length (m)
W = pond breath (m)
θ_f= hydraulic retention time (days)
T = air temperature (^oC)

The next run of the simulation selectsnew sets of the random design values of temperature T, influent faecal coliform concentration $(N_e)_a$, temperature coefficient of faecal coliform removal φ , dispersion numbers d_f, and the hydraulic retention time θ_f , from their proposed uniform distribution range. (24)

4-5-1-7 Random design value of the effluent in first maturation pond

In the first maturation pond, the computer program uses Monte Carlo simulations to compute the random design value of the effluent faecal coliform concentration. The computer program selects randomly the input design variables of temperature T, influent faecal coliform concentration $(N_e)_{f,}$ temperature coefficient of faecal coliform removal φ, and dispersion numbers $d_{m1,}$ from the proposed uniform distribution range and the random design value of the hydraulic retention time (θ_{m1}) is determined from equation (4.67). (24)
To calculate the random design value of the effluent faecal coliform concentration in the first maturation pond, the computer program uses the empirical equations 4.74, 4.75, 4.76, 4.77 and 4.78 are as follows:

$$d_{m1} = 1/\ (L/W)_{m1} \qquad (4.74)$$

$$K_{FC20m1} = 0.917.\ (H_{m1})^{-0.877}.\ (\theta_{m1})^{-0.329} \qquad (4.75)$$

$$K_{FCTm1} = K_{FC20\ m1}.\ \varphi^{(T-20)} \qquad (4.76)$$

$$(N_e)_{m1} = (N_e)_f * [(4a_{m1} e^{(1/2dm1)}) / (1+a_{m1})^2.\ e^{a\ m1\ /2d\ m1} - (1-a_{m1})^2\ .e^{-a\ m1\ /2d\ m1})] \quad (4.77)$$

$$a_{m1} = (1+ 4\ K_{FCTm1}\ .\ \theta_{m1}\ .\ d_{m1}\ )^{0.5} \qquad (4.78)$$

Where:
m1=The subscript refers to first maturation pond
$(N_e)_{m1}$ = effluent faecal coliform concentration (per 100 ml)
$(N_e)_f$ = influent faecal coliform concentration (per 100 ml)
K_{FCTm1} = faecal coliform die-off rate at temperature T ^oC
$K_{FC20\ m1}$ = faecal coliform die-off rate at 20^oC
H_{m1} = pond depth (m)
Φ= temperature coefficient of faecal coliform removal = 1.07
d_{m1} = dispersion numbers
L = pond length (m)
W = pond breath (m)
θ_{m1}= hydraulic retention time (days)
T = air pond temperature (^oC)

For the next run of the simulation, the computer program selects new sets of the random design values of temperature T, influent faecal coliform concentration $(N_e)_f$, temperature coefficient of faecal coliform removal φ, dispersion numbers d_{m1}, and hydraulic retention time θ_{m1}, from the proposed uniform distribution range. (24)

4-5-1-8 Design of the number of second and subsequent maturation ponds

Design of the number of second and subsequent maturation ponds are based on the 95 % of effluent faecal coliform concentration. The effluent faecal coliform concentration is <1000FC per 100ml.The computer program selects "one" second maturation pond at a time when carrying out the Monte Carlo simulations. The design calculations are repeated with this "one" second maturation pond, using new sets of input design parameters selected from the proposed uniform distribution range. The design calculations are repeated until the final (1000th) simulation has been completed. (24)

The computer program is designed to prepare the frequency cumulative data of the random design values of the effluent faecal coliform concentration obtained from the total number of simulations (1000). The 95% value of effluent faecal coliform is selected from the frequency cumulative data and is compared with the standard faecal coliform concentration of 1000 FC per 100ml for unrestricted crop irrigation. If the selected 95% value of the effluent faecal coliform concentration is more than 1000 FC per100ml, the computer program adds one maturation pond to the second maturation pond. The design calculations are then repeated with these "two" second maturation ponds until the number of required simulations (1000) are run. The computer program once again compares the selected 95% value of effluent faecal coliform concentration with the standard effluent faecal coliform concentration of 1000 FC per 100ml. If the selected 95% value of FC is less than 1000 FC per 100ml,the computer program provides the design solution for the number of second and subsequent maturation ponds. (24)

4-5-1-9 Random design value for the areas of the ponds

When the 95% value of the effluent faecal coliforms concentration in the last maturation pond is less than 1000 FC per 100ml, the design of oxidation ponds area is completed. The computer program uses Monte Carlo simulations to compute the random design area of the anaerobic pond, facultative pond, first maturation pond and the second and subsequent maturation ponds. The computer program determines randomly the design values of flow rate (Q_i) and net evaporation rate (e) from their proposed uniform distribution range.
When designing oxidation ponds area, the computer program uses the computed random values of the hydraulic retention times in anaerobic, facultative and maturation ponds. The depths of the oxidation ponds are treated as constant values. And the program determines the random design area of the anaerobic pond as shown in equation (4.79):

$$A_a = \theta_a Q_i / H_a \qquad (4.79)$$

Where:
a =The subscript refers to anaerobic pond
The variables A_a, θ_a, Q_i are random design values
A_a= area of anaerobic pond (m^2)
θ_a= hydraulic retention time of anaerobic pond (days)
Q_i= influent flow (m^3/day)
H_a= pond depth of anaerobic pond (m)
The computer program uses Monte Carlo simulations to compute the random design area of the facultative pond as shown in equation (4.80):

$$A_f = 2\, \theta_f Q_i \, / \, (2\, H_f + 0.001\, e\theta_f) \qquad (4.80)$$

Where:
f =The subscript refers to facultative pond
The variables A_f, θ_f, Qi and e are random design values
A_f= area of facultative pond (m^2)

θ_f = hydraulic retention time of facultative pond (days)
Q_i = influent flow (m^3/day)
H_f= pond depth of facultative pond (m)
e = net evaporation rate (mm/day)

Monte Carlo simulation is used by the program to compute the random design area of the first maturation pond as shown in equations 4.81 and 4.82:

$$(Q_e)_f = Q_i - 0.001eA_f \quad (4.81)$$
$$A_{m1} = 2\,\theta_{m1}\,(Q_e)_f \;/\; (2\,H_{m1} + 0.001\,e\theta_{m1}) \quad (4.82)$$

Where:
m1=The subscript refers to first maturation pond
f =The subscript refers to facultative pond
The variables A_{m1}, θ_{m1}, $(Q_e)_f$ and e are random design values
A_{m1} = area of first maturation pond (m^2)
θ_{m1}= hydraulic retention time of first maturation pond (days)
$(Q_e)_f$ = influent flow rate into first maturation pond (m^3/day)
H_{m1}= pond depth of first maturation pond (m)
e = net evaporation rate (mm/day)
The computer program uses Monte Carlo simulations to compute the random design area of the second and subsequent maturation ponds as shown in equations 4.83 and 4.84:

$$(Q_e)_{m1} = (Q_e)_f - 0.001eA_{m1} \quad (4.83)$$
$$A_m = (2\,\theta_m\,(Q_e)_{m1}) / (2\,H_m + 0.001\,e\theta_m) \quad (4.84)$$

Where:
m=The subscriptrefers to second and subsequent maturation pond.
m1=The subscriptrefers to first maturation pond.
f =The subscript refers to facultative pond.
The variables A_m, θ_m, $(Q_e)_{m1}$ and e are random design values;
A_m= area of second and subsequent maturation ponds (m^2)
θ_m= hydraulic retention time of second and subsequent maturation ponds (days)
$(Q_e)_{m1}$= influent flow rate into second maturation pond (m^3/day)
H_m = pond depth of second and subsequent maturation ponds (m)
e = net evaporation rate (mm/day)

4-5-1-10 Analysis of the output design data

The computer program presents the designs of oxidation pondsarea, hydraulic retention time and effluent faecal coliform concentration in 50% and 95% form.
The output design data of oxidation pondsis calculated by Monte Carlo simulations are presented in statistical form. Frequency cumulative curves and histograms are generated to present the distribution of the effluent faecal coliform concentration. So the designer is able to choose the acceptable percentile value to forms the basis for designing the oxidation pondsarea. The

choice of the percentile value depends onthe cost and the uses of the final treatedeffluent. A lower percentile value (50%) of the effluent faecal coliform concentration can be chosen, if the available cost to design the oxidation ponds is not adequate. (24)

Example 3

Design the oxidation ponds system to treat domestic wastewater that has typical characteristics of that found in developing countries and the treated effluent is to be used for unrestricted crop irrigation?

The average deterministic single values of the input design parameters are allowed to vary by ± 20% to establish the range of their variation, while some input design parameters such as pond depth are kept constant.

The range of the input design parameters is presented as follows:
Per capita BOD contribution (g/person. day) = (32, 48)
Per capita wastewater production (l/person day) = (96, 144)
Design population = (80000, 120000)
Net evaporation (mm/day) = (3.2, 4.8)
Depth of anaerobic pond (m) = (4.0, 4.0)
Depth of facultative pond (m) = (1.5, 1.5)
Depth of first maturation pond (m) = (1.0, 1.0)
Depth of second and subsequent maturation ponds (m) = (1.0, 1.0)
Number of influent faecal coliform concentration (per 100 ml) = ($7x10^7$, $1.2x10^8$)
Temperature (^oC) = (16, 24)
Temperature coefficient of faecal coliform removal = (0.856, 1.284)
First-order faecal coliform removal constant rate in anaerobic pond (day^{-1}) = (1.6, 2.4)
Dispersion numbers in facultative pond $1/(L/B)_f$ = (0.08, 0.12)
Dispersion numbers in first maturation pond$1/(L/B)_{m1}$= (0.04, 0.06)
Dispersion numbers in 2^{nd} and subsequent maturation ponds$1/(L/B)_m$ =(0.04, 0.06)
Minimum hydraulic retention time in 2^{nd} and subsequent maturation ponds (days) = (3, 5)
Minimum hydraulic retention time anaerobic pond (days) = (1)
Minimum hydraulic retention time in facultative pond (days) = (4)
Minimum hydraulic retention time in maturation pond (days) = (3)
Number of simulations = 1000

Solution

The Visual Basic program was run with 1000 simulations using the range of the input design parameters shown above andthe results of the output design data are shown in these tables. (24)

Table 18.Effluent fecal coliform concentration from Monte Carlo simulations (1000 runs).

Statistical parameter	Number of faecal coliforms per 100 ml of effluent
Mean	260
Minimum	0
Maximum	51,000
50% value	2
95% value	780

Table 19. Area of oxidation ponds from Monte Carlo simulations (1000 runs) (m^2)

Statistical parameter	Anaerobic	Facultative	1st Maturation	Subsequent maturation ponds; no. of maturation ponds = 6
Mean	3,647.20	68,493.83	55,865.74	47,502.15
Minimum	1,944.09	25,470.52	22,611.80	22,462.24
Maximum	6,326.91	145,044.06	117,279.76	80,283.23
50% value	3,553.48	63,807.82	49,635.81	46,365.51
95% value	5,345.29	118,253.87	98,280.69	69,407.90

Table 20. Hydraulic retention time of oxidation ponds from Monte Carlo simulations (1000 runs) (days)

Statistical parameter	Anaerobic	Facultative	1st Maturation	Subsequent maturation ponds; no . of maturation ponds = 6
Mean	1.16	8.28	4.6	3.98
Minimum	1	4.87	3	3
Maximum	1.51	13.39	7.3	5
50% value	1.1	7.81	3.47	4
95% value	1.45	12.56	7.01	4.88

In Table 18,The design output of the effluent faecal coliform concentration shown that a series of anaerobic pond, facultative pond and seven maturation ponds .The ponds achieve the design criteria based on a 95% ; FC = 782 per 100 ml , less than 1000 per 100 ml for unrestricted crop irrigation.

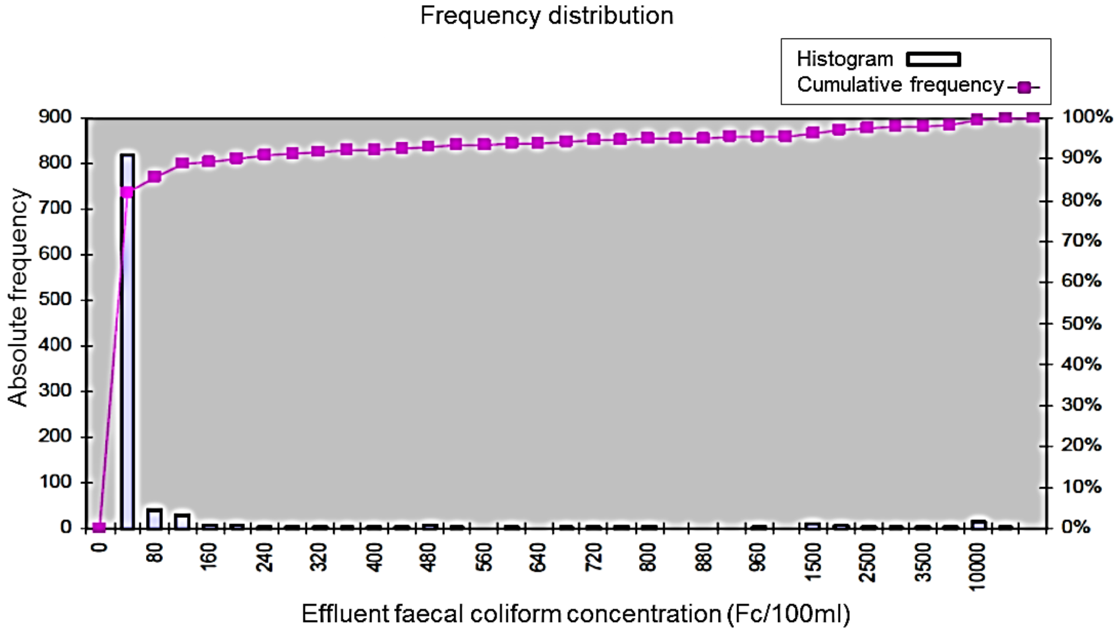


Figure 25: Frequency cumulative curve and histogram of the effluent FC from Monte Carlo simulations (1000 runs).

The design output of the effluent faecal coliform concentration presented as a frequency cumulative curve in Figure 25 and a range of 20 - 60 FC per 100 ml is the highest predicted frequency (818) of the effluent faecal coliform concentration presented in the histogram in Figure 25.From Table 18, the 95% of the effluent faecal coliform concentration provided by the computer program as 782 FC per 100ml and it confirmed from the cumulative frequency curve. To design the area of oxidation ponds depending on the available cost and the acceptable health risk, the designer select any required percentile value from the graph.(24)

The effluent faecal coliform concentration 95% of the 1000 calculated values would be less than 1000 FC per 100ml, assuming that the average values of the input design parameters vary by ±20%. Presented in Tables 19 and 20, the design output of oxidation ponds area and hydraulic retention time. The oxidation pondswould have a 95% probability that the effluent faecal coliform concentration would be less than 1000 FC per 100ml, If designed by **Monte Carlo simulations.**(24)

Chapter 5
Summary of the different design approaches and proposed design methods under Egyptian conditions

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Table 21. Summary of the Design Criteria of the oxidation ponds in Egypt and world wide

Design Criteria	Egypt	Sweden	United States	Brazil
Air Temperature(^{0}C)	**> 20**	**Below zero**	**< 10**	**10-25**
1- Anaerobic ponds				
Depth (m)	2.5 - 5 m	3-5 m	2.5 - 5 m	3-5 m
Retention time (day)	3-5	1-3	20-50	3-6
Rate of removal BOD_5	50-70%	50-70%	75 %	______
Rate of length/ Breadth	2:1-3:1	3:1	______	1 to 3
Rate of the organic load (Kg BOD_5 / m^3 . D)	0.125-0.3	0.1-0.4	0.32	0.1-0.35
2- Facultative ponds				
Depth (m)	1.5 - 2 m	3 m	1.2-2.5 m	1.5 – 2.0
Retention time (day)	According to the method of the design	20-150	5-30	15 - 45
Rate of removal BOD_5	According to the method of the design	70-80%	______	______
Rate of Breadth / length	2:1-3:1	< 2:1	______	2 to 4
Rate of the organic load(Kg BOD_5/ m^3.D)	300 - 200	______	______	______

Design Criteria	Egypt	Sweden	United States	Brazil
3- Maturation ponds				
Depth (m)	1 m	0.9-1 m	1-1.5 m	0.8-1.2
Retention time (day)	3-10	3 days in the coldest temperature , 4-5 days , 20 - 30 day	4-15	a function of the pond shape and the required coliform removal efficiency
Rate of removal BOD5	———	70%	———	———
Rate of Breadth / length	———	———	———	1 to 3
Rate of the organic load(Kg BOD 5 / ha . D)	———	———	17	———
4- Facultative aerated lagoons				
Depth (m)	———	2.5-4 m	———	2.5-4 m
Retention time (day)	———	5 -10	———	5-10
Rate of removal BOD5	———	———	———	———
Rate of Breadth / length	———	———	———	2 to 4
Rate of the organic load(Kg BOD 5 / ha . D)	———	———	———	———
5- Complete - mix aerated lagoons				
Depth (m)	———	2.5-4m	———	2.5-4
Retention time (day)	———	2-4	———	2-4
Rate of removal BOD5	———	———	———	———
Rate of Breadth / length	———	———	———	1 to 2
Rate of the organic load(Kg BOD 5 / ha . D)	———	———	———	———

Design Criteria	Egypt	Sweden	United States	Brazil
6- Sedimentation ponds				
Depth (m)	———	≥1.5 m	———	3-4
Retention time (day)	———	2-4	———	~ 2
Rate of removal BOD_5	———	———	———	———
Rate of Breadth /length	———	———	———	———
Rate of the organic load (Kg BOD_5 / ha . D)	———	———	———	———

When compare between Design Criteria of the oxidation ponds in Egypt and Sweden, United States and Brazil, it appears the weather in Sweden and United States is very cold. But the weather in Brazil and Egypt is the same. So that I am recommend application **the different design methods from Brazil** in Egypt.

Chapter 6
Application of Brazilian design methods under Egyptian conditions

Chapter 6
Application of Brazilian design methods under Egyptian conditions

In this chapter presents case study and design report about “the waste water treatment plant - Jaradah - town of El Arish” .I discuss the problems on these plants and applied Brazilian design methods to solve this problems and determine the best alternative from them.

6-1 The waste water treatment plant - Jaradah - town of El Arish

٦-١-1 Introduction

Treatment plant was established for the city of Arish capacity design 50.000 m^3 / day in the south east of the city (See Appendix 1).
It begins to construct in 1988 and entered to the service in 1998 and the plant operates by system of oxidation ponds and it can summarized data of plant as follows in table (22). (16)

Table 22. Information about the waste water treatment plant in- Jaradah - town of El Arish

Treatment plant name	**Jaradah**
Treatment plant site	**road of the Rafah Arish**
Treatment plant dimensions	**1500m * 530 m**
Design capacity	**50,000 m^3 / day**
Type of treatment	**using oxidation ponds**
The operator	**International Company**
Sewage outlet	**Tree forest of 200 acres**
Service areas	**El Arish town except the section of Masaeed and Land associations**

6-1-2 Plant components

Jaradah treatment plant operated by oxidation ponds and the contents of the plant are as follows: (see table 22)

Inlet Chamber and Manual Screen;
Measuring flow device and screens;
Mixing chlorinated ponds;

Measuring flow device at the entrances to the mixing chlorinated ponds.
Basins of sludge drying at 2 m depth;
Places storing sludge;
Treatment water pumping plant for agriculture.

Building of chlorine and contains of:
*(10) Chlorine cylinders of volume1 ton;
*Electric Crane 2 ton payload by car Almonrl loads 3 tons;
*Two 2 chlorine regulator;
*Two (2) remote control manual and an automatic capacity of 10 kg / h;
*Device to measure the proportion of residual chlorine (zero-5) part / million;
*Device to measure the proportion of chlorine leaking;
*Two (2) controllers in residual chlorine;
*Two (2) of the sample pump;
*Four (4) a warning against leaking chlorine;
*Masks chlorine;
*Sex (6) aeration fans in the building;
*Load balance (2000) kg;
*Pumping unit soda disposal of 12 m^3 / h lifter 30 meters (number 2 pump);
*Tower balance;
*Electrical control panel and distribution panel and low pressure plate number (2) cell;
*Four (4) Danish pump brand "Danfoss".

Factory and Administration Building;
Umbrella cars;
Generate electricity building (generator capacity of 1340 KVA Czech);
Gate Building, control and guard room;
Electrical transformers building;
Electrical Panel feeding building.

Table 23. Number of all type of the ponds in- Jaradah - town of El Arish

Type of ponds	Number
Anaerobic ponds	**2**
Facultative ponds	**2**
Maturation ponds	**4**

6-1-3 Quantities of the received flow

The plant receiving water from the main pump station in Arish by 2 force main with diameter 600 mm to the treatment plant and describes measurements made during field visits amount of water that reaches the plant 38,000 m^3 / day and constitute this amount 76% of the capacity design of the plant and data show the actual operating of the plant. It works intermittent where the water up on payments, which could adversely affect the efficiency of the treatment plant and the attached table

(23), shows the extension measurements of the amount of water coming into the plant. (16)

6-1-4 Processing units

Waste water treatment plant in town of El Arish depends on the natural oxidation ponds, one of the inexpensive and techniques need small amounts of electrical energy and labor isn't highly trained. The oxidation ponds are frequently used in Sinai due to the availability of desert land and the lack of skilled manpower and treatment plant consists of three types of series ponds as follows:

6-1-4-1Anaerobic ponds

No- of ponds = 2
Ponds length =233 m
Ponds wide= 83 m
Water depth =3 m

6-1-4-2 Facultative ponds

No- of ponds =2
Ponds length= 532 m
Ponds wide= 233 m
Water depth = 2 m

6-1-4-3 Maturation ponds

No- of ponds= 4
Ponds length =356.6 m
Ponds wide= 27.5 m
Water depth =2 m

6-1-5 Factory and laboratory tests

There is a laboratory in the plant. Laboratory tests achieved to know the specifications for the influent of wastewater and the final effluent. (16)

6-1-6 Hydraulic calculations

Evaluation of treatment stages
Number of anaerobic ponds = 2
The volume of the 1anaerobic pond = 58,017 m^3
Area of 1 pond = 19339 m^2
The volume of the anaerobic ponds = 116,034 m^3
Area total = 38678 m^2
Retention Time in the anaerobic ponds = 2.3 days (3 - 5 days according to the Egyptian code)
Number of facultative ponds = 2
The volume of 1 pond = 247,912 m^3
Area of 1 pond = 123956 m^2
The volume of the total facultative ponds = 495,824 m^3
Area facultative ponds = 247,912 m 2 = 24.8 hectares

Retention Time in the facultative ponds = 9.9 day (5-10 day according to the Egyptian code)

Surface loading rate in facultative ponds = $\frac{50000 \times 192}{24.8}$

= 38.7 kg / ha / day

The volume of the one maturation pond = 39,226 m^3 each stage of maturation ponds, which account for 2 on series

The total volume maturation ponds = 78,452 m^3

Area of 1 pond = 9806.5 m^2

Area total = 39226 m^2

Retention Time in the Maturation ponds = 15.7 days (3 - 10 days according to the Egyptian code). (2;16)

It is clear that all the design values less than the design values in Egyptian Code. Therefore the plant can not contain the values more than the design values. (16)

6-1-7 laboratory Analyses for wastewater

Due to analysis the wastewater in the laboratory leads to know specifications for the influent of wastewater and the final effluent.

-Specification the influent wastewater to the plant.

BOD_5 = 192 mg / L

TSS = 1236 mg / L

-Specifications final effluent water

BOD_5 = 36 mg / L

TSS = 1476 mg / L. (16)

6-1-8 Efficiency treatment in the plant

Adopting an accumulation rate of Sludge in the anaerobic pond is 0.04 m^3/inhab.year

Annual accumulation of sludge = .04*347223=13889 m^3/year

Thickness = $\frac{\text{Annual accumulation x time}}{\text{Pond area}}$ (22)

Thickness = 13889 *1 / 38678 = 0.36 cm/year

Time to reach 1/3 of the pond depth:

Time = $\frac{H/3}{\text{Yearly thickness}}$

Time = 3/3 / 0.36 = 2.8 year

According to the equations (5.1) and (5.2)

$$\eta_{BOD} = \frac{B1 - B2}{B1} = 81\% \qquad (٥,١)$$

$$\eta_{TSS} = \frac{S1 - S2}{S1} = -19\% \qquad (5.2)$$

Where:

B1 = Influent BOD
B2 = effluent BOD
S1 = Influent TSS
S2 = Influent TSS

It is expected that due to the occurrence of the retention time less than minimum retention time which permitted in Egyptian CODE. It could lead to big anaerobic reactions in facultative ponds will leading to the escape of solids from these ponds and non-completion of interaction within the maturation ponds leading to increased solids as that the plant is not been clean and sludge remove since the beginning of the operation, leading to the accumulation of sludge in the ponds and exit at intervals with the final effluent.(16)

6-1-9 The final evaluation of the plant (Hydraulic evaluation)

The plant operates efficiently is satisfactory in terms of biological treatment. There is sufficient ground in plant to the second phase expansion process similar to the current stage.

6-1-10 My Notes

The retention time is an important parameter in design the oxidation ponds. So there is a big problem resulted from the retention time because the retention time less than minimum retention time which permitted in Egyptian Code. These problem effects on the efficiency in the design and the operation process at the pond. Neglect taking into account future expansions of the design that effect on the performance of oxidation pond and lead to a lack of efficiency of the plant.

In design “Maturation ponds” the important factor must be taken into account in the design is the disappearance of bacteria and the production of good effluent is the retention time, control in-depth to prevent the conditions that lead to the growth of mosquitoes.

In the Egyptian Code edition 1997 completely similar to the edition 2008 in the values and methods and equations and this is evidence not keep up with the progress and non-application of different methods in the wastewater treatment plants and other the world methods that applies in other countries. Whereon these different methods achieve a highly efficient in the wastewater treatment and economical and do not represent a workload on the state.

* I apply the different design methods from Brazil in “the waste water treatment plant - Jaradah - town of El Arish” and determine the best alternative from them.

Alternative 1

Design a treatment system composed of primary facultative ponds for the waste water treatment plant - Jaradah - town of El Arish in Egypt.
Given the following data from the report:
Population = 347,223 inhabitant
Influent flow: Q = 50,000 m^3/d
Influent BOD: S_o = 192 mg/L , effluent BOD= 36 mg/L
Influent TSS = 1236 mg/L , effluent TSS= 1476 mg/L
Temperature: T = 23°C

Solution

1- Inlet Channel
Q max = 0.94 m3/s
Assume depth = 0.35m , velocity = 0.6m/s
Area = Q / V = 0.94 / .6 = 1.57 m^2
B (net) = A / D inlet channel
B (net) = 1.57/0.35 = 5 m, L = 2*5=10 m
Check on Velocity:
V = Qmax / A*D = 0.54 m/s
from manning Equation :
Q max = 1/n *(R) $^{2/3}$*(S) $^{0.5}$, n = .015, R = L*B / (L+2B)=2.64
get S = .0001

2-Screen:
Qmax=0.94m^3/s , assume V = 1m/s , spacing S = 0.03
Area = Qmax/V = 0.94m^2
get B = A/D inlet channel = 0.94/0.35 = 2.68 m
No of opening = B / S = 2.68 / 0.03 = 89
W (width) = No of opening * B + (Noof opening + 1) $^{0.5}$ * S
W = 4.04 m
Check on Velocity

Vact = V * sin θ
Taken θ = 35
V act = 1 * sin 35
V act = 0.57 m/s (≤ 0.6 m/s) safe
Use two Screen with width = W/2 = 2.02 m (And one standby)

a) Calculation of the influent BOD_5 load for the Primary facultative pond.
Load = concentration x flow = $\frac{192\ g/m^3\ .50{,}000\ m^3\ /d}{1000\ g/kg}$ = 9,600 kg/ d

b) Adoption of the surface loading rate from figure 14 at temperature 23^0c

Ls = 291 kg BOD/ha.d

c) Calculation of the required area.

$$A_{net} = L / Ls = \frac{9{,}600 \text{ kg/d}}{291 \text{ kg/ha.d}} = 35 \text{ ha} = 350{,}000 m^2$$

d) Adoption of a value for the pond depth.
H = 2.0 m (adopted)
e) Calculation of the resulting volume.
$V = A.H = 350000m^2 \times 2.0m = 700000\ m^3$
f) Calculation of the resulting detention time

$$t = \frac{V}{Q} = \frac{700000\ m^3}{50{,}000\ m^3/d} = 15\ d$$ (OK range 15-45 days)

f) Dimensions of the pond.
Adopt 2 ponds.
Area of 1 pond = 350000/2 = 175,000 m^2
L:B =2:1
$A = L.B = [2.B].B = 2.B^2$
$B = (175000/2)^{0.5} = 296$ m
L = (L/B) x B = 2 B = 2x 278 m = 592 m
• Length: L = 592 m
• Breadth: B = 296 m
So V 1 pond = 175000*2 = 350000 m^3

g) Adoption of a value for BOD removal coefficient (K)
Complete-mix regime, at 20°C, $K = 0.35d^{-1}$

Correction for the temperature of 23°C:

Adopting a value for the temperature coefficient ϴ = 1.05:
$K_T = K_{20}.\Theta^{(T-20)} = 0.35 \times 1.05^{(23-20)} = 0.41\ d^{-1}$
h) Estimation of the effluent soluble BOD.
Using the complete-mix model (considering a not predominantly longitudinal cell).

$$S = \frac{S_0}{1 + K_T.t} = \frac{192}{1+.41*15} = 26.9 \text{ mg/L}$$

Note:

If the dispersed-flow model had been adopted, with the dimensions L, B and H determined in item m below, together with equations from (Table 9, Equation 4.33, and K = 0.15 d^{-1} for 20°C, ϴ = 1.035), this would have lead to:
- d = 0.50 (according to Eq. 4.33)
$a = (1 + 4K.t.d)^{0.5} = 2.3$

$$S = S_o.\frac{4ae^{1/2d}}{(1+a)^2e^{a/2d} - (1-a)^2\ e^{-a/2d}}$$

S = 44.3 mg/L

i) Estimation of the effluent particulate BOD
An effluent SS concentration equal to 60 mg/L, and considering that each 1 mgSS/L implies a BOD_5 of around 0.30 mg/L :
Particulate BOD_5 = 0.3 $mgBOD_5/mgSS$ x 60 mgSS/L = 18 $mgBOD_5/L$

j) Total effluent BOD
Total effluent BOD = Soluble BOD + Particulate BOD
Total effluent BOD = 44.3 + 18 = 62.3 mg/L
K) Calculation of BOD removal efficiency

$$E = \frac{S_0 - S}{S} . 100 = \frac{192 - 62.3}{192} .100 = 66 \%$$

It cannot use this method individually because the efficiency is very low and use the effluent in the tree forest.

0) Sludge accumulation
Accumulation per year = 0.05m m^3/inhab. x 347,223 inhab. =17361 m^3/year
Thickness in 1 year:

$$\text{Thickness} = \frac{17361\ m^3/\text{year x 1year}}{350000\ m^2}$$
$$=0.05\ \text{m/year} = 5\ \text{cm/year}$$

$$\text{Time} = \frac{H/3}{\text{Yearly thickness}}$$

Time = 2/3 / 0.05 = 13 year
So the sludge should be removed approximately every 13 year.

p) Layout of the system.

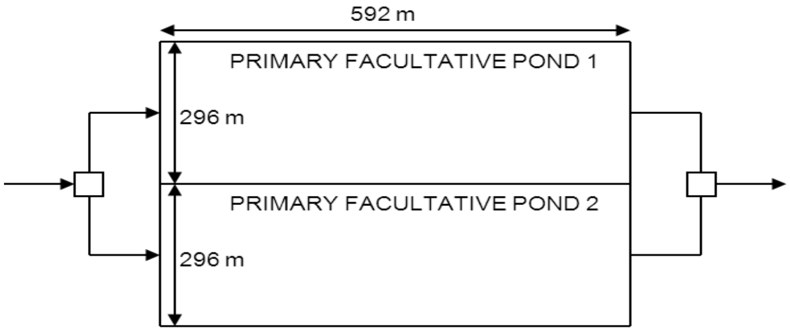


Figure 26. Dimensions of the Primary Facultative Ponds

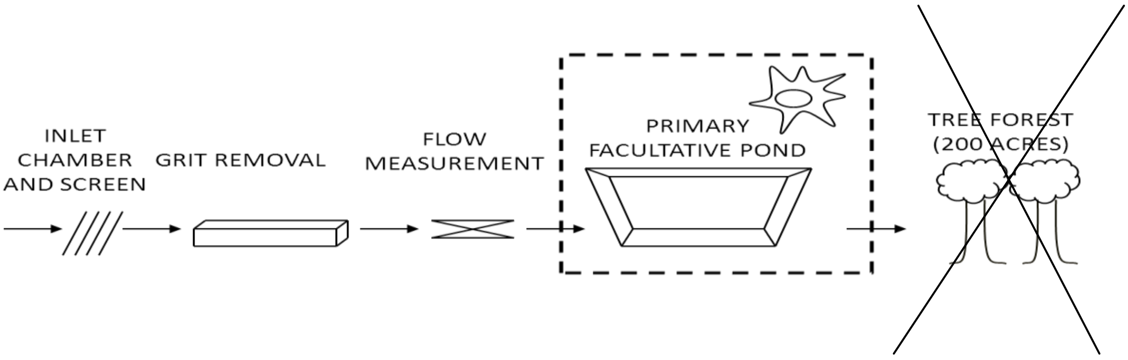


Figure 27. Layout of the Alternative 1 (Primary Facultative Ponds)

Alternative 2

Design a maturation pond system to treat the effluent from a primary facultative pond (Alternative 1) for the waste water treatment plant - Jaradah - town of El Arish in Egypt.
Given the following data from the report :
Population = 347,223inhab
Influent flow = 50,000m^3/d
Temperature: T = 23°C (liquid)
Faecal (thermo tolerant) coliform concentration in the raw wastewater:
N_o = 5 X 10 ^{7} FC/I00mL
<u>Data from the primary facultative ponds (Alternative 1):</u>
Number of ponds in parallel: 2
Length of each pond: L = 592 m
Breadth of each pond: B = 296 m
Depth: H = 2 m
Hydraulic detention time: t = 15 d.
Concentration of helminth eggs in the raw sewage: 200 eggs/L (assumed)

Solution

1. Coliform removal in the facultative ponds

a) Hydraulic regime to be adopted in the calculations
Adopt the dispersed flow regime.

b) Dispersion number d
Adopting Equation 4.33, and knowing that the L/B ratio in each facultative pond is 2.0 (556 m/278 m = 2.0):
d = 1/(L/B) = 1/2.0 = 0.50

c) Coliform removal coefficient
Using Equation 4.37 for dispersed flow, the value of the bacterial decay coefficient is obtained:
K_b(dispersed flow) = $0.542.H^{-1.259} = 0.542 \times 2^{-1.259} = 0.23\ d^{-1}$(20°C)
Correcting K_b, for 23°C:
$K_{bT}= K_{b20}.\Theta^{(T-20)} = 0.23 \times 1.07^{(23-20)} = 0.28\ d^{-1}$

d) Effluent coliform concentration
Adopting the equation for dispersed flow (Table 11), and knowing that the detention time in the facultative ponds is 15 days:
$a = (1 + 4K_b.t.d)^{0.5} = (1 + 4 \times 0.28 \times 15 \times 0.50)^{0.5} = 3.1$

$$N = N_o.\frac{4ae^{1/2d}}{(1+a)^2 e^{a/2d} - (1-a)^2 e^{-a/2d}}$$

$$= 5.0 \times 10^7.\ \frac{4 \times 3.1 \times e^{1/(2x0.50)}}{(1 + 3.1)^2 .e^{2.9/(2x0.50)} - (1 - 3.1)^2.e^{-3.1/(2x0.50)}}$$

$= 4.5 \times 10^6$ FC / 100 mL
This effluent concentration from the facultative pond is the influent concentration to the maturation ponds.

The coliform removal efficiency in the facultative pond is:

$$E = \frac{N_o - N}{N} \times 100 = \frac{5.0 \times 10^7 - 4.5 \times 10^6}{5.0 \times 10^7} \times 100 = 91\ \%$$

2. Alternative: a) three maturation ponds in series

a) $V = t * Q = 3 * 50{,}000 = 150{,}000\ m^3$

b) Determination of the required area and dimensions.

$$Area_{net} = \frac{Volume}{Depth} \longrightarrow A = \frac{V}{H} = \frac{150{,}000 m^3}{2\ m} = 75{,}000\ m^2 = 7.5\ ha$$

c) Adopt 3 ponds (n = 3)

Area of each pond: $75{,}000\ m^2/3 = 25000\ m^2$ (L:B=2:1-3:1) . (2)

L:B =2:1

$A = L.B = [2.B].B = 2.B^2$

d) $B = (25000/2)^{0.5} = 111\ m$

$L = (L/B) \times B = 2\ B = 2 \times 111\ m = 222\ m$

- Length: L = 222 m
- Breadth: B = 111 m

Rectangular ponds could have been also adopted, in order to improve the hydraulic characteristics and minimize the dispersion number.

g) Coliform concentration in the final effluent

Calculation according to the dispersed flow model:

$d = 1 / (L/B) = 1/2.0 = 0.5$

The value of the coliform die-off coefficient is given by:

K_b (dispersed flow) $= 0.542.H^{-1.259} = 0.542 \times 1.0^{-1.259} = 0.54\ d^{-1}$(20°C)

If Equation 4.36 (based on H and t) had been used, a value of $K_b = 0.58\ d^{-1}$ would have been obtained

For T = 23°C, the value of K_b is:

$K_{bT} = K_{b20}.\Theta^{(T-20)} = 0.54 \times 1.07^{(23-20)} = 0.66 d^{-1}$

The effluent coliform concentration from the 1st pond in the series is:

$a = (1 + 4K.t.d)^{0.5} = (1 + 4 \times 0.66 \times 4.0 \times 0.5)^{0.5} = 2.51$

$N_o = 4.5 \times 10^6$ FC / 100 mL

$$N = N_o . \frac{4ae^{1/2d}}{(1+a)^2 e^{a/2d} - (1-a)^2 . e^{-a/2d}}$$

$$= 4.5 * 10^6 \times \frac{4 \times 2.51.e^{1/(2x0.5)}}{(1+2.51)^2 . e^{2.51/(2x0.5)} - (1-2.51)^2\ e^{-2.51/(2x0.5)}}$$

$$= 0.8 \times 10^6\ FC/100mL$$

The removal efficiency in the 1st pond of the series is:

$$E = \frac{N_o - N}{N_o} \times 100 = \frac{4.5*10^6 - 0.8 \times 10^6}{4.5*10^6} \times 100 = 0.82 = 82\%$$

Considering that the three ponds have the same dimensions, the efficiency of the series of n = 3 ponds can be calculated:

$E_n = 1 - (1 - E_1)^n = 1 - (1 - 0.82)^3 = 0.994 = 99.4\%$

The coliform concentration in the final effluent is:

$N = N_o . (1 - E) = 4.5*10^6.(1 - 0.994) = 2.34 \times 10^4$ FC/100mL

Calculation according to the complete-mix model:

For illustration and comparison, the calculation for the complete-mix hydraulic regime is presented.
Coefficient K_b(20°C) for complete mix, based on the coefficient K_b for dispersed flow (K_b= 0.54 d^{-1}, for T = 20°C), t = 4.0 d and d = 0.5 so:
$K_{b\,mix} / K_{b\,disp} = 1.0 + [0.0540 \times (K_{b\,disp}.t)^{1.8166} \times d^{-0.8426}]$
$= 1.0 + [0.0540 \times (0.54 \times 4.0)^{1.8166} \times 0.5^{-1.4145}] = 1.58$
$K_{mix} = 1.58 \times K_{disp} = 1.58 \times 0.54 = 0.85\ d^{-1}$(20°C)
For T =23°C, K_b is corrected to K_b =0.81 d^{-1}.
The coliform concentration in the final effluent is given directly by the following equation, considering the total detention time of 12 d in all the ponds and the number of ponds n = 3 (see Table 11):

$$N = \frac{N_o}{(1 + K_b t / n)^n} = \frac{4.5*10^6}{(1 + 0.81 * 12/3)^3} = 5.9*10^4 \text{ FC/100ml}$$

The efficiency of the 3 maturation ponds is:

$$E = \frac{N_o - N}{N_o} \times 100 = \frac{4.5*10^6 - 5.9*10^4}{4.5*10^6} = 0.987 = 98.7\%$$

h) Overall removal efficiency
The overall efficiency of the primary facultative ponds - maturation ponds system in the removal of coliforms is:
• Dispersed-flow model for the maturation ponds:

$$E = \frac{N_o - N}{N_o} \times 100 = \frac{4.5*10^6 - 2.7 * 10^4}{4.5*10^6} \times 100 = 99.4\%$$

• Complete-mix model for the maturation ponds:

$$E = \frac{N_o - N}{N_o} \times 100 = \frac{4.5*10^6 - 5.9*10^4}{4.5*10^6} \times 100 = 98.69\%$$

Log units removed = - log (1 - E/100) = - log (1 - 99.4/100)
=2.22 log units removed

Notes:

The removal efficiency resulted from the dispersed-flow is 99.4 % and 98.69% resulted from complete-mix models. The effluent coliform estimations led to 2.7 x 10^4 FC/100mL from dispersed-flow model and 5.9 x 10^4 FC/100 mL from Complete-mix model. The proposed system of ponds does not comply with the WHO guidelines for unrestricted irrigation (1 x 10^3 FC/100 mL), but it can comply with some water body standards, depending on the dilution ratio of the receiving watercourse. The high removal of faecal coliform appears in the maturation ponds. The total detention time and/or number of ponds can be increased, if higher removal efficiencies are required. The increase in the detention time achieved through the increase in the surface area, and not in the depth. If the depth of the pond is decreased and the efficiency will increased, the value of K_b will be increased. In each individual pond, the detention time must be achieved, if a higher number of ponds in series are required.

A total = area (1 Primary facultative pond + 3 maturation ponds in series)
A total = 350000+75000 = 425000 m^2

3. Alternative: b) Single pond with baffles

j) Volume of the pond
Adopt a detention time equal to 12 days.
Volume of the maturation pond:
$V = t \times Q = 12\ d \times 50{,}000 m^3/d = 600{,}000\ m^3$

k) Dimensions of the pond
Depth: H = 1.0 m (adopted)
Surface area: $A = V/H = 600{,}000\ m^3 / 1.0m = 600{,}000\ m^2$ (60ha)

Adopt square external dimensions, but internal dimensions divided with 3 baffles .The baffles can be of tarpaulin, wood, earth banks, or other appropriate material.
$A = LB = 600{,}000\ m^2$ (60 ha)
External dimensions:
Length: L = 775 m
Breadth: B = 775 m
The internal L/B ratio of the pond will be (Equation 4.35):

$$L/B = \frac{L}{B}(n+1)^2 = \frac{775}{775} * (3+1)^2 = 16$$

Due to the division of the internal area with 3 baffles , the pond will have 4 compartments, each one with a length of 775 m and a width of 775/4 =193.75 m.

The pond can be considered as behaving as a rectangular pond, with a L/B ratio = 16, total length L = 775 x 4 = 3100 m and width 193.75 m.

l) Hydraulic regime to be adopted in the calculations

Adopt the dispersed-flow regime.
m) Dispersion number
Adopting Equation 4.33, with L/B = 16:
d = 1/(L/B) = 1/16= 0.06
"If the formula of Agunwamba had been used, the value d = 0.11 would have been obtained, along with d = 0.06". (20;21;22)

n) Coliform die-off coefficient
The value of the bacterial die-off coefficient can be given by (Equation 4.37):
K_b (dispersed flow) = $0.542.H^{-1.259} = 0.542 \times 1.0^{-1.259} = 0.54\ d^{-1}$ (20°C)
If equation 4.36 (based on H and t) had been used, a value of $K_b = 0.40\ d^{-1}$. would have been obtained.

For T = 23°C, the value of K_b is:
$K_{b\,T} = K_{b20}.\ \Theta^{(T-20)} = 0.54 \times 1.07^{(23-20)} = 0.66\ d^{-1}$

o) Effluent coliform concentration

Adopting the equation for dispersed flow (Table 11):

$a = (1 + 4K.t.d)^{0.5} = (1 + 4 \times 0.66 \times 12.0 \times 0.06)^{0.5} = 1.73$

$$N = N_o. \frac{4ae^{1/2d}}{(1 + a)^2 e^{a/2d} - (1 - a)^2 . e^{-a/2d}}$$

$$= 4.5*10^6 . \frac{4 \times 1.73 .e^{1/(2x0.06)}}{(1 + 1.73)^2 .e^{1.73/(2x0.060)} - (1 - 1.73)^2 . e^{-1.73/(2x0.06)}}$$

$= 9.53 \times 10^3$ FC/100mL

This system also does not comply (although it comes close) with the WHO guidelines for unrestricted irrigation (1×10^3 FC/100 mL), but it can comply with some water body standards, depending on the dilution ratio of the receiving watercourse.

<u>In this specific Alternative , the results are slightly better than in the case of the 3 maturation ponds in series.</u>

In any case, the high contribution given by the maturation ponds in the removal of faecal coliforms can be clearly seen.

p) Removal efficiencies

The efficiency of the maturation pond is:

$$E = \frac{N_o - N}{N_o} \times 100 = \frac{4.5*10^6 - 9.53 \times 10^3}{4.5*10^6} x100 = 99.78\%$$

The overall efficiency of the primary facultative ponds - maturation pond systems in the removal of coliforms is:

$$E = \frac{N_o - N}{N_o} \times 100 = \frac{5 X 10^7 - 1 \times 10^3}{5 X 10^7} x 100 = 99.998\%$$

Log units removed = - log (1 - E/100) = - log (1 - 99.998/100)

= 4.69 log units removed.

A total = area (1 Primary facultative pond + Single pond with baffles)

A total = 350000 + 600000 = 950000 m^2

4. Comparison between the two alternatives a and b.

Table 24. Comparison between the two alternatives a and b for maturation ponds system

Item	Alternative: a) 3 maturation ponds in series	Alternative: b) 1 maturation pond with 3 baffles (4 compartments)
Number of ponds	3 in series	1
Number of baffles	———	3
Total detention time (d)	4	12
Net area required (ha)	7.5	60
Length of each pond (m)	222	775
Width of each pond (m)	111	775
Depth (m)	2.0	1.0
Fc in the influent to the facultative pond (Fc/100mL)	5×10^{7}	5.0×10^{7}
Fc in the influent to the maturation pond (Fc/ 100mL)	4.5×10^{6}	4.5×10^{6}
Fc in the final effluent (Fc/100 mL)	27×10^{3}	9.53×10^{3}
Efficiency of the maturation ponds (%)	98.70	99.78
Removal of coliforms efficiency (primary facultative + maturation) (%)	99.4	99.998
Log units removed (global)	2.22	4.69

It can be observed that both alternatives a and b are equivalent from the point of view of land requirements and not so different in terms of the quality of the final effluent. It is still possible to have an optimization in the design in each alternative leading to improvements in the effluent quality. In the selection of the alternative, other items should be investigated, related to costs, topography, soil and other local factors.

*** Estimate the concentration of helminth eggs in the effluent from a system composed of facultative pond - baffled maturation pond.**

q) Removal of helminth eggs in the facultative pond
For design purposes, the removal efficiency of helminth eggs in the facultative pond is given by equation 4.31:

$$E = 100 \times [1- 0.41.e^{(-0.49.t+0.0085.t^2)}]$$

$$E = 100 \times [1 - 0.41.e^{(-0.49 \times 15+0.0085 \times 15^2)}]$$

$$E = 99.82\%$$

The concentration of eggs in the effluent from the facultative pond is:
$C_c = C_o$ X (1 - E/100) = 200 x (1 - 99.82/100) = 0.36 eggs/L

The effluent from the facultative pond already complies with the guidelines of the WHO for restricted and unrestricted irrigation (1 egg/L).

r) Removal of helminth eggs in the maturation pond
Again, for design purposes, the, removal efficiency of helminth eggs in the maturation pond is given by equation 4.31:
$E = 100 \times [1 - 0.41.e^{(-0.49.t+0.0085 * t^2)}]$
$= 100 \times [1- 0.41.e^{(-0.49\times12.0+0.0085\times12.0^2)}] = 99.61\%$

The concentration of eggs in the effluent from the maturation pond (final effluent of the system) is:
$Cc = C_o$ X (1 - E/100) = 0.36 x (1 - 99.61/100) = 1.4 x 10^{-3} eggs/L
This value corresponds, in practical terms, to a concentration of zero in the effluent.

5. Arrangement of the maturation ponds (including the primary facultative ponds).

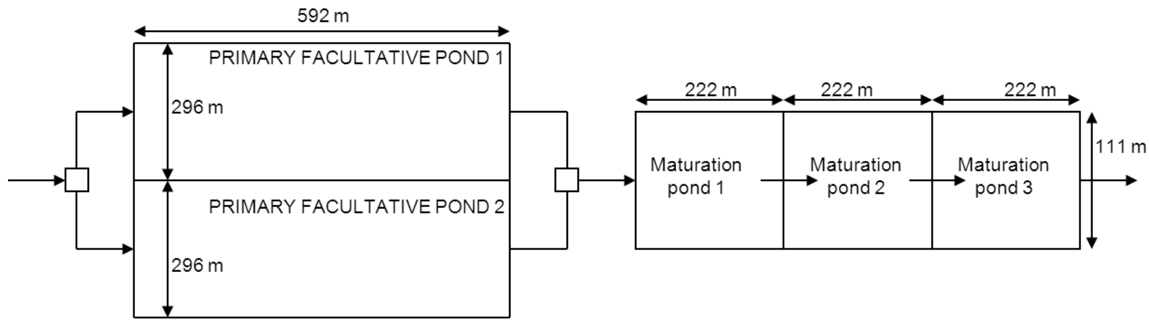


Figure 28. Dimensions of the 3 maturation ponds in series .

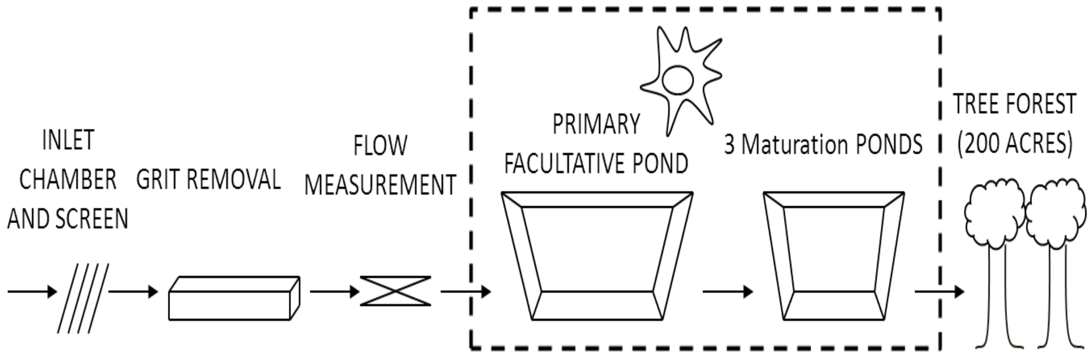


Figure 29. Layout of the Alternative 2 (3 maturation ponds in series)

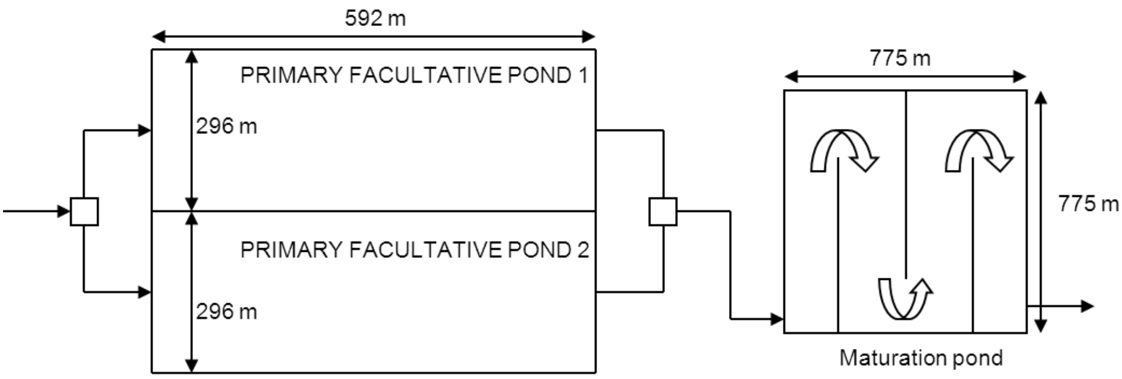


Figure 30. Dimensions of the maturation pond with 3 baffles.

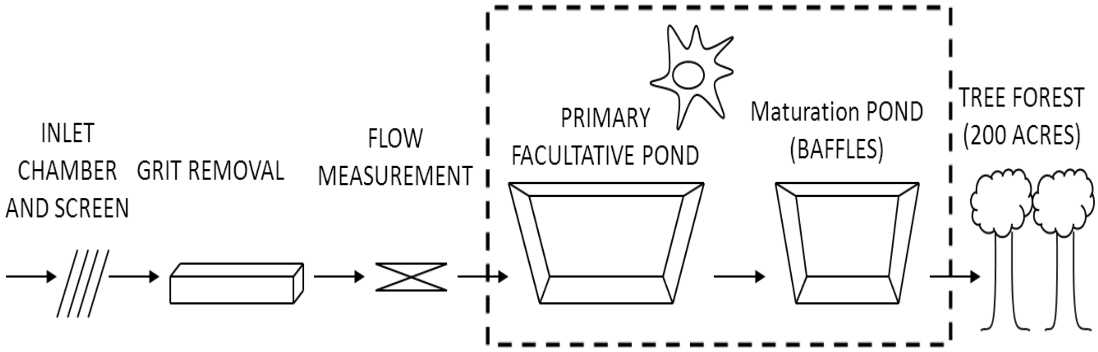


Figure 31. Layout of the Alternative 2 (1 maturation pond with 3 baffles)

Alternative 3

Design a facultative aerated lagoon system for the waste water treatment plant - Jaradah - town of El Arish in Egypt.
Given the following data from Alternative 1:
• Population = 347,223 inhab
• Influent flow: Q = 50,000 m^3/d
• Influent BOD: S_o = 192 mg/L
• Temperature: T = 23°C (liquid).

Solution

For design Anaerobic ponds :

* Calculation of the influent BOD_5 load.

Load = concentration x flow $= \dfrac{192\ g/m^3 .50{,}000\ m^3/d}{1000\ g/kg}$ = 9,600 kg/ d

L = 9,600 kgBOD_5/d
V = t * Q = 3 * 50,000 = 150,000 m^3

* Determination of the required area and dimensions.

Area $_{net} = \dfrac{Volume}{Depth} \longrightarrow A = \dfrac{V}{H} = \dfrac{150{,}000 m^3}{3\ m}$ = 50,000 m^2 = 5 ha

Adopt 2 ponds(n = 2)
Area of each pond: 50,000 m^2/2 = 25,000 m^2(L:B=2:1-3:1) . (2)
L:B =2:1
A = L.B = [2.B].B = 2.B^2
B =$(25000/2)^{0.5}$ = 111 m
L = (L/B) x B = 2 B = 2x 111 m = 222 m
• Length: L = 224 m
• Breadth: B = 112 m
Possible dimensions of each pond: 222 m x 111 m
* Concentration of effluent BOD.
BOD removal efficiency: E = 70 %
BODeffl = (1 - E/100).So = (1 - 70/100) x 192 = 0.3 x 192 = 57.6 mg/L
The effluent from the anaerobic pond is the influent to the primary facultative pond.

For Design a facultative aerated lagoon.

a) Detention time
t = 6 d (adopted)
b) Effluent soluble BOD
Assuming the complete-mix model and adopting the coefficient K = 0.7 d^{-1} for 20°C, corrected for 0.8 d^{-1} for 23°C:

Soluble BOD_5: $S = \dfrac{S_o}{1+K.t} = \dfrac{192}{1+0.8*6}$ = 33.10 mg/L

Lower values of S will be obtained if settling and anaerobic pond of the influent particulate BOD are considered.

c) Estimation of the effluent particulate BOD

Assuming that the effluent contains 80 mg/L of suspended solids, the concentration of effluent particulate BOD_5 will be approximately:

Particulate BOD_5 = 0.35 mgBOD_5/mgSS x 80 mgSS/L = 28 mg BOD_5/L

d) Total effluent BOD

Total BOD = soluble BOD + particulate BOD = 33.10 + 28 = 61.10 mg/L

To reduce the effluent BOD concentration, the detention time could be increased. However, this may not be economical, owing to the need of large volume increases for a small reduction in S. The configuration of the pond could also be changed, approaching a plug-flow pond. Besides that, the settling conditions in the outlet zone could be improved by the exclusion of some aerators (already done in this alternative).

The efficiency of the system in the removal of BOD is:

$$E = \frac{S_o - S}{S_o} = \frac{192 - 61.10}{192} \times 100 = 68.18\%$$

e) Required volume

$V = t.Q = 6\ d \times 50{,}000\ m^3/d = 300{,}000\ m^3$

j) Required area

Adopting a depth H = 4 m:

$$A = \frac{V}{H} = \frac{300{,}000 m^3}{4m} = 75{,}000\ m^2\ (7.5\ ha)$$

A total = area (2 anaerobic ponds + 1 facultative aerated lagoon + 3 maturation ponds in series in alternative 2)

A total = 50000+75000+75000 = 200000m^2

g) Oxygen requirements

$$OR = a.Q.(S_o - S) = \frac{1.0 \times 50{,}000\ m^3/d \times (192 - 33.10)\ g/m^3}{1000\ g/kg}$$

$$= 7945\ kgO_2/d = 331\ kgO_2/h$$

h) Power requirements

Adopt high-speed floating aerators. The oxygenation efficiency in standard condition is adopted as:

$OE_{standard}$= 1.8 kgO_2/kWh (range 1.2-2 kgO2/kWh)

The oxygenation efficiency in the field can be adopted as around 60% of the standard OE. Thus:

OE_{field} = 0.60 x 1.8 kgO_2/kWh = 1.08 kgO_2/kWh

The required power is:

$$P = \frac{OR}{OE}$$

$$= \frac{331\ kgO_2/h}{1.08\ kgO_2/kWh} = 306.5\ kW \sim 388\ HP$$

i) Aerators

Adopt 8 aerators, each of 50 HP.

Therefore, the total installed power is 8 x 50 HP = 400 HP (317 kW)

j) Pond dimensions

Adopt two ponds in parallel. With two ponds, there is a larger flexibility during the occasional periods of sludge removal (one pond being cleaned and one pond in operation).

The pond can have the following dimensions:
L:B =2:1
A = L.B = [2.B].B = $2.B^2$
B =$(75,000/2)^{0.5}$ = 194 m
L = (L/B) x B = 2 B = 2x 194 m = 388 m

Considering an square area of influence for each aerator, and leaving the final zone without aerators. The aerators can have the following dimensions:
B aerator =194/2 = 97 m
L aerator = 388/4 = 97m

(8 squares with dimensions 97m x 97m)

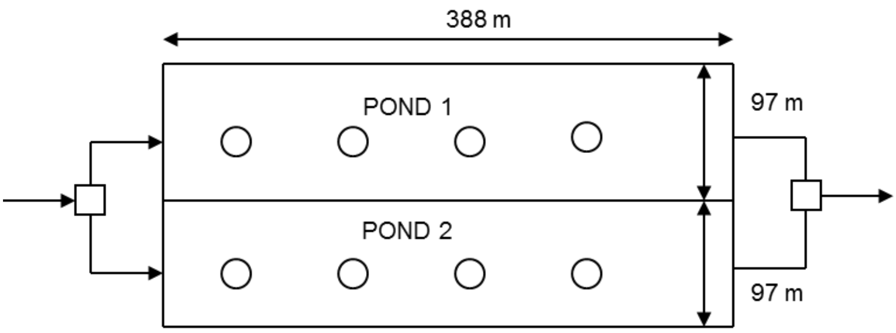


Figure 32. Dimensions of the Facultative Aerated Lagoon

k) Verification of the power level
The average power level in the whole lagoon is:

$$\Phi = \frac{P}{V} = \frac{320{,}000W}{300{,}000m^3} = 1.07\ W/m^3$$

This power level is expected to maintain solids in suspension. The estimation of 80 mg/L is reasonable (see Table 15), considering that there will be some settlement on the unaerated zone of the lagoon. The power level in the aerated zone only is larger, since the volume of the aerated zone is 75% of the total pond volume (7/8 of the pond length have aerators and 1/8 is without aerators - see item j).

I) Sludge accumulation

Annual accumulation = 0.05 m^3/inhab.year x 347,223inhab = 17,361 m3/year
Thickness in 1 year:

$$\text{Thickness} = \frac{17{,}361\ m^3/\text{year} * 1\ \text{year}}{75{,}000\ m^2} = 0.23\ m/\text{year}$$

$$\text{Time} = \frac{H/3}{\text{Yearly thickness}}$$

Time = 4/3 / 0.23 = 5.8 year

So the sludge should be removed approximately every 5.8 years.

n) Arrangement of the system

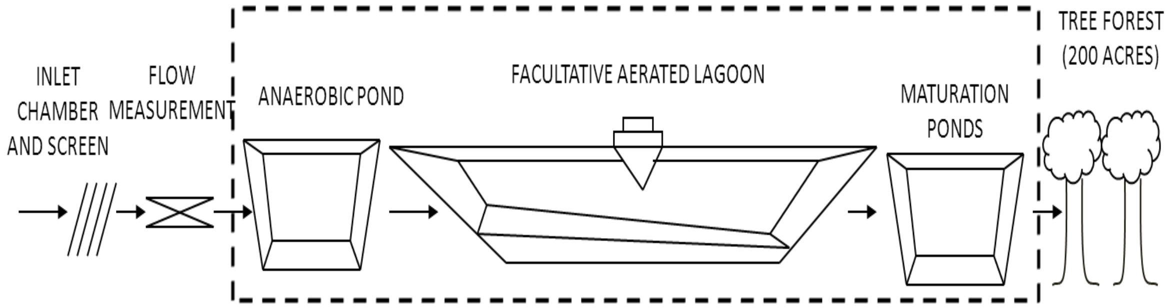


Figure 33.Layout of the Alternative 3 (Facultative Aerated Lagoon)

Alternative 4

Design a complete-mix aerated lagoon followed by a sedimentation pond, for the waste water treatment plant - Jaradah - town of El Arish in Egypt.
Given the following data from the report :
- **Population = 347,223inhabitant**
- **Influent flow: Q = 50,000m^3/d**
- **Influent BOD: S_o = 192 mg/L**
- **Temperature: T = 23°C (liquid).**

Solution

Aerated lagoon
a) Adoption of the detention time
t = 3 d (adopted)
b) Required volume
$V = t.Q = 3\ d \times 50{,}000 m^3 / d = 150,\ 000\ m^3$
c) Required area
Adopting a depth H = 4 m:
$$A = \frac{V}{H} = \frac{150,\ 000}{4m} = 37{,}500\ m^2\ (\ 3.75\ ha\ )$$
L:B =2:1
$A = L.B = [2.B].B = 2.B^2$
$B = (37500/2)^{0.5} = 137\ m$
$L = (L/B) \times B = 2\ B = 2\times 137m = 274\ m$
A total = area (2 anaerobic ponds + 1 complete-mix aerated lagoon + 2 sedimentation ponds + 3 maturation ponds in series in alternative 2)
A total = 50000+37500+33333+75000 = 195833 m^2

d) Estimation of the concentration of volatile suspended solids (VSS) in the aerated lagoon
Kinetic coefficients (see Table 17):
- Y = 0.6 (adopted)
- K_d = 0.06 (adopted)

Estimation of the effluent soluble BOD concentration (S):
S = 80 mg/L (initial estimate)
$$X_v = \frac{Y.(S_o - S)}{1 + K_d\ .t} = \frac{0.6 \times (192 - 80)}{1 + 0.06 \times 3} = 57mg/L = VSS_o$$
e) Estimation of the effluent soluble BOD
Assuming the complete-mix regime, and adopting the coefficient K' = 0.017 (mg/L)$^{-1}$ (d)$^{-1}$ after correction for 23°C:

$$\text{Soluble BOD}_5\text{: } S = \frac{S_o}{1 + K'.X_v.t} = \frac{192}{1 + 0.017 \times 57 \times 3} = 49.1mg/L \sim 50$$
j) Estimation of the effluent particulate BOD
Considering that the effluent from the aerated lagoon contains 57 mg/L of

volatile suspended solids, the effluent particulate BOD from the aerated lagoon will be:
BOD_{5part} = 0.6 $mgBOD_5$/mgVSS x 57 mg VSS/L = 34.2 $mgBOD_5$/L

This value is high for direct release into the receiving body, which justifies the need of the sedimentation pond downstream. Assuming that the sedimentation pond presents an efficiency of 80% in the removal of these volatile suspended solids, the VSS concentration in the final effluent from the system will be:

$$VSS_e = \frac{(100 - E)}{100} . VSS_o = \frac{(100 - 80)}{100} * 57 = 11.4 mgVSS/l \sim 15 mgVSS/l$$

Thus, the particulate BOD in the final effluent will be:
BOD_5 part = 0.6 $mgBOD_5$/mgVSS x 15mgVSS/l = 9 $mgBOD_5$/L
g) Effluent total BOD
Total BOD = soluble BOD + particulate BOD = 50 + 9 = 59 mg/L
The efficiency of the system in the removal of BOD is:

$$E = \frac{S_0 - S}{S_o} = \frac{192 - 59}{192} .100 = 69.27\ \%$$

h) Oxygen requirements
The oxygen requirements are around 1.1 to 1.4 of the removed BOD_5 load.
Adopting the value of 1.2 kgO_2/ $kgBOD_{rem}$:

$$OR = a .Q (S_0 - S) = \frac{1.1 \times 50{,}000\ m^3/d.\ (192 - 59)\ g/m^3}{1000\ g/Kg} = 7{,}315\ kgO_2/d$$

$$= 304.8\ KgO_2/h$$

i) Energy requirements
Adopt high-speed floating mechanical aerators. The Oxygenation Efficiency OE, in standard conditions, is in the order of:
OE = 1.8 kgO2/kWh
The oxygenation efficiency in the field can be adopted as around 60% of the standard OE. Thus:
OE_{field}= 0.60 x 1.8 kgO2/kWh = 1.1 kgO_2/kWh
The required power is:

$$P = \frac{OR}{OE} = \frac{304.8 kgO_2/h}{1.1\ kgO_2/kWh} = 277\ kW = 210\ HP$$

j) Aerators
Adopt 6 aerators, each of = 210/6=35 HP

Therefore the total installed power is 6 x 35 HP = 210 HP (277 kW)
Each aerator will be responsible for an area of influence of 68.5 m x 68.5 m (The dimensions of the pond are 274 m x 137 m).
According to table 16, for the power of 35 HP, the influence area is inside the oxygenation zone and close to the mixing zone. The depth of the pond is also satisfactory.

k) Verification of the power level

$$\Phi = \frac{P}{V} = \frac{277{,}000\ W}{150{,}000\ m3} = 1.85\ W/m^3$$

This power level is enough to maintain all the solids in suspension (Table 15).

Sedimentation pond

l) Design of the sedimentation pond

• Clarification zone (reserved for the liquid):

Detention time: t = 1.0 d (adopted)

Volume: V clarif= t.Q = 1.0 d x 50,000m^3/d = 50,000 m^3

Depth: H clarif = 1.5 m (adopted)

Required area:

$$A = \frac{V}{H} = \frac{50{,}000 m^3}{1.5} = 33333\ m^2 (3.33\ ha)$$

-Sludge zone (reserved for the storage and digestion of the sludge):

Add an additional depth of 1.5 m.

-Total dimensions and values (clarification and sludge zones):

Total area: 33,333 m^2

Depth: 1.5 m + 1.5 m = 3.0 m

Total volume: 33,333 m^2 x 3.0 m = 99,999 m^3

Assume number of ponds: 2

A one pond = 33,333/2 = 16666.5 m^2

L:B =2:1

A = L.B = [2.B].B = $2.B^2$

B $=(1666.5/2)^{0.5}$ = 92 m

L = (L/B) x B = 2 B = 2x 92 m = 184 m

Dimensions of each pond:

Detention time in a still clean pond:

$$t = \frac{V}{Q} = \frac{99{,}999}{50{,}000} = 1.99 \simeq 2.0\ d$$

m) Sludge accumulation

The load of influent solids to the settling pond is composed of volatile suspended solids VSS (determined in Section d) and fixed suspended solids SSF. Assume a ratio of 0.75 for VSS/SS . Hence, the ratio SSF/ VSS will be:

SSF/ VSS = (1 - 0.75)/0.75 = 1/3

The influent solids loads to the pond per year are:

<u>Volatile solids</u>: VSS = 50,000m^3/d x 0.153 kgVSS/m^3 x 365 d/year
= 2.79×10^6 kgVSS/year

<u>Fixed solids: SS_F</u>: = 50,000m^3/d x (0.153/3) kgSS_F/m^3 x 365 d/year
= 0.93×10^6 kgSS_F/year

Assuming a removal of 85% of the solids in the settling pond, the loads of volatile and fixed suspended solids that will be added to the sludge layer in the pond are:

$M_v = 0.85 \times 2.79 \times 10^6 = 2.37 \times 10^6$ kgVSS/year

$M_F = 0.85 \times 0.93 \times 10^6 = 0.79 \times 10^6$ kgSS_F/year

Estimate the sludge accumulation after a period of t years, and assuming a fraction of dry solids in the sludge of 8% (Water content = 92%) and K_v = 0.5 year $^{-1}$:

$$V_t = \frac{\frac{M_V}{K_V} . (1 - e^{-Kv.t}) + t.M_F}{1000. \text{(Dry solids fraction)}}$$

$$V_t = \frac{\frac{2.37*10^{6}}{0.5}\,(1-e^{-0.5*t}) + t * 0.79*10^{6}}{1000 \times 0.08}$$

Table 25. The sludge accumulation for different values of (t) .

Time (years)	Accumulated volume (m^3)	Ratio Vsludge / Vpond = H sludge / Hpond	Sludge height (m)
0.5	18,044	0.18	0.04
1.0	33,188	0.33	0.09
1.5	46,075	0.46	1.38
2.0	57,203	0.57	1.71
2.5	66,962	0.67	2.01
3.0	75,655	0.76	2.28
3.5	83,516	0.84	2.52

Column 2: equation above
Column 3: (column 2)/99,999 m^3, where 99,999 m^3 is the volume of the sedimentation pond
Column 4: (column 3) x 3.0 m, where 3.0 m is the total height of the sedi mentation pond
It is observed that after a period of around 1.75 years of operation, the volume reserved for sludge accumulation (corresponding to the height of 1.5 m) is totally used. Therefore, the removal of the sludge from the pond is necessary before this period.
0) Arrangement of the system.

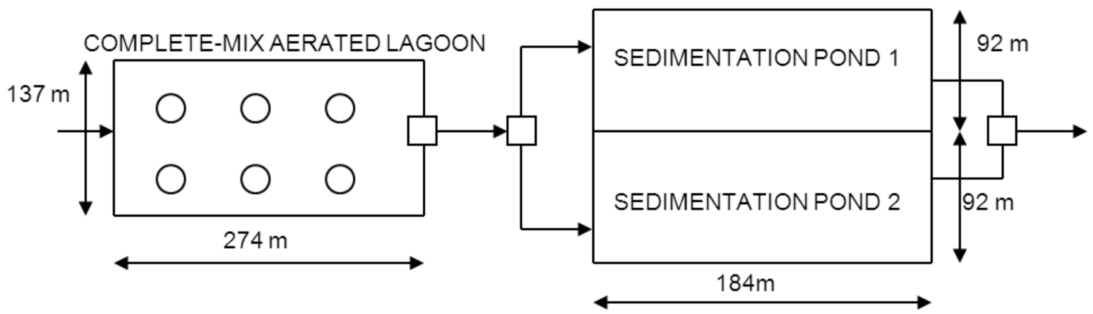


Figure 34.Dimensions of the complete-mix aerated lagoon followed by sedimentation ponds.

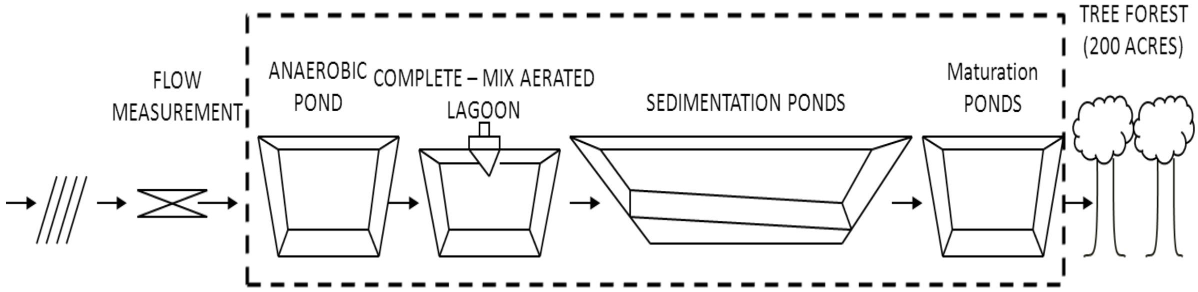


Figure 35.Dimensions of the complete-mix aerated lagoon followed by sedimentation ponds.

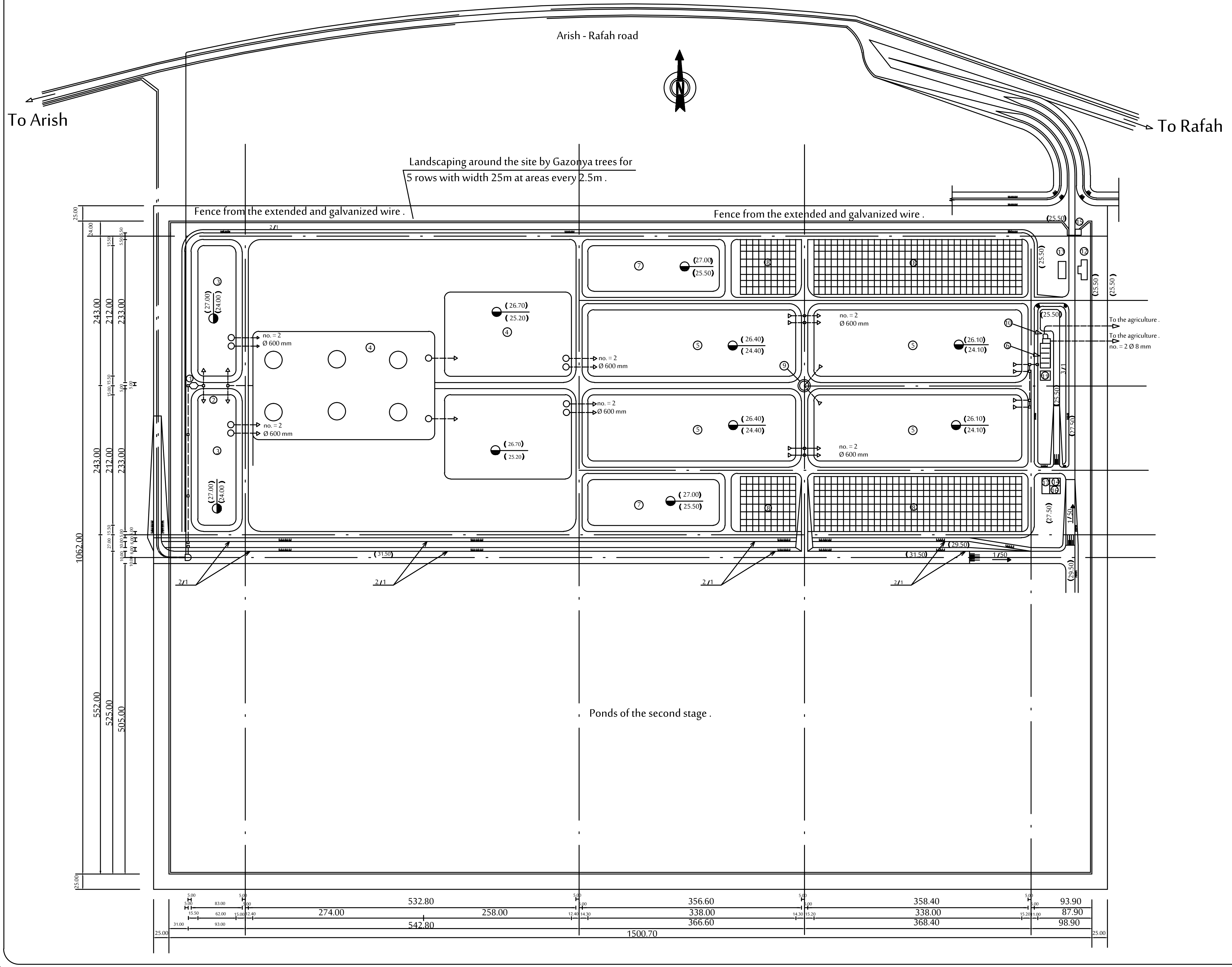
Table 26. Compare between the old design and new design after application Brazilian methods in town El Arish

Design	Old design (Egyptian code)				New design (Brazilian code)					
					Facultative ponds				Maturation ponds	
					without aerator	with aerators				
	Anaerobic ponds	New Anaerobic ponds	Facultative ponds	Maturation ponds	Primary Facultative ponds	Facultative aerated lagoons	Complete Mix Aerated Lagoons	Sedimentation ponds	3 ponds	Baffels
Q m3 / day	50,000	50,000	50,000	50,000	50,000	50,000	50,000	50,000	50,000	50,000
t (day)	2,3	5	9,9	1,57	15	6	3	1	4	12
H (m)	3	5	2	2	2	4	4	3	2	1
L (m)	233	224	532	356,6	592	388	274	184	222	775
B (m)	83	112	233	27,5	296	97	137	92	111	775
n	2	2	2	4	2	2	1	2	3	1
A one pond	19,339	25,000	123,956	9806,5	175,000	75,000	_____	16666,5	25,000	_____
A total	38,678	50,000	247,912	39,226	350,000	75,000	37,500	33,333	75,000	600,000
V one pond	58,017	125,000	247,912	39,226	350,000	300,000	_____	33,333	50,000	_____
V total	116,034	250,000	495,824	78,542	700,000	300,000	150,000	99,999	150,000	600,000
Sludge thickness	0,36m/year	0,28m/year	_____	_____	0,05m/year	0,23 m/year	_____	0,09 m/year	_____	_____
E %	81%	70%	81%	_____	66%	68,18%	69,27%	_____	98,70%	99,78%

Note :

From the calculations for design "the waste water treatment plant - Jaradah - town of El Arish" and application "Brazilian design methods", I proposed Alternative 4 "Complete Mix Aerated Lagoons - Sedimentation ponds " because of:

1- Need small areas.
2- Increase the rate of the removal efficiency of BOD.
3- Ensure the spread of suspend solids in the pond.
4- Good distribution for the oxygen.
5- Minimum retention time. (3 days in Complete Mix Aerated Lagoons and 1 day in sedimentation ponds)
6- Little amount of settlement sludge at the pond.



General Notes :

- All slopes is 3:1 .
- All the roads with width 10 m .
- In the bottom ponds is filtered layer PVC with thickness 1 mm .
- The ground level after (29.50) for the first stage . (27.50) for the second stage .
- Landscaping around the site by Gazonya trees for 5 rows with width 25m at areas every 2.5m and it irrigated from the wastewater treatment .

Water level in the pond .
Bottom level in the pond .

Number	Description
1	Inlet Chamber and Manual Screens .
2	Measuring flow device at the entrances .
3	Anaerobic pond .
4	complete-mix aerated lagoon - sedimentation pond .
5	Maturation pond .
6	Mixing chlorinated ponds .
7	Basins of drying sludge .
8	Storing sludge sites .
9	Submerged pump station to recycle flows from maturation ponds to facultative ponds .
10	Treatment water pumping station for the agriculture .
11	Storage of chlorine cylinders and injection .
12	Factory and Administration Building .
13	Umbrella cars .
14	Generate electricity building .
15	Gate Building , control and guard room .
16	Electrical transformers builing .
17	Electrical Panel feeding building .

Figure 31 : The waste water treatment plant - Jaradah - town of El Arish with complete-mix aerated lagoon - sedimentation pond .

References

References

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