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ABBREVIATIONS

BOD	Biochemical Oxygen Demand
COD	Chemical Oxygen Demand
DAF	Dissolved Air Flotation
EPA	US Environmental Protection Agency
FC	Faecal Coliform
FOG	Fates, Oils and Greases
FWS	Free Water Surface constructed wetland
LeAF	Lettinga Associates Foundation
MPN	Most Probable Number
NPK	Nitrogen – Phosphorous – Potassium
SSF	Sub-surface Flow
TDS	Total Dissolved Solids
TKN	Total Kjeldahl Nitrogen
TOC	Total Organic Carbon
TS	Total Solids
TSS	Total Suspended Solids
TVS	Total Volatile Solids
UASB	Up-flow Anaerobic Sludge Blanket
UNEP	United Nations Environment Program
USDA	US Department of Agriculture
VFA	Volatile Fatty Acid
VSB	Vegetated Submerged Bed constructed wetland
VSS	Volatile Suspended Solids
WSP	Waste Stabilisation Pond

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CHAPTER 10 SEWAGE TREATMENT AND DISPOSAL

10.1 INTRODUCTION

10.1.1 General

This Chapter of the Design Manual deals with the off-plot treatment of waste water effluents, primarily of a sanitary nature with a limited industrial component. It does not address the specific problems associated with the more toxic constituents found in specific wastewaters such as that from tanneries and the like, this being regarded as something that must be dealt with by factory site pre-treatment before such waste waters are allowed to enter a municipal sewer.

It now includes of sections dealing with further aspects of primary treatment, including grinding and the removal of fats, grease, and oils; an addition to the section on secondary treatment with an introduction to up-flow anaerobic sludge blanket units in the light of the increasing need for carbon capture; and bearing in mind the increasing problems associated with water quality in receiving waters a new section on tertiary treatment by natural and constructed wetlands.

10.1.2 Need for Waste Water Treatment

Waste water contains impurities resulting from domestic, commercial and industrial area disposal. The impurities, which are normally physical, chemical and biological in nature, are shown below:

TABLE 10.1: IMPURITIES IN WASTE WATER

IMPURITIES	NECESSITY FOR IDENTIFICATION
PHYSICAL IN NATURE - Total solids (TS) - Total volatile solids (TVS) - Total suspended solids (TSS) - Volatile suspended solids (VSS) - Total dissolved solids (TDS) - Colour - Odour - Temperature	- To asses the reuse of a waste water and to determine the most suitable type of operations and processes for its treatment. - To assess the condition of waste water. - To determine if odours will be a problem. - For the design and operation of biological processes in treatment facilities. - To determine the amount of oxygen required to stabilize the waste water completely.
CHEMICAL IN NATURE - Chemical oxygen demand (COD) - Total organic carbon (TOC) - Total Kjeldahl nitrogen (TKN) - Organic nitrogen (Org.N) - Free ammonia (NH_4+) - Nitrites (NO_2-) - Nitrates (NO_3) - Total phosphorus (TP) - Organic phosphorus (Org. P) - Inorganic phosphorus (In-org. P) - Chlorine (Cl)	- A substitute for Biochemcal oxygen demand (BOD_5) - To determine presence of Nutrients and the degree of decomposition in the waste water. - To asses the suitability of waste water for agriculture.

Cont'd

IMPURITIES	NECESSITY FOR IDENTIFICATION
CHEMICAL IN NATURE - Sulphate (SO_4^{2}) - Heavy metals	- To assess the treatability of the sludge - To assess the suitability of the waste water for re-use and the toxicity effects in treatment.
BIOCHEMICAL IN NATURE - Five day carbonaceous (Biochemical Oxygen Demand (BOD_2)) - Nitrogen oxygen demand (NOD)	- To determine the amount of oxygen required to stabilize a waste biologically. - To determine the amount of oxygen required to oxidize biologically the Nitrogen in the waste to Nitrate.
BIOLOGICAL IN NATURE - Toxicity - Coliform organisms (MPN-Most Probable No.) - Bacteria, Protozoa, Helminths, viruses	- To test the toxicity of waste water and treated effluent. - To identify presence of pathogens. - To asses presence of specific organisms in connection with plant operation and effluent re-use.

The impurities in waste water has to be adequately treated prior to disposal or re-use in order to:-

(i) Protect receiving waters from faecal contamination

(ii) Protect receiving waters from deleterious oxygen depletion and ecological damage (eutrophication)

(iii) Produce microbiologically safe effluent for agricultural re-use.

(iv) Preferably, to reduce methane and carbon dioxide from entering the atmosphere.

10.2 SEWAGE TREATMENT METHODS

10.2.1 Physical Treatment Methods

These include screening, sedimentation, filtration, and floatation.

These methods remove suspended solids in waste water in order to avoid development of sludge deposits which can lead to anaerobic conditions when discharge is to an aquatic environment. These treatments also protect pumps from being damaged by sand, gravel, etc. Floatation can also be used to remove fats, oils and greases which may interfere with biological processes.

10.2.2 Chemical Treatment Methods

These include chlorination, ozonization and chemical precipitation. The methods can remove nutrients like phosphorus and nitrogen which when discharged into aquatic environments can lead to growth of undesirable aquatic life (eutrophication) or the pollution of groundwater.

The methods also can eliminate pathogenic organisms in waste water and therefore reduce possibilities of transmission of communicable diseases.

However, both chlorination and the production of ozone are expensive and chlorine is an environmentally unfriendly product to produce and can be hazardous in its use.

10.2.3 Biological Treatment Methods

Biological treatment involves the conversion of dissolved and colloidal organic matter in waste water to biological cell tissue and to gas and the subsequent removal of the tissue usually by gravity settling in order to remove the undesirable BOD. The biological methods include anaerobic ponds, up-flow anaerobic sludge blanket units (when gases are to be minimised or re-used) , facultative ponds, maturation ponds, trickling filters, aerated lagoons and activated sludge plants and constructed wetlands.

10.2.4 Effluent Polishing

Where the quality of effluent produced by earlier processes is insufficient to meet the requirements for receiving waters, a polishing or tertiary treatment is needed and the use of natural or constructed wetlands is a method well suited to this.

10.3 CHOICE OF PRINCIPAL TREATMENT METHODS

The choice of waste water treatment method largely depends on simplicity, construction and operation cost, efficiency required or standards and land availability. Table 10.2 has been a guide used in the past for choosing a suitable treatment method but does not take into account the release of greenhouse gases. It considers the principal or secondary treatment stage, the primary treatment stage of screening, grit removal etc., being common to all.

TABLE 10.2: SUITABILITY OF SECONDARY WASTE WATER TREATMENT METHOD

TREATMENT METHOD	WASTE STABILIZ-ATION PONDS	CONVENTIONAL TREATMENT METHODS				
		ACTIVATED SLUDGE	AERATED LAGOONS	TRICKLING FILTERS	BIO-FILTERS	OXIDATION DITCHES
1. Cost (million US$)						
- Capital	5.68		6.98		7.77	4.80
- Operational	0.21		1.28		0.86	1.49
2. Energy consumption (kWh/yr)	NIL	1,000,000	800,000		120,000	
3. Efficiency						
- BOD removal						
- Total nitrogen removal	> 90% 70 – 90%					
- Total phosphorus removal	30 – 45%					
- Pathogenic organic removal	100%	90 – 99%	90.99%	90.99%	90 – 99%	90.99%
4. Land requirement (ha)	46	< 20	50	< 20	25	20
Source: Mara at el (1992)						

From a consideration of Table 10.2 above, it can be concluded that in general, a treatment process including waste stabilization ponds is more effective and affordable for Tanzania where land is usually relatively cheap and available. For this reason and except in special circumstances where constructed wetlands may be worth considering, or provide a tertiary treatment, a process that where possible includes waste water stabilization ponds is recommended and this Manual deals with this method in some detail. Because of an increasing need for carbon capture, it may however be worth considering replacing first stage anaerobic ponds by Up-flow Anaerobic Sludge Blanket (UASB) reactors so that the gases produced (methane and CO_2) are captured rather than released to the atmosphere.

General guidelines for preparation of an Environmental Impact Assessment document for a wastewater treatment plant are given in Appendix 10.1.

10.4 PRIMARY OR PRE-TREATMENT

All wastewater should undergo one or more types of pre-treatment before proceeding further in a sewage treatment works. First and foremost of this is screening.

10.4.1 Screening

Screening before any treatment is a pre-requisite for any sewage treatment so as to capture and remove gross solids grit, and other sizable objects and untreatable items such as stones and pieces of wood. Screens can also reduce the problems associated with such things as plastic bags whether or not they contain such waste as domestic garbage, illegally disposed of into sanitary sewers.

The criteria controlling screen design are as follow:-

Width of spaces between bars for coarse screens	75 – 150 mm
Width of bars for coarse screen	20 mm
Width of spaces between bars for medium screens	18 – 50 mm
Width of bars for medium screens	10 – 12 mm
Velocity of flow at maximum flow	0.9 m/sec
Velocity of flow at minimum flow	0.3 m/sec

(Ref: "Manual of British Practice in Water – Pollution Control – Preliminary Processes" Institute of Water Pollution Control, 1972)

10.4.2 Grinding

Whilst screens, provided they are cleaned at appropriate intervals, deal with larger matter un-amenable to treatment, they are not always able to deal with all material that due to size or constituents may cause subsequent treatment process problems.

In such instances, it may be desirable or even necessary to reduce the size of matter still further before it proceeds into treatment units and one of the more successful ways of doing this is by grinding. However, grinders require power and where this is unavailable, their benefits cannot be taken into account, and screening has to be all the more rigorous as a result.

One of the more successful grinding methods is to use a low speed twin screw grinder of the type illustrated. These can be used either in a pipe or channel to effectively macerate raw sewage, including plastic bags, as well as sewage sludges and screenings and in the case of the channel mounted type are available for flows from 0 – 780 m^3/hr.

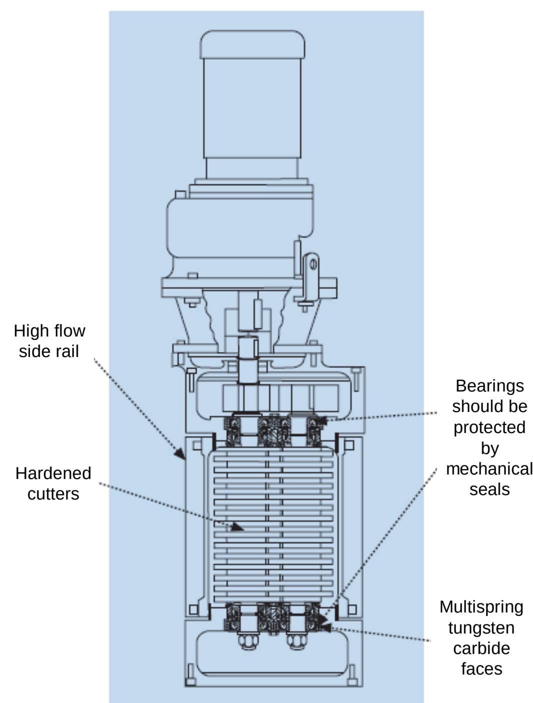


FIGURE 10.1: TWIN SCREW CHANNEL MOUNTED GRINDER

10.4.3 Fats, Oils and Greases (FOG)

Particles of FOG can be extremely deleterious to biological treatment processes and are also undesirable in receiving waters. It is always preferable to deal with as much of this problem at the principal sources such as food processing plants, abattoirs, restaurants, garages and workshops, before discharge into a sewer and this was discussed in Chapter 9. Enforcement of regulations pertaining to waste from such sources is eminently preferable to having to deal with the problem at a sewage treatment works but where this is unavoidable then a preliminary storage channel or tank with a turbulent air entraining inlet and low horizontal through-flow to allow for the FOG particles to hydrolyze and rise towards the surface may be necessary.

Devices such as Dissolved Air Flotation (DAF) bubble aerators or venturi aerators are a means of solids-liquid separation that transfer solids to the liquid surface through attachment of fine air bubbles to solid particles. The phenomenon consists of three processes

1. bubble generation (DAF) or introduction into the effluent stream (venturi aerator);
2. attachment of solids to the bubbles; and
3. solids separation.

Both require sufficient generating or inflow pressure to function and the air thus introduced helps lift FOG particles to the surface for decanting or skimming.

Where such mechanised processes are necessary it is obviously far preferable to do so with the relatively small volumes generated at source rather than the much larger and diluted volumes at the entry to a sewage treatment works.

10.5 Secondary Treatment

The principle treatment process in any sewage treatment works is termed the secondary treatment stage, and here there is a wide range of alternatives that have evolved over the years. Whilst the more mechanically oriented alternatives are used in colder climates or where space is at a premium, a system using all or in part, as noted above, Waste Stabilisation Ponds (WSP) are best suited to warm countries such as Tanzania.

10.5.1 Waste Stabilisation Ponds (WSP)

A WSP system comprises a combination of anaerobic, facultative and maturation ponds, or several such units in parallel. In essence, anaerobic and facultative ponds are designed for BOD removal and maturation ponds for pathogen removal, although some BOD removal occurs in maturation ponds and some pathogen removal in anaerobic and facultative ponds. The final effluent quality depends largely on the size and number of maturation ponds and a relatively non-toxic influent.

10.5.1.1 Anaerobic Ponds

Anaerobic wastewater treatment is the biological treatment of wastewater without the use of air or elemental oxygen. Many applications are directed towards the removal of organic pollution in wastewater, slurries and sludges. The organic pollutants are converted by anaerobic micro-organisms to a gas containing methane and carbon dioxide, known as "biogas".

Anaerobic ponds function as an open, non-mixed, unheated, single stage anaerobic digester (somewhat like a septic tank). They are designed to conserve heat and maintain anaerobic conditions effective in bringing about rapid stabilization of strong organic waste. They can receive very high organic loading, usually greater than 100g BOD per cubic metre per day, (100g BOD/m^3/d), (which is equivalent to greater than 3000 kg/ha d. Their primary function is BOD removal and they work extremely well in warm climates. A properly designed and not significantly under-loaded anaerobic pond will achieve around 60 % BOD removal at 20°C and as much as 75 % at 25°C. However and because they are not enclosed, they do not prevent the escape of gases such as methane and CO_2, nor facilitate their capture.

10.5.1.2 Facultative Ponds

A facultative pond is usually the second stage and main unit in a WSP system receiving effluent from the aerobic stage and treating wastewater of moderate strength. These ponds are designed mainly for BOD removal on the basis of a relatively low surface loading of 100 – 400 kg BOD/ha d. The ponds depths should be in the range of 1m to 2.4m. Operating depth of liquid should be from 0.6 to 1.5 m with 0.9 m free-board above the high water level.

The upper region is maintained in aerobic condition while anaerobic conditions prevail towards the bottom of the pond due to the fermentation of settled sludge. The conversion of organic carbon to methane in the benthal deposits contributes significantly to the reduction of BOD in a facultative pond.

A pond depth of at least 1m is necessary to prevent the growth of vegetation from the pond bottom, and a depth of water in excess of 1.5m tends to lead to predominantly anaerobic conditions in the pond contents.

Since a pond approximates to a complete mixed reactor and as part of the influent organic matter is converted to algae, it is not possible to reduce the effluent BOD to values less than about 60mg/1.

Indeed, the design objective should be to obtain a relatively high degree of BOD removal, while maintaining a stable balance between aerobic and anaerobic zones during all seasons of the year.

Wind has an important effect on the behaviour of facultative ponds as it induces a vertical mixing of the pond liquid. Good mixing ensures a more uniform distribution of dissolved oxygen, bacteria and algae and hence a better degree of waste stabilization.

In the absence of wind-induced mixing, the algal population tends to stratify in a narrow band, some 20 cm thick, during daylight hours. This concentrated band of algae moves up and down through the top 50 cm of the pond in response to changes in incident light intensity, and causes large fluctuations in effluent quality (DO and suspend solids) if the effluent take off point is within this zone.

10.5.1.3 Maturation Ponds

These are the last stage of a WSP system and 1m to 1.5m deep, designed mainly for pathogen removal. They receive the effluent from facultative ponds. The size and number of maturation ponds in the system is determined by the required bacteriological quality of the final effluent. These ponds usually show less vertical biological and physiochemical stratification and remain well oxygenated throughout the day.

Algal population is much more diverse than in facultative ponds and diversity increases from pond to pond along any series.

10.5.2 Process Design of Waste Stabilisation Ponds

Process design of WSP has become well established over the years, and is indicated in the following sections.

10.5.2.1 Effluent Quality Requirements

Effluent requirements are generally expressed in terms of:-

- Organic matter (commonly as BOD and COD)
- Suspended solids
- Nitrogen (total N, ammonia, Oxidized N)
- Total phosphorus
- Numbers of faecal coliform bacteria
- Numbers of human intestinal nematode eggs (*Ascaris lumbricoides*, *trichuris trichiura* and the human hookworms).
- Numbers of human trematode eggs (*schistosoma spp*)

The ultimate standard of effluent to be achieved will vary depending on whether or not the effluent is used for irrigation or dilution of the receiving waters. The required standards are summarized in Table 10.3 below:-

TABLE 10.3: STANDARDS FOR EFFLUENT FROM SEWAGE TREATMENT WORKS

DISCHARGE	BOD (mg/1)	TSS (mg/1)	E.Coli/100 ml
To Rivers	25	50	5,000
Unrestricted Irrigation (WHO)	25	50	1,000

The above standards require to be related to waste stabilization pond design and usually such standards can be achieved by appropriate design of maturation ponds. Exceptions may however occur where either FOG loads and/or heavy metal loads are high. The inclusion of the E. Coli standard is specifically related to treatment by waste stabilization ponds and does not relate to other forms of sewage treatment. Additional treatment for suspended solids removal and/or chlorination may be required for irrigation purposes.

The requirement for the above effluent standard is dictated by several factors:

(1) at times of low river flow dilution of the final effluent may be low. Ideally it should not be less than 8:1, and where it is more stringent standards may be required.

(2) river levels are lowest in the dry season when the demand for irrigation water may be highest.

(3) in some of the rivers there is access to the water for laundry and cleaning purposes.

(4) for rivers which may be dry for parts of the year special consideration is always required.

10.5.2.2 Design Parameters

The five most important parameters are temperature, net evaporation, flow, BOD and faecal coliform numbers. Helminth eggs are also important if the effluent is to be used in agriculture or acquaculture.

a) Temperature and Net Evaporation

The usual design temperature is the mean air temperature in the coolest month. This provides a small margin of safety as pond temperatures are 2-3°C warmer than air temperatures in the cool season (the reverse is true in the hot season).

Net evaporation (= evaporation minus rainfall) has to be taken into account in the design of facultative and maturation ponds (Shaw, 1962), but not in anaerobic ponds, as these generally have a scum layer which effectively prevents significant evaporation. The net evaporation rates in the months used for selection of the design temperatures are used; additionally a hydraulic balance should be made for the hottest month.

b) Flow

The mean daily flow should be measured very carefully, since the size of the ponds, and hence their cost, is directly proportional to the flow. A suitable design value for sanitary sewage is 85 % of the in-house water consumption, and this can be obtained from records of the Water Undertaker.

c) BOD

BOD may be measured using 24-hour flow-weighted composite samples. It can also be estimated from the following equation;

$$L_i = 1000B/q \qquad 10.1$$

Where

L_i = Wastewater BOD, mg/1

B = BOD contribution, g/capital d

q = Waste water flow, 1/capital d

Values of BOD very between 30 and 70 g per capita per day, and a suitable design value for Tanzania is 40 g per capita per day (*Mara, et al 1992*).

d) Faecal Coliforms

Grab samples of the wastewater may be used to measure the faecal coliform concentration. The usual range is $10^7 – 10^9$ faecal coliform per 100 ml, and in the absence of more specific information, a suitable design value is 1×10^8 per 100 ml.

e) Helminth Eggs

Grab samples may be used to count the number of human intestinal nematode eggs. The usual range is 100 – 1000 eggs per litre, the latter serving as conservative value for design.

10.5.3 Design Criteria for Waste Stabilisation Ponds

The design criteria for each type of WSP are described in the following sections.

10.5.3.1 Anaerobic Ponds

These are designed on the basis of volumetric BOD loading (v, g/m^3 d), which is given by:

$$v = Li \times Q/V_a \qquad 10.2$$

Where,

Li = influent BOD, mg/1 (=g/m^3)

Q = flow, m^3/d

V_a = anaerobic pond volume, m^3

The permissible design value v increases with temperature and the values in Table 10.4 may be safely used for design purposes.

TABLE 10.4: DESIGN VALUES OF PERMISSIBLE VOLUMETRIC LOADING ON AND % BOD REMOVAL IN ANAEROBIC PONDS AT VARIOUS TEMPERATURES

TEMPERATURE T in (°C)	VOLUMETRIC LOADING (g/m^3 d)	BOD REMOVAL (%)
< 10	100	40
10 – 20	20T – 100	2T + 20
> 20	300	60 *
* *Higher values may be used if local experience indicates that this is appropriate*		

The upper limit of v = $300 g/m^3 d$ for pond design is set in order to provide an adequate margin of safety with respect to odour.

Volumetric BOD loading (v) should lie between 100 and 400 g/m^3 d, in order to maintain anaerobic conditions and to avoid odour release. The upper limit for design in Table 10.4 is set at $300 g/m^3 d$, in order to provide an adequate margin of safety with respect to odour. This is appropriate for normal domestic or municipal waste waters which contain less than 500mg BOD/1.

After selecting the value of v, then the anaerobic pond volume is calculated from equation 10.3.

The mean hydraulic retention time in the pond (θa,d) is determined from;

$$\theta a = Va/Q \qquad 10.3$$

10.5.3.2 Facultative Ponds

Facultative ponds are designed on the basis of surface BOD loading equation;

$$s = 10 \times Li \times Q/A_f \qquad \textit{(Mara 1976)} \qquad 10.4$$

Where, A_f = facultative pond area, (m^2)

s = Surface BOD loading (kg/ha d)

Q = Inflow wastewaters (m^3/d)

Li = Influent BOD (mg/1)

The permissible design value of ‘s’ increases with temperature (T, °C).

There are several equations for finding the value of ‘s’ but the following equation should be used:

$$s = 350 \times (1.107 - 0.002 \times T)^{T-25} \quad \textit{(Source Mara et al 1992)} \qquad 10.5$$

Where T = Temperature in °C

Table 10.5 below can be used to select suitable values of S at different temperature.

TABLE 10.5: VALUES OF PERMISSIBLE BOD SURFACE LOADING ON FACULTATIVE PONDS AT VARIOUS TEMPERATURES (FOR EQUATION 10.5)

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

Once a suitable value of S has been selected, the pond area is calculated from equation 10.4. The retention time (θ_f, d) is calculated from the equation,

$$\theta_f = A_f \times D/Q_m \qquad 10.5$$

Where,

D = pond depth, m

Q_m = mean flow, m^3/day

Given by,

$$Q_m = (Q_i + Q_e)/2$$

Where,

Q_i = influent flow, m^3/day

Q_e = effluent flow, m^3/day

Which is given by,

$$Q_e = Q_i - (\text{evaporation} + \text{seepage})$$

Assuming seepage is negligible for well constructed ponds, then;

$$Qe = Qi - (0.001/(Af \times e)) \qquad 10.6$$

Where, e = evaporation rate, min/d

Therefore,

$$\Theta_f = (2 \times A_f \times D) / (2 \times Q_i - 0.001 \times A_f \times e) \qquad 10.7$$

BOD removal in facultative ponds operating well is estimated at 70 – 80 percent.

10.5.3.3 Maturation Ponds

a) Faecal Coliform Removal

The assumption is made that faecal coliform removal can be modelled by first order kinetics which is given by;

$$Ne = Ni / (1 + K_T \times \Theta) \qquad 10.9$$

Where,

Ne = number of FC per 100 ml of effluent.

Ni = number of FC per 100 ml of influent

K_T = first order rate constant for FC removal, per day.

Θ = retention time in days.

For a series of anaerobic, facultative and maturation ponds, the equation becomes;

$$Ne = Ni / [(1 + K_T \times \Theta_a) \times (1 + K_T \times \Theta_f) \times (1 + K_T \times \Theta_m)] \qquad 10.10$$

Where,

Θ_a = anaerobic retention time in days

Θ_f = facultative retention time in days

Θ_m = maturation retention time in days

n = number of maturation ponds

This equation is applies where the maturation ponds are of equal size. To minimize hydraulic short-circuiting Marais (1974) recommended a retention time of 3 days.

The value of K_T is temperature dependant, and given by;

$$K_T = 2.6 \times (1.19)^{T-20} \qquad \textit{(Marais 1974)} \qquad 10.11$$

Usually K_T changes by 19 percent for every 1°C change in temperature. Table 10.6 on the following page can be used for selection of K_T values.

Maturation ponds should carefully be designed to avoid overloading which can lead to interference in FC removal.

The loading on the first maturation pond is calculated on the assumption that 70 percent of the BOD has been removed in the preceding pond(s). *(source: Mara at el; 1992)*

Thus;

$$s(m1) = 10 \times (0.3 \times Li) \times (Q / A_{m1}) \qquad 10.12$$

or since $Q \times \Theta_{m1} = A_{m1}{}^{D}$

$$s(m1) = 10 \times (0.3 \times Li) \times (D / \Theta_{m1}) \qquad 10.13$$

TABLE 10.6: VALUES OF THE FIRST ORDER RATE CONSTANT FOR FAECAL COLIFORM REMOVAL AT VARIOUS TEMPERATURES (FROM EQUATION 10.11)

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

The maturation pond area is calculated from a rearrangement of equation (10.7) to obtain equation 10.14

Therefore,

$$Am = (2 \times Q_i \times \Theta_m) / (2 \times D + 0.001 \times e \times \Theta_m) \qquad 10.14$$

b) Helminth Egg Removal

Helminth eggs are removed by sedimentation and thus most egg removal occurs in anaerobic or primary facultative ponds. Therefore maturation ponds ensure that the final effluent contains at most one egg per litre. The percentage egg removal for design purpose is calculated using the following equation;

$$R = 100\,[1\text{-}0.41 \exp(-0.49\Theta + 0.0085\Theta^2] \qquad 10.15$$

Where,

R = percentage egg removal

Θ = retention time, (days)

The equation has to be applied sequentially to each pond in the series, so that the number of eggs in the final effluent can be determined.

The values of R obtained from equation 10.15 are presented in Table 10.7 on the following page.

c) BOD Removal

Maturation ponds are not designed for BOD removal. However, it is estimated that each pond can remove 25 % of BOD after 90% removal by Anaerobic and Facultative ponds cumulatively. *(Mara & Pearson 1987)*

TABLE 10.7: DESIGN VALUES OF PERCENTAGE HELMINTH EGG REMOVAL IN INDIVIDUAL ANAEROBIC, FACULTATIVE OR MATURATION PONDS FOR HYDRAULIC RETENTION TIMES IN THE RANGE 1-20 DAYS (FROM EQUATION 10.15)

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

d) Phosphorus and Nitrogen Removal

(a) **Nitrogen**

Nitrogen ($NH_3 + NH_4^+$) removal in individual facultative and maturation ponds in series for areas with temperature below 20°C, is given by;

$$C_e = (C_i) / \{[1 + [(A / Q) \times (0.0038 + 0.000134 \times T)\times\exp((1.041 + 0.044 \times T) \times (pH - 6.6))]\} \quad 10.16$$

For areas with temperature above 20°C;

$$Ce = (Ci) / \{1 + [5.035 \times 10\text{-}3 \times (A/Q)] \times \exp(1.540 \times (pH\text{-}6.6))]\} \quad 10.17$$

Where, C_e = ammonical nitrogen concentration in pond effluent, mg N/L

C_i = ammonical nitrogen concentration in pond influent, mg N/L.

A = pond area, (m^2)

Q = influent flow rate, (m^3/d)

T = temperature, °C (range, 1-28°C).

Θ = retention time, d, (range 5-231d)

The pH value for all equations should be estimated as;

$$pH = 7.3 \times \exp(0.0005 \times A) \quad 10.18$$

Where, A = influent alkalinity, mg $CaCO_3$/1

e) **Phosphorus Remova**

There is no equation to determine the phosphorus removal in WSP. However, it is known that if the BOD removal in a pond system is 90 %, phosphorus may be taken as 45 %.

NOTE: The size and number of ponds required in a waste stabilization pond system is shown in the design example given in Appendix 10.2.

10.6 PHYSICAL DESIGN OF WASTE STABILISATION POND SYSTEMS

Having completed the process design of a waste stabilization pond system; it has to be translated into a physical design. Things to consider at this stage include land availability to accommodate the calculated pond dimensions; embankments, pond inlet and outlet structures, preliminary treatment works required, and the necessity of by-pass pipes.

The physical design part is as important as the process design because it can significantly affect treatment efficiency as explained below:-

10.6.1 Pond Location

Ponds should be located at least 200m (preferably 500m) downwind from the community they serve and away from any likely area of future expansion to discourage people from visiting the ponds and avoid odour problems.

Ponds should not be located within 2km of airports to avoid risks to air navigation from birds attracted to the ponds. Vehicular access to the ponds should be provided for construction operational and maintenance reasons. Also to minimize earth works, the site should be flat or gently sloping and the soil must be suitable (with minimum seepage to minimize compaction work (see Section 10.6.2).

10.6.2 Geotechnical Considerations

The principal objectives of a geotechnical investigation are to ensure correct embankment design and to determine whether the soil is sufficiently impermeable, or requires the pond to be lined.

The maximum height of the ground water table should be determined to asses risks of ground water pollution.

The following are the soil properties which must be measured at the proposed pond location and samples should be taken at a depth of 1m greater than the envisage pond depth:

(i) Particle size distribution - e.g. medium to coarse sands are not suitable for embankments.

(ii) Maximum dry density and optimum moisture content (modified proctor test) – embankments soil should be compacted to 90% of dry density.

(iii) Atterberg limits

(iv) Organic content - e.g. peaty + plastic soils are not suitable for embankments

(v) Coefficient of permeability.

NOTE: The soil samples should be as undisturbed as possible.

Embankments slopes are commonly 1:3 internally and 1:1.5-2 externally, and external slopes should be planted with grass to increase stability. Externally embankments have to be protected from storm water erosion by providing adequate drainage. Internal embankments have to be protected against erosion caused by wave action by providing pre-cast concrete slabs or stone rip rap from just below top water level to embankment top.

The pre-cast slabs also prevent vegetation growth down the embankment into the pond, which otherwise creates a suitable habitat for mosquito or snail breeding.

10.6.3 Hydraulic Balance

To maintain the liquid level in the ponds, the inflow must be at least greater than net evaporation and seepage at all times. Therefore the relationship to be maintained is:

$$Qi = \geq 0.001 \times A\,(e + s) \tag{10.19}$$

Where,

Q_i = inflow to first pond (m^3/d)

A = total area of pond series (m^2)

e = net evaporation (i.e. evaporation less rainfall) (mm/d)

s = seepage (mm/d)

NB: Seepage losses must be at least smaller than the inflow less net evaporation so that the water level in the pond can be maintained.

The maximum permissible permeability of the soil layer making up the pond base is determined from Darcy's law:-

$$k = [Qs \times 8 / 6400 \times A]\,[\blacktriangle_1 / \blacktriangle h\,] \tag{10.20}$$

Where,

k = maximum permissible permeability (m/s)

Qs = maximum permissible seepage flow (Qi - 0.001 A e) (m3/d)

A = base area of pond (m^2)

$\blacktriangle_1$ = depth of soil layer below pond base to aquifer or more permeable stratum (m)

$\blacktriangle h$ = hydraulic head (= pond depth + $\blacktriangle_1$) (m)

If the permeability of the soil is more than the maximum permissible, the pond must be lined. The following is a guide for coefficients of permeability in relation to seepage.

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

$k > 10^{-8}$ m/s $\rightarrow$ the ponds will seal naturally

$k > 10^{-9}$ m/s $\rightarrow$ there is no risk of ground water pollution.

10.6.4 Pond Geometry

The optimal pond geometry is that which minimizes hydraulic short-circuiting. In general however, anaerobic and primary facultative ponds should be rectangular with a length-to-breadth ratio of 2-3 to 1 so as to avoid sludge banks forming near the inlet.

Secondary facultative and maturation ponds should whenever possible have higher length; breadth ratios (up to 10:1) so that they better approximate plug flow conditions. A single inlet and outlet to each are sufficient and should be located in diagonally opposite corners of the pond.

Waste stabilization pond systems serving more than 10,000 people should have two or more series of ponds in parallel to allow a continuous waste water treatment process even at times of maintenance.

10.6.5 Inlet and Outlet Structures

Inlet and outlet structures should be as simple and inexpensive as possible, but nevertheless perform correctly. They should also permit sampling at different points with ease.

The inlets to anaerobic and primary facultative ponds should discharge well below the liquid level to minimize short circuiting (especially in deep anaerobic ponds) and thus reduce the quantity of scum which is important in facultative ponds.

Inlets to secondary facultative and maturation ponds should also discharge below liquid level preferably at mid depth in order to reduce possibilities of short circuiting. For recommended inlet designs, see Figure 10.2 for anaerobic and primary facultative ponds and Figure 10.3 for secondary facultative and maturation ponds.

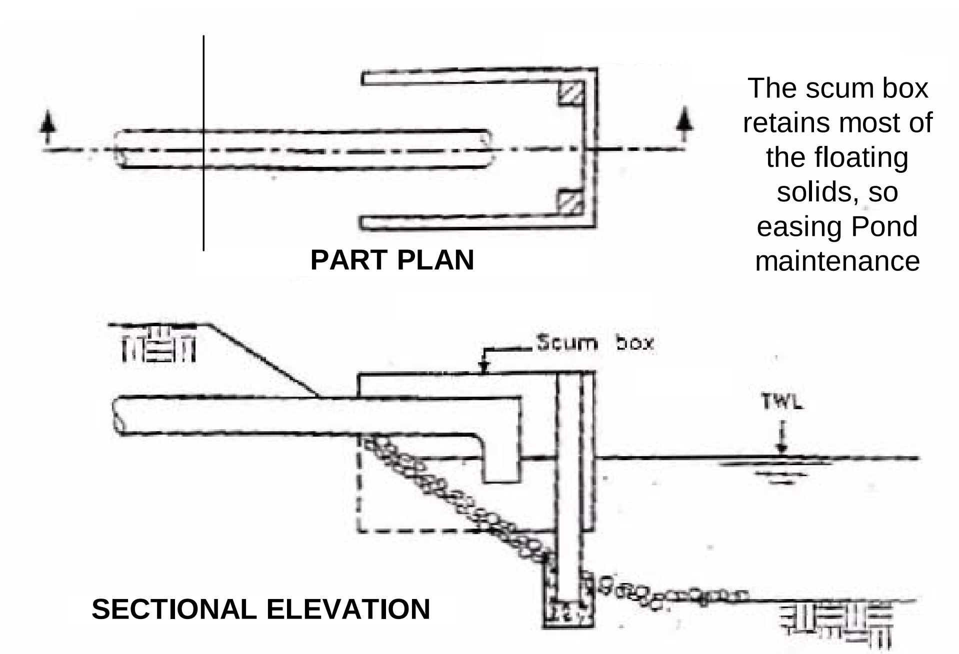


FIGURE 10.2: INLET ARRANGEMENT FOR ANAEROBIC AND PRIMARY FACULTATIVE PONDS

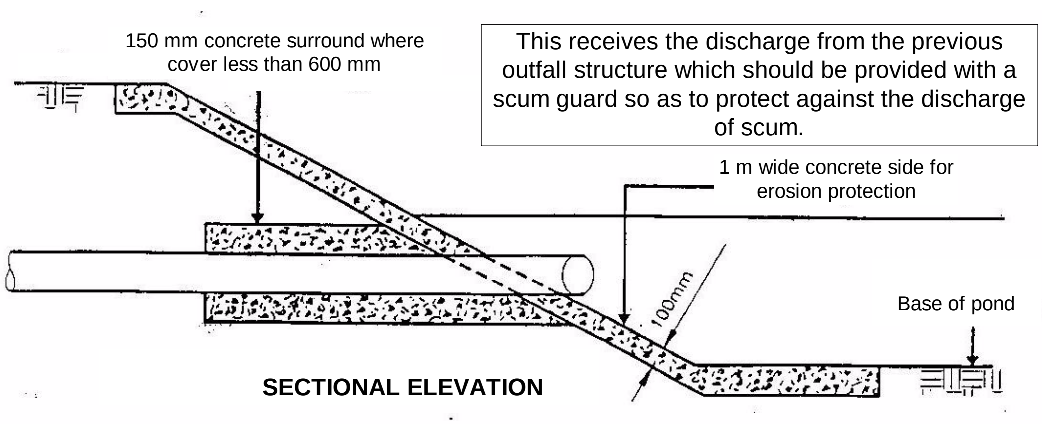


FIGURE 10.3: INLET ARRANGEMENT FOR SECONDARY FACULTATIVE AND MATURATION PONDS

Outlets of all ponds should be protected against the discharge of scum by the provision of a scum guard, see Figure 10.4.

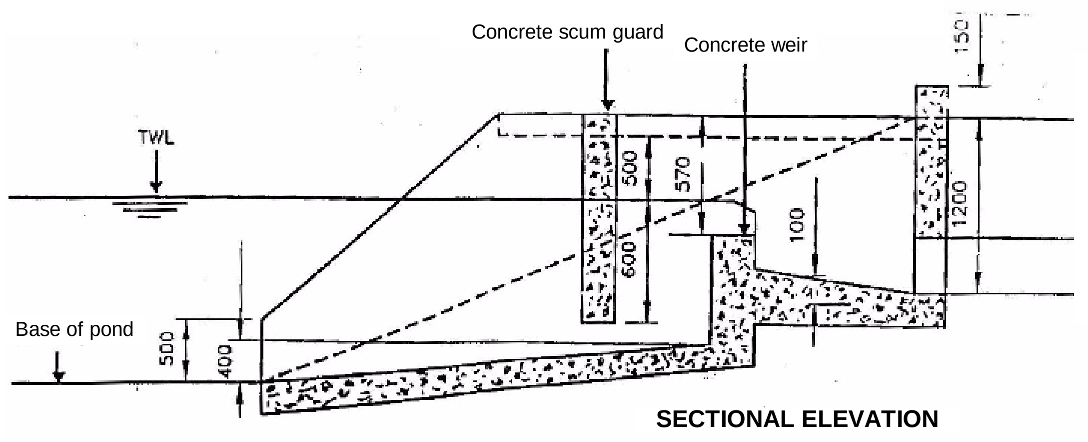


FIGURE 10.4: TYPICAL POND OUTLET STRUCTURE

The weir length is calculated from equation 10.21. Knowing the pond depth one can calculate the required height of the weir above the pond base:

$$q = 0.0567 \times h^{3/2} \qquad 10.21$$

Where,

q = flow per metre length of weir, (1/s)

h = head of water above weir, (mm)

The discharge pipe connects with the inlet structure shown in Figure 10.3. The concrete scum guard depth is described in Section 10.6.5. As an alternative, a variable depth wooden scum guard may be used.

The take-off level for the effluent which is controlled by the scum guard depth is important as it has a significant influence on effluent quality.

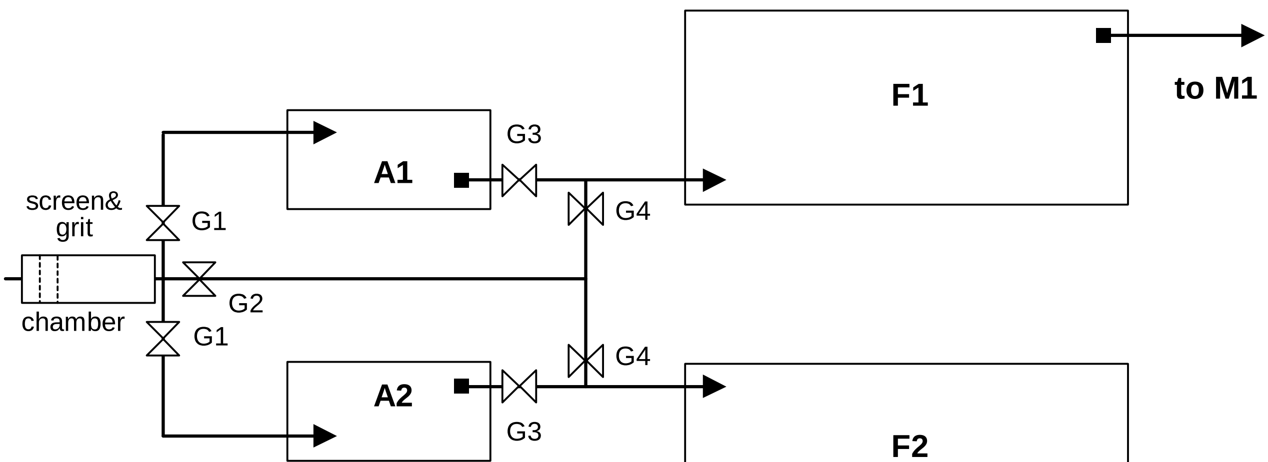
Recommended take-off levels are as follows:-

- Anaerobic ponds: 300mm
- Facultative ponds: 600mm
- Maturation ponds: 50mm

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

10.6.6 By Pass Pipework and Schematic Arrangement

It is necessary to provide a bypass to anaerobic ponds so that facultative ponds may be commissioned first as should always the case. This is also necessary during desludging operations. Figure 10.5 on the following page shows a schematic arrangement for two series of waste stabilization ponds in parallel.



During normal operation, gates G1 and G3 are open and gates G2 and G4 are closed.
To bypass one (or both) anaerobic ponds, gates G1 and G3 are closed and gate G2 and one (or both) of gates G4 are opened.

FIGURE 10.5: BY-PASS PIPEWORK AND SCHEMATIC ARRANGEMENT

10.7 UASB REACTORS AS AN ALTERNATIVE TO ANAEROBIC PONDS

Given the increasing need for carbon removal to reduce the problems of global warming, Upflow Anaerobic Sludge Blanket (UASB) Reactors are finding increasing favour as an alternative to anaerobic ponds where the level of technological operation allows. They allow for the capture of bio-gas, essentially methane and CO_2, which can then be used for the on-site generation of electricity should this be required. (For example security lighting, and influent lift pumping).

The UASB Reactor was first proposed in 1972 by Gatze Lettinga, a Dutch scientist, and is now estimated to be the basis for nearly three quarters of the world's anaerobic systems for treating industrial and residential wastewater.

By intent, Professor Lettinga made the core technology freely available as he foresaw its use especially in warmer developing countries and so did not patent it.

In a UASB reactor, a blanket of granular sludge forms and is suspended in the tank. Wastewater flows upwards through the blanket and is processed by the anaerobic micro-organisms. The upward flow combined with the settling action of gravity suspends the blanket with the aid of flocculants, and after start-up, the blanket begins to reach maturity at around 3 months. Small sludge granules begin to form whose surface area is covered in aggregations of bacteria. In the absence of a support matrix, the flow conditions create a selective environment in which only those micro-organisms, capable of attaching to each other, survive and proliferate. Eventually the aggregates form into dense compact biofilms referred to as 'granules' Each granule is an enormous "microbe metropolis" containing billions of individual cells and perhaps thousands to millions of different species.

Biogas with a high concentration of methane is produced as a by-product, and this can be captured and used as an energy source, to generate electricity to cover its own running power and for other site uses. The technology needs monitoring when put into use to ensure that the sludge blanket is maintained, and not washed out (thereby losing the effect). In colder climates, the heat produced as a by-product of electricity generation is reused to heat the digestion tanks.

The blanketing of the sludge enables a dual solid and hydraulic (liquid) retention time in the digesters. Solids requiring a high degree of digestion can remain in the reactors for periods up to 90 days whilst sugars dissolved in the liquid waste stream are quickly converted into gas in the liquid phase which can exit the system in less than a day.

According to Stoke's law, sedimentation rates are a function of particles size squared and due to their large particle sizes, anaerobic sludge granules have exceptional settling properties. The rapid settling velocities permits the application of high hydraulic loads without having to be concerned about wash-out of biologically active sludge particles (responsible for the bio-conversions). Because high hydraulic loads are tolerated, UASB reactors can handle wastewater streams with relatively low concentrations of substrate, even as low as a few hundred milligrams COD per litre (previously considered impossible for anaerobic treatment). Granular sludge settles extremely rapidly and is completely clarified within a few minutes. By comparison dispersed sludge (like that from an anaerobic digester at a municipal sewage treatment plant) has not even begun to clarify in the same time scale. Flocculent sludge, also clarifies rapidly but not as fast and as granular sludge.

Heavy metals are handled reasonably well by UASB reactors. However as with all biological processes, slug-loads should be avoided and the introduction of a UASB reactor into the waste treatment stream should not be taken as an excuse to allow industries to discharge their heavy metals without pre-treatment and especially as night-time slug-loads to try and avoid detection.

Studies of ionic chromium, cadmium, lead, copper, zinc, and nickel on methane production and volatile fatty acid (VFA) production in a UASB reactor have looked at the problem. Experimental results suggest that the effects of the heavy metals depended on the sludge zones, VFA types and metal types. The metal concentrations that caused 50% inhibition of methane production and total VFA production ranged from 210-2640 mg/l and 350->>5000 mg/l, respectively. The metals' relative toxicity to total VFA degradation was in the order of Cu>>Cr>Cd>>Zn>Ni>Pb and Cu>>Cd=Cr>>Ni>Zn>Pb for bed and blanket sludges, respectively Results have also confirmed that copper and lead are the most toxic and least toxic metals tested, respectively. In UASB biogranules, the organisms responsible for methane production are also less resistant to metal toxicity than those responsible for VFA production.

For more information, the document entitled "Design and Operation of UASB for Treatment of Domestic Wastewater" can be downloaded from the Wageningen University/Lettinga Associates

Foundation (LeAF). The following figures, abstracted from this publication illustrate a general layout, the UASB process, its design and critical parameters.

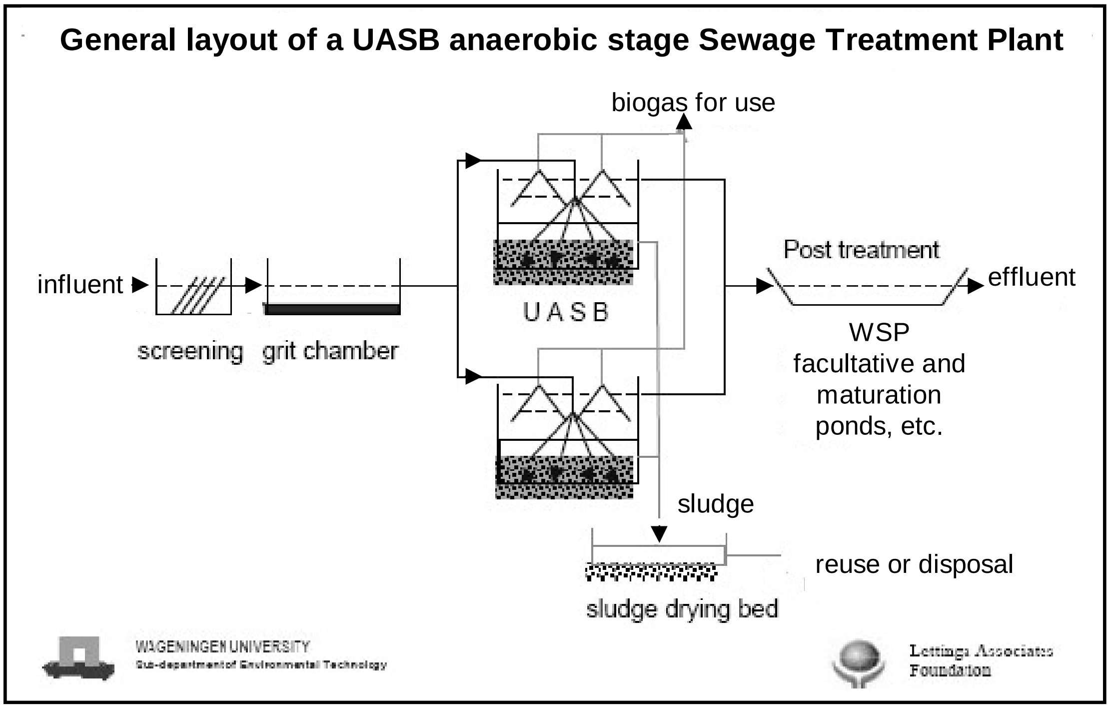


FIGURE 10.6: GENERAL LAYOUT OF A SEWAGE TREATMENT PLANT INCORPORATING A UASB

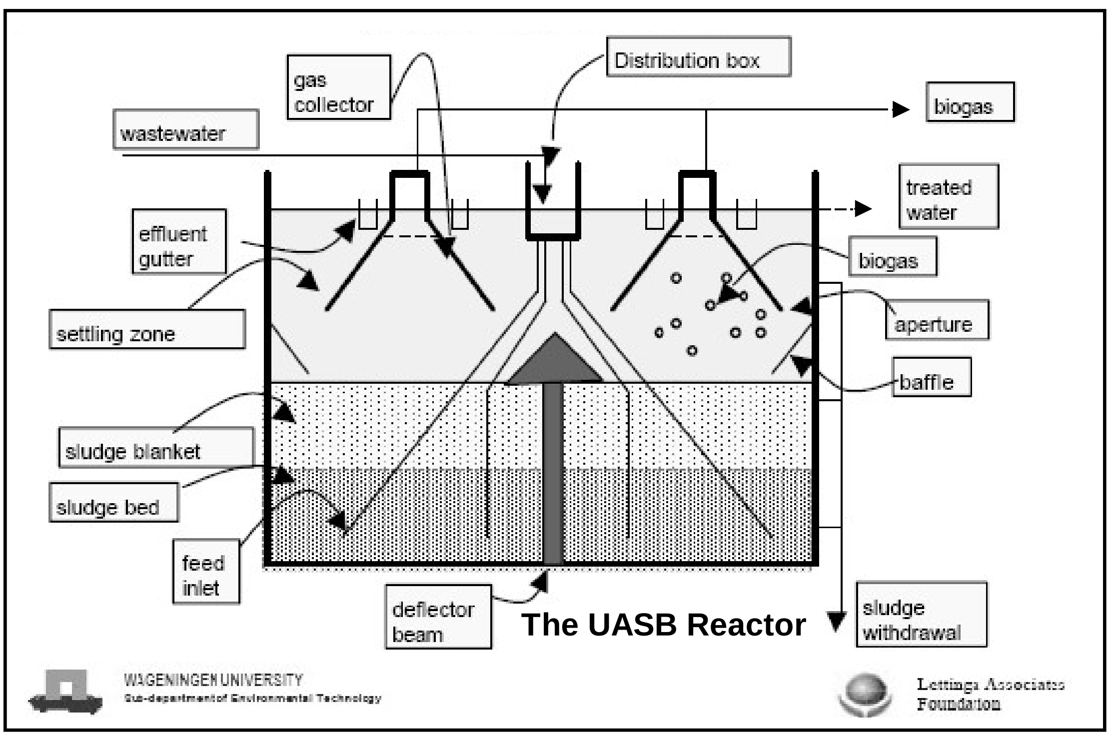


FIGURE 10.7: LAYOUT OF A UASB REACTOR

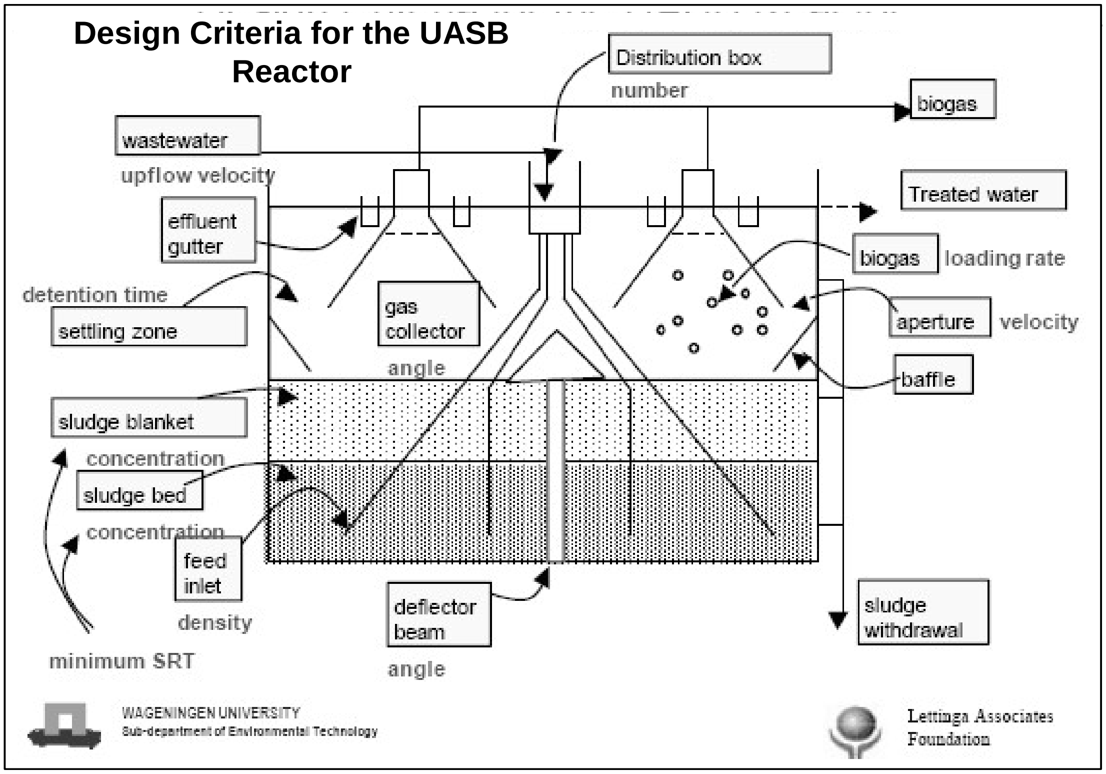


FIGURE 10.8: DESIGN CRITERIA FOR THE UASB REACTOR

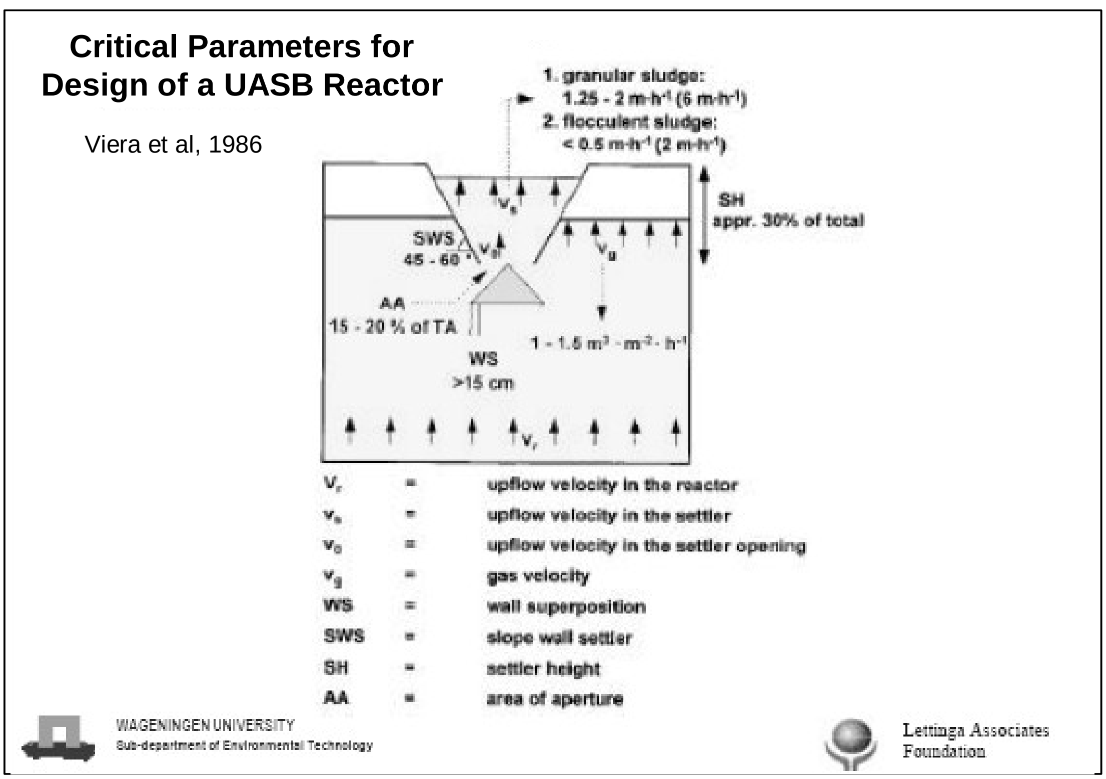


FIGURE 10.9: CRITICAL DESIGN PARAMETERS FOR A UASB REACTOR

10.8 TERTIARY TREATMENT IN WETLANDS

If for any reason, the Secondary Treatment Process is unable to meet effluent quality requirements or is not expected to do so, then a Tertiary Treatment Stage is necessary. For this purpose and whenever possible, consideration should be given to the use of Wetlands, either natural or constructed..

Additionally, and under some circumstances, constructed wetlands can be used instead of WSP as the secondary treatment stage or subsequent to a UASB reactor in stead of WSP.

The primary effect of wetlands, is one of phytoremediation and whilst most wetland plants will take up relatively large quantities of nutrients such as nitrogen, phosphorous, and potassium (NPK), the ability of different species to take up heavy metals, varies considerably.

Nevertheless, Phytoremediation has a large potential for treatment of pollutants in the environment, even if today, plants are not widely used. Phytoremediation processes include rhizofiltration (use of plant to accumulate compounds from aqueous solutions into roots), phytostimulation (use of plant to stimulate naturally occurring microbial degradation), phytostabilization (use of plant to prevent compounds from mobilizing or leaching in soil) and phytoextraction (use of plant to remove contaminants from soils into plant roots or shoots).

One of the greatest advantages of phytoremediation is its lower cost than other competing technologies. In addition to cost, phytoremediation offers other advantages : it is a non destructive in-situ technology applicable to a variety of contaminants, it is capable of remediating the bioavailable fraction of pollutants and accumulating heavy metals (Cu, Co, Ni, Zn, Cd, Pb). Some plants are seasonally dependent and hyperaccumulator plant species can also have a very low growth rate, making necessary to select those varieties capable of hyperaccumulation with high biomass production. It requires relatively large available surface area and it is applicable only to surface soils.

One of the major problems is the need, in some cases, to harvest the biomass and to dispose it as hazardous waste. Further research and development efforts are ongoing to increase remediation performances and to reduce treatment time, especially for high concentration pollutions or complex pollutions with mixtures of heavy metals and/or organic compounds. These efforts concern, for example, the optimisation of plant growth and plant uptake (plant selection, cultivation techniques, and fertilizers), harvesting techniques and the utilisation of harvested biomass.

10.8.1 Natural Wetlands

In Tanzania, natural wetlands occupy over 7% of the country's surface area. Most natural wetlands take the form of swamps with macrophytes vegetation typical to such areas including reeds, bulrushes, cattails, and sedges. Some wetlands are naturally seasonal in nature and the controlled discharge of effluent from WSB or a constructed wetlands can both maintain the natural swamp area through the dry season and allow it to polish the effluent before this reaches the watercourse. Some naturally occurring swamp plants are also effective at taking up toxic heavy metals into their root systems, (phyto-remediation). However where this is the case it must be determined that the heavy metals do not get into the leaf growth if such is going to be grazed or harvested as forage for cattle. There is also, limited information as yet on heavy metal uptake by species only indigenous to Eastern Africa.

However, for example, in one of the most common tropical wetland sedges, *Cyperus papyrus*, reportedly Cu and Pb mainly accumulate (adsorbed) in the roots, while Zn is also transferred to the leaves. The contribution of *Cyperus papyrus* to heavy metal removal may not only be actual accumulation in the plant and successive harvesting, but also the changing of the local conditions leading to an increased precipitation of heavy metals in the wetland soil. Another common sedge is *Cyperus Latifolius*, however its efficacy in this regard is not known. Root depths are limited.

Another plant is the common reed (*phragmites mauritanus*). A robust perennial with long rhizomes and culms to 7 m, usually simple. The leaves are lamina 15–100 cm long, green, rough beneath (at least towards the apex); somewhat stiff to ± pungent. Spikelets are 8–16 mm, usually with 4–8 florets. Root depths are moderate but usually greater than that for sedges.

Cattails such as *Typha latifolia* and *Typha domingensis* are common in natural wetlands and their efficacy in NPK uptake is reported to be good in natural wetlands. They have also been used in SSF constructed wetlands although their root depths are reportedly no better than sedges.

A common plant found on the edge of East African seasonal wetlands is *Miscanthus* or *Pennisetum purpureum*, (Elephant grass, Napier grass, or Uganda grass). It is a tall erect perennial with thick stems up to 4.5 m high, found on moist soils in areas with over 1,000 mm of rainfall annually. It is widely distributed along the banks of watercourses, and grows best on deep soils of moderate to fairly heavy texture. It tolerates short droughts, but does not withstand prolonged waterlogging.

Another is *Acroceras macrum* (Nile Grass), a perennial grass with stems up to 1 m high. The rhizomes and stolons from a dense sward. Commonly found on floodplains, river banks and swamp edges in eastern and southern Africa. It provides very palatable and nutritious dry-season grazing. and gives large yields of hay. It withstands waterlogging but not drought.

Whilst NPK uptake by these two grasses is good, there is as yet little information on their ability to take up different heavy metals, nor where such metals might accumulate.

The non-indigenous but established water hyacinth also consumes these nutrients, as well as heavy metals such as cobalt, lead, cadmium and chromium. However the weed has other more negative consequences and should never be introduced where it does not already exist, even though a good biological sieve of such pollutants.

Probably the best non-indigenous grass that can be planted is vetiver grass, (*vetiveria zizantoides*). It is already used for erosion control on slopes but elsewhere increasingly for phyto-remediation due its capacity to tolerate both drought and water logged conditions, its high biomass production, high nutrient uptake rates and exceptional transpiration rates.

Establishment normally requires only seasonal water-logging periods but recent experience from Australia indicates it can also be planted on pontoons floating in waste stabilisation ponds. It grows best when temperatures are of the order of 25°C but tolerates a wide range in this.

Originally indigenous to Northern India, and parts of South and South-east Asia, vetiver grass has neither stolons nor rhizomes but a massive finely structured root system, reportedly reaching 3-4m depth in the first year alone.

This massively thick root system reinforces the soil and at the same time makes it very difficult to be dislodged under high velocity flows.

It is highly efficient in absorbing dissolved N, P, Hg, Cd and Pb in polluted water and used in many parts of the tropical World for erosion control. It is drought-tolerant but also thrives in wetlands, it is therefore highly suitable for use in any wetland system to remove pollutant from industrial as well as agricultural wastes such as nutrients and agrochemicals from polluted water discharged from cropping lands and aquaculture ponds.

Currently, it is planted throughout much of India, South and South-east Asia, Australia and in parts of South America and Southern and other sub-Saharan parts of Africa, including Kenya, Uganda and Tanzania, but primarily for erosion control purposes.

FIGURE 10.10: VETIVER GRASS ROOT SYSTEM

It's growing acceptability worldwide is illustrated by the fact that it has recently been approved by USDA for planting in Hawaii where it is not indigenous. There is a Southern Africa Vetiver Network and a Pacific Rim Vetiver Network who can provide further useful information.

Because of fears that if allowed to spread beyond the area selected for its use, it is a potential weed, the planting of sterile cultivars (*Monto Vetiver*) are required in some parts of the World because of this. In general however, it spreads only by asexual reproduction or tillering, which is too slow to supply enough young plants to large areas. This naturally restricts its distribution.

Almost all (>90%) of the heavy metals absorbed by vetiver grass stays in the roots, therefore the leaves are largely uncontaminated and to get rid of the pollutants you have to remove the roots and dispose of them by deep burying or incineration.

As well as its heavy metal uptake, vetiver has the potential of producing up to 132 t/ha/year of dry matter yield as compared to 23 t/ha/year and 20 t/ha/year for Kikuyu grass and Rhodes grass, respectively. With this level of production vetiver planting has the potential of exporting up to 1,920 kg/ha/year of N and 198 kg/ha/year of P as compared to 687 kg/ha/year of N and 77 kg/ha/year of P for Kikuyu grass and 399 kg/ha/year of N and 26 kg/ha/year of P for Rhodes grass, respectively. Vetiver growth can respond positively to N supply up to 6,000 kg/ha/year and to ensure this extraordinary growth and N uptake, P supply level should be at 250 kg/ha/year.

10.8.2 Constructed Wetlands

A constructed wetland is defined here as an engineered or constructed wetland that utilizes natural processes involving wetland vegetation, soils, and their associated microbial assemblages to assist, at least partially, in treating an effluent or other water source. In general, these systems should be engineered and constructed outside naturally occurring flood plains.

The degree of wildlife habitat provided by a constructed wetland, or sections of such wetlands, varies broadly across a spectrum. At one end of the spectrum are those systems that are intended only to provide treatment for an effluent or other water source, in order to meet the requirements of treating the wastewater, and that provide little to no wildlife habitat. At the other end are those systems that are intended to provide water reuse, wildlife habitat, and public use, whilst also providing a final polishing function for a pre-treated effluent or other water source. Except in specific cases, the latter type of system is to be preferred.

It is considered that the use of well designed constructed wetlands should be encouraged. The functioning of different elements of a constructed wetland can be adjusted to enhance the performance of particular processes. Thus, a constructed wetland can be tailored for the particular pollutants it is receiving (*Hammer, 1992*).

Because the use of Constructed Wetlands in Eastern Africa is still in their infancy, the information provided here is primarily a set of guiding principles, guidelines and outline criteria to form the basis for more project specific design. More detailed information to assist in specific design is obtainable from the various publications on the subject, including those suggested in Section 10.8.2.6.

Their are two types of Constructed Wetlands, namely Free (or Open) Water Systems and Subsurface (or Vegetated Submerged Bed) Water Systems. In some instances a Subsurface Water System may precede an Open Water System. Both horizontal and vertical flow systems exist.

Schematics of the two systems are illustrated in the Figures below and on the following page, although it must be remembered that an open water system does NOT have to comprise units or regular shaped ponds, either in plan or in depth, and if intended to also provide for public recreational and educational purposes should deliberately be designed as a series of irregular, inter-connected ponds to look as natural as possible.

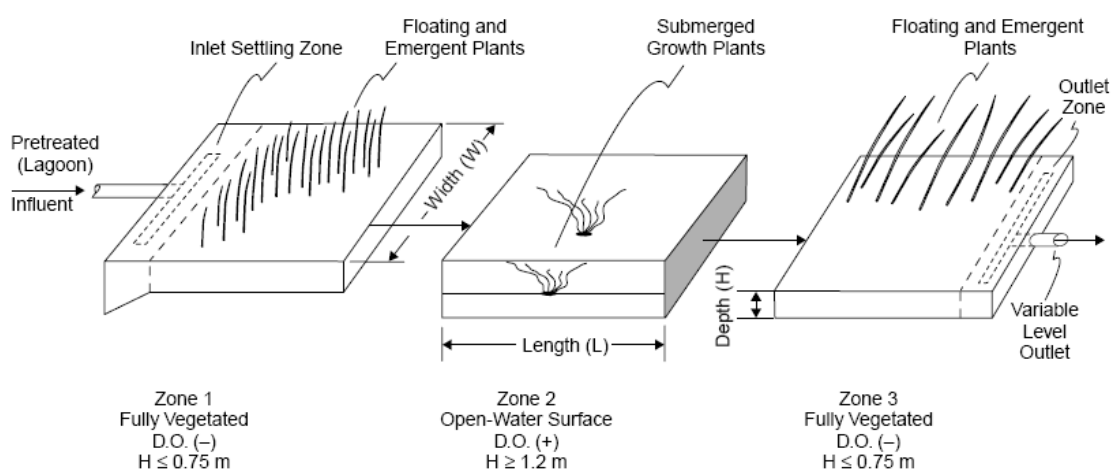


FIGURE 10.11: SCHEMATIC OF OPEN WATER CONSTRUCTED WETLAND

10.8.2.1 Guiding Principles for Constructed Wetlands

The Guiding Principles are to:

- Provide a framework for promoting sustainable, environmentally safe constructed wetland projects;
- be usable nationally under a variety of settings and circumstances;
- educate and inform designers and other interested parties, as well as water authority decision makers;

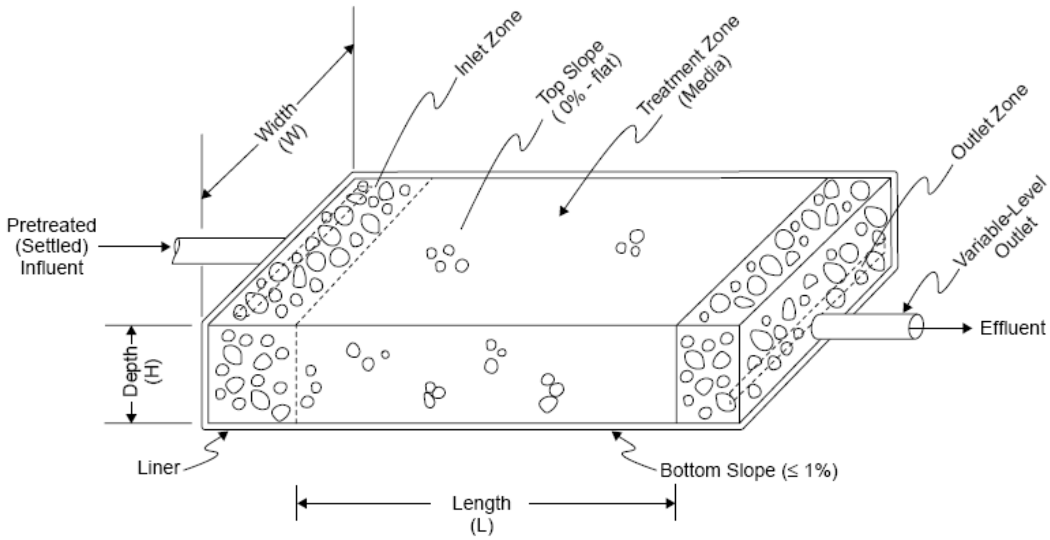


FIGURE 10.12: SCHEMATIC OF SUB-SURFACE CONSTRUCTED WETLAND

- provide guidance for environmental performance, especially for projects which are intended to provide water reuse, wildlife habitat, and public use, in addition to other possible objectives;
- highlight opportunities to restore and create wetlands;
- be applied, when appropriate, to any effluent or other source water treatment system as long as the source is adequately treated to meet applicable standards, protects the existing beneficial uses, and does not degrade the receiving waters;
- create opportunities for beneficial uses of dredged material, if feasible;
- minimize risks from contamination and/or toxicity;
- be applied in a watershed context; and
- be flexible enough to accommodate national differences in climate, hydro-geomorphology, wildlife habitat needs, etc.

a) Siting

Constructed treatment wetlands should generally be constructed on dry land (outside naturally occurring waters) and outside floodplains or floodways in order to avoid damage to natural wetlands and other aquatic resources. Also, wetlands so constructed elsewhere have been found to be somewhat more predictable than natural wetlands in terms of pollutant removal efficiency and in structural soundness, probably due to the engineering of constructed wetlands so located to provide favourable flow capacity and routing patterns.

However, opportunities may exist to use pre-treated effluent, or other source waters, to restored degraded wetland systems. In general, one should only locate constructed treatment wetlands in existing wetlands, or other waters, if (1) the source water meets all applicable water quality standards and criteria, (2) its use would result in a net environmental benefit to the aquatic system's natural functions and values, and (3) it would help restore the aquatic system to its historic, natural condition.

Siting in degraded wetlands should be avoided if the functions and values of the existing wetland will be adversely affected or water quality standards are likely to be violated.

b) Watershed Considerations

When developing a constructed wetland, consideration should be given to its role within the watershed, as well as within the broader ecosystem context of the region. Aspects of this role include: potential water quality impacts (physical, chemical, biological, thermal) to surface waters and groundwater; surrounding and upstream land uses; location of the wetland in relation to wildlife areas; potential threats from the introduction of non-native plant or animal species; and local peoples' perception of the appropriateness of constructed wetlands in their watershed. Whenever possible, the constructed wetland project should be planned in the context of a community-based watershed program.

c) Water-Depleted and Effluent-Dependent Ecosystems

Constructed wetland projects may provide valuable ecological benefits in regions where water resources, and especially wetlands, are limited due to climatic conditions and human-induced impacts, heavily farmed areas, and developed areas. Pre-treated effluent from wastewater treatment plants and seasonal return irrigation flows may be the only sources of water available for these areas and their dependent ecosystems.

d) Other Site Selection Factors

The suitability of a site for constructing a wetland may depend on the condition of one or more of the following factors: substrate, hydrology/geomorphology, vegetation, presence of Endangered Species or critical habitat, wildlife, cultural/socio-economic impacts, the surrounding landscape, land use/zoning considerations, and potential impacts to safety and health, such as impacts from major flooding events. Project designers should carefully examine these factors and consult as necessary in determining the most appropriate site(s) for the project, and should follow the necessary environmental impact review procedures or other requirements in selecting the final project location and characteristics.

10.8.2.2 Guidelines

a) Minimising Negative Impact

Adverse impacts to receiving waters should be avoided. Potential adverse impacts may include, but are not limited to: disruption of the composition and diversity of plant and animal communities; alteration of the existing hydrologic regime of natural wetlands or adjacent surface water bodies; introduction and spread of noxious species; threats to fish and wildlife from toxins and/or pathogens; and degradation of downstream water quality and groundwater sources.

b) Natural Structure

Constructed wetland designs should avoid rectangular basins, rigid structures and straight channels whenever possible. The use of soft structures, diverse and sinuous edges in design configuration, and bio-engineering practices that incorporate the existing natural landscape and native vegetation in constructed wetlands is encouraged. Gravity should be used to advantage and project design should be for minimal maintenance.

c) Buffer Zones

Margins of constructed wetland system should be designed as natural transition zones, including woody vegetated buffer areas around the site. Where appropriate, the facility should be integrated with other natural resource features to provide wildlife corridors and open space.

d) Vector Control

Facilities must be designed to minimize mosquito problems by minimizing the potential formation of stagnant water and by using natural biological control mechanisms, such as fish, (where native), bats, and birds.

e) Exclusion Devices

Wildlife exclusion devices, such as noise-making devices or netting and fencing, should be considered if the effluent or other water source being treated is toxic or presents a significant threat to wildlife. Such devices may be necessary in facilities that are designed only for treatment, but their need should be decided on a case-by-case basis.

f) Dedicated Water Source

If necessary, plans should be made on how the wetland habitat is to be maintained during periods of drought. Projects that are intended to provide wildlife habitat should have a dedicated water source for the life of the project and, if possible beyond to meet the long term hydrological needs of the desired aquatic and terrestrial communities. When doing this, efforts should be made to ensure adequate water supplies remain in adjacent streams for aquatic use and if groundwater is used, be sure that its mineral content is not toxic to indigenous plant species.

g) Biological Diversity and Physical Heterogeneity

To the extent possible, the constructed wetland should be designed to provide habitat with a broad range of native species, unless ecological considerations dictate otherwise. Vegetative species diversity should be maximised where appropriate, without increasing the proportion of weedy, non-indigenous, or invasive species at the expense of native species. Project plans should include mechanisms to control or eliminate undesirable species. The biological diversity of the project may be linked to, or dependent upon, physical heterogeneity. This can include having both surface and subsurface flow while providing some areas of open water, creating nesting islands for waterfowl, and leaving some land and buffer areas for other nesting species. Developing a wide variety of wetland types provides a range of diversity for different types of wildlife.

h) Seasonality and Capacity Exceedence

Project design should be able to accommodate extremes in meteorologic conditions and temporary exceedence of water storage and treatment capacity. Considerations should be made for extremes in temperature and precipitation which can impact normal operations.

i) Need for Pre-treatment Settling Basins

Either protect from storm water inflows or make provision for sediment collection/settling basins for treatment of storm water inflows and for additional treatment of wastewater. Design and locate the basins for ease of maintenance and to achieve greatest protection of wetland habitat and receiving waters. Monitor basins sediments, wetland vegetation tissues, and water quality to ensure the system is functioning properly and not becoming an attractive nuisance problem to wildlife. Identify an upland disposal site to dispose of accumulated sediments that is consistent with sediment disposal requirements and monitoring criteria and standards. Note that special disposal requirements may be applied for sediments containing hazardous waste materials.

j) Multiple Cells

The use of multiple cells may allow for residuals clean-out, repair of flow control structures, and specialized management of specific effluents without disruption of the overall systems operations. They also facilitate the flexibility of the system to manage different portions of the system (i.e., individual cells) for different purposes, such as the use of cells nearest the influent source to settle out sediment, final cells to strip out algae produced within the system, and other cells used to encourage the development of habitat and food production for specific wildlife species, etc. From a wastewater treatment standpoint, multiple cells often provide better treatment in part because 'short circuiting' is minimized.

k) Maintenance Access

The constructed wetland should be designed so that maintenance vehicles and personnel can safely and easily access the site with a minimum of disturbance. Proper access design will facilitate proper operation and maintenance of the wetland so that it performs as design.

l) Public Acceptance

The designer should consider the public's perception of any constructed wetland project and its affects on neighbouring populations and adjacent land uses. Potential concerns like drinking water contamination, unpleasant odours, mosquitoes, access by small children and other safety and health issues should also be taken into account. Planning the project with community involvement early in the process, will help ensure public support and approval for the goals and objectives while developing a safe project for everyone to enjoy.

m) Public Use

When appropriate, encourage public access and use, work with local educators to design informative displays to install at the project, and help foster community education programs, especially for projects developed to include for water reuse and wildlife habitat. In some cases, public access may need to be prevented due to safety and health concerns.

n) Pilot Projects and Design Criteria

A pilot project may be necessary for designing a full-scale project. If a pilot is not utilized, then design considerations should be fully described and made available to future operators and regulatory staff.

10.8.2.3 Sub-surface Flow Constructed Wetlands

Whilst there is now a considerable literature database for open or surface-flow constructed wetlands, that for sub-surface flow (SSF) systems outside the USA and Europe is not.

A SSF constructed wetland system has three important components, which interact in a complex manner to provide an ideal medium for wastewater treatment. These are;

- A relatively shallow bed of soil (between 0.5 and 1.5m deep) in which the plants are planted, contained by a waterproof membrane to prevent wastewater leakage.
- A suitable plant which ideally should thrive under water logged conditions, tolerates a high level of pollutants, exhibit a high capacity of absorbing these pollutants and have high biomass production under such extremely adverse conditions.
- Micro-organisms (fungi and bacteria) in the planted soil and provide most of the treatment. The plant's root and rhizome systems bring air into the soil immediately surrounding them. Further away, the environment is anaerobic. These aerobic and anaerobic zones host an appropriate range of micro organisms responsible for the impressive performance of such systems.

SSF Constructed Wetlands planted with locally indigenous reeds are efficient for BOD and TSS removal and useful in dealing with Faecal Coliform (1 - 2 log reduction). However they reportedly do not deal effectively with Nitrogen as a result of the limited aerobic conditions. Typical retention times for those using gravels as the support medium are 2 – 3 days.

Except where land is at a premium, gravel media systems are said to be 3-4 times more expensive than free surface constructed wetlands but size for size, perhaps twice as efficient for BOD removal. However, gravel media systems have better hydraulic properties than soils and hence keep overall size down but raise the cost.

Until now elsewhere, they have principally been used for stand alone treatment for relatively small communities and due to their lack of odour, for on-plot treatment. However both use as a first stage in secondary effluent treatment and for tertiary effluent polishing are also possible. A working system where the first-stage secondary treatment is sub-surface flow and the next and final stage several connected free-surface flow ponds has been in use to successfully treat the effluent from the Carnivore Restaurant and the Splash Swimming Pool complex in Nairobi, Kenya for nearly 15 years, and a visit is well worth while for those interested. Further information on this constructed wetland is provided in Appendix 10.3.

As noted above, the most common plants used to date have been from the *phragmites* family and there is a still a lack of literature on the efficacy and design of vetiver grass systems, which with its deeper root system is anticipated to be more effective for nitrogen removal especially in warmer climates as well as for toxic metals. In addition to vetiver, such plants as sugar cane, bananas and sweet potatoes have also been suggested as suitable, but here there is even less information currently available.

Research projects on the use of vetiver are ongoing in a number of different parts of the World including Europe and the USA, China, Australia and South Africa and again the site <www.vetiver.org> is probably the best to keep up to date on this. Of these countries, the most comparable results are probably those from Australia, and vetiver pilot studies there have so far suggested that with coarse soils, systems planted with vetiver removed 98-96% TSS, 91-72% total COD, 81-30% dissolved COD, and 82-93% of total N and P. Using sandy loams instead of gravels would also bring down costs.

Twin cell systems in parallel are recommended, especially where toxic metal removal is included, giving time for periodic removal and deep burying or incineration of the contaminated roots.

The major concerns in the design of SSF constructed wetlands include:

- Hydraulic and hydrological conditions,
- BOD and TSS removal mechanisms,
- Nitrogen removal efficiency,
- Vegetation selection and management,
- Construction details, and
- Cost.

a) Hydraulic and hydrological conditions

Hydraulic under-design and bed clogging can result in partial surface flow. Bed clogging is a risk if the primary treatment process is unable to settle out inorganic loads.

Whilst Darcy's Law has been found to give acceptable hydraulic design results for moderate sized gravel beds, providing the system is designed to depend on a minimal hydraulic gradient, and if the Q in equation is taken as the 'average' flow, pilot trials would be necessary where the use of soils were intended. That is the flow per unit time equals the hydraulic conductivity of a unit area of the medium perpendicular to the flow direction times the total cross-sectional area, perpendicular to flow times the hydraulic gradient of the water surface in the flow system.

b) BOD and TSS removal mechanisms

BOD removal has been approximated by a first order plug-flow equation where the ratio of effluent BOD to influent BOD is equal to 'e' raised to a power of ($-K_T \times t$) where 'K_T' is a temperature dependant rate constant and 't' is the residence time in days. When compared to facultative ponds and open water surface wetlands the applicable rate constant has been estimated at better than 2 times and 9 times respectively.

TSS removal is reportedly very effective with almost all of this occurring in the first day. Whilst a kinetic design model is not available for TSS removal, based on the relationships established, it follows the same pattern as BOD_5, and may be considered as comparable as long as subsurface flow is maintained in the bed.

c) Nitrogen and Phosphorous removal efficiency

Biological nitrification followed by denitrification is believed to be the major pathway for ammonia removal in both types of constructed wetlands, as they are presently operated

Based on results for beds planted with reeds and sedges in USA and Europe, Nitrogen removal is not good and this is believed to be a result of the very limited aerobic areas formed in the vicinity of the root systems in an otherwise aerobic environment. Phosphorous removal has been similarly disappointing. Phosphorus removal in most constructed wetland systems is not very effective probably because of the limited contact opportunities between the wastewater and the soil.

No formulas have been reported for Nitrogen and Phosphorous removal, however the evidence available indicates that removal is significantly better with vetiver but again pilot trials would presently be necessary until there is more information on its efficacy. Given the nature and extent of the root system as illustrated in Figure 10.10, a significantly greater aerobic zone is present that probably accounts for this.

d) Vegetation selection and management

As discussed, there is increasing evidence that vetiver grass rather than cattails, reeds and sedges is the better plant and possibly the best plant available as it is also far less invasive and thus much easier to manage.

e) Construction details

The aspect ratio (L:W) of the wetland bed is a very important consideration in the hydraulic design of SSF wetland systems, since the maximum potential hydraulic gradient is related to the available depth of the bed divided by the length of the flow path. A range of between 0.4:1 and 3:1 is usually selected.

Slope up to 10 % have been used and this is also very important as the combination of aspect ratio and slope determines the possible load.

The depth of media selected will depend on the design intentions for the system. If the vegetation is intended as a major oxygen source for nitrification in the system, then the depth of the bed should not exceed the potential root penetration depth for the plant species to be used. This will ensure availability of some oxygen throughout the bed profile, but may require management practices which assure root penetration to these depths. Given that the root depths of for cattails and sedges may be no more than 30 cm and for reeds tends to be no more than 60 cm, this is then a very real limitation compared with vetiver grass where root depths can reach 3 metres or more.

In addition to the internal hydraulic concerns above, it is necessary to have adequate inlet and outlet structures for the bed to assure proper distribution and collection of flow and maximum utilization of the media provided in the bed.

A surface inlet manifold, with adjustable outlets, the full width of the bed is seen as providing the maximum flexibility for future adjustments and maintenance and is recommended. Coarse rock (8 to 15 cm) should be used in this entry zone, and when coupled with an adequate hydraulic gradient for the bed, should ensure rapid infiltration and prevent ponding and algae development. In continuously warm and sunny climates, shading of this entry zone with either vegetation or a structure may also be necessary.

The use of a full width perforated subsurface manifold connected to an adjustable outlet is said to offer the maximum flexibility and reliability as the outlet device for SSF wetland systems. Initially, the surface of the bed can be flooded to encourage, development of newly planted vegetation and to suppress undesirable weeds. However, since the manifold is buried and inaccessible following construction, careful grading is required during construction and clean-out risers should be provided in the line.

f) Cost

As yet there is insufficient information to provide firm cost data for SSF constructed wetlands using soils rather than gravels and vetiver rather than cattails, reeds and sedges.

g) Design

Reference can be made to the documents indicated in Section 10.8.2.5 below but as the development of SSF systems is still ongoing, designers should also seek for more up-to-date publications, especially from Australia.

10.8.2.4 General Construction Guidelines

a) Construction Practices/Specifications/Drawings

Good construction practices should be followed during construction of the wetland. Examples include properly evaluating the site, limiting damage to the local landscape by minimizing excavation and surface runoff during construction, and maximizing flexibility of the system to adapt to extreme conditions. Construction specifications and drawings should be produced that clearly convey procedures to be used and the required quality of the final product.

b) Soils

If possible, avoid soil sources that contain a seed bank of unwanted species. Carefully consider the soils permeability and the implications for groundwater protection. Highly permeable soils may allow infiltration and possible contamination of groundwater and could prevent the development of hydrological conditions suitable to support wetland vegetation. It may be necessary to use an impermeable barrier in some instances.

c) Vegetation Selection

In general, a diversity of native, locally obtained species should be used. Whenever possible, avoid harvesting native plants from existing wetlands. Species should be chosen both for water quality and wildlife habitat functions, if that is the intent of the project. The use of weedy, invasive, or non-native species should be avoided where possible and where not ones that do not propagate by flower and seeds should be selected. For subsurface constructed wetlands, vetiver grass is however acceptable. It is also necessary to consider the plants' abilities to adapt to various water depths and soil and light conditions at the site.

10.8.2.5 Operation and Maintenance Guidelines

a) Management Plan

A long-term operations, maintenance, monitoring, and funding plan should be drawn up, which identifies the party or parties responsible for maintenance and monitoring of the project, their responsibilities, and the funding mechanisms. The information obtained should be reported annually to the Ministry of Water to assist in improving these Guideline principles in light of national experience.

b) Regular Inspections and Maintenance Activities

The Plan should require regular inspections of the constructed wetland. The definition of "regular" is case-specific and will depend on the design and operation of the treatment wetland. These considerations should be described in the maintenance plan. Examples of maintenance activities that should be conducted during these inspections include checking weir settings and the inlet and outlet structures, cleaning off surfaces where solids and floatable substances have accumulated to the extent that they may block flows, removing nuisance species and maintaining the appearance and general status of the vegetation and wildlife populations, and removing sediment accumulations in settling basins. Time and energy can be saved by conducting the routine monitoring activities, such as sample collections and wildlife counts, at the same time as inspections.

c) Operator Training

Operators should be trained and/or certified in the operation and maintenance of constructed wetlands.

d) Contingency Plan

Project designers and the Wetland Operators should jointly develop a contingency plan to address problems which could develop during facility operations. Such problems may be due to: unrealistic or unattainable goals; design, construction, operational errors; or unpredictable events. The first situation can be addressed by revising project goals or regulatory criteria (e.g. water quality standards), the second by reducing system capacity, increasing its area, or operational changes, and the third by anticipation through conservative design. Contingency plans should include measures for determining and remedying nuisance conditions, addressing any toxicity observed in the wetland, and dealing with upstream treatment plant failure or bypass. Auxiliary storage basins can be helpful for dealing with many of these situations should the need arise.

10.8.2.6 Monitoring

a) Reference Wetland

If present in the region, a reference site may be useful as a basis of comparison to identify various changes and impacts to the constructed wetland ecology and to evaluate its success. Where more than one is available, select that which has the more common features (e.g., depressional, riverine), class, size, vegetative cover, climatic and geographic region (preferably nearby and within the same watershed), while allowing for natural variability, as a reference to measure the success of the project.

b) Methods and Criteria

Depending on the primary goals and objectives of the project, site monitoring can be used to determine the chemical, physical, and biological health of the wetland and its success in treating effluent. Monitoring criteria may include water quality (surface and groundwater), sediment quality, temperature, hydrology (fluctuation, loading, variability and flow pattern monitoring by means of tracer studies), plant, benthic macro-invertebrate, fish tissue analyses, toxicity testing, seasonal vegetation mapping or physical sampling, habitat structure and diversity (including species richness), and wildlife use surveys (birds, amphibians, macro-invertebrates, and fish, if appropriate).

Also, nuisance insects should be monitored to evaluate the need for vector control measures. Where appropriate, methods for monitoring should draw from the scientific literature for assessing biological conditions. The specific details of the monitoring plan should be determined through discussions with the Ministry of Water.

c) Early Identification of Potential Problems

Potential problems should be anticipated as far as possible and monitoring should look out for potential dangers to the wetland ecosystem, such as bioaccumulation, avian botulism and other avian diseases, vector problems, invasion of non-native plants and animals, debris accumulation, and nuisance conditions, and be such that the it is practicable to respond quickly. Potential responses to any such problems should be described in the contingency plan.

d) Timeframe

Preferably the constructed treatment wetland should be monitored for the entire life of the project to help ensure that the wetland system performs as designed and its ecological integrity is maintained.

10.8.2.7 Design Sources

A recommended starting point for designers for both open-surface flow and SSF gravel bed constructed wetlands is the US EPA design manual 'Constructed Wetlands Treatment of Municipal Wastewaters', published in 2000. A useful background, if somewhat theoretical document is 'Waste Stabilization Ponds and Constructed Wetlands – Design Manual' , resulting from a research project under the auspices of the Danish University of Pharmaceutical Sciences, and the University of Dar Es Salaam, Tanzania. It is obtainable from the UNEP website
<www.or.jp/Ietc/Publications/Water_Sanitation/ponds_and_wetlands/Design_Manual.pdf >.

Contact should also be made to the University of Dar es Salaam itself at <www.udsm.ac.tz/faculty/foe/wetlands/> for the latest information on research and development in this field in Tanzania.

Another reference worth reviewing for the latest information available from the USA is EPA's 'North American Treatment Wetland Database' which is regularly updated.

For information on constructed wetlands using vetiver grass, the site <www.vetiver.org> should be consulted as progressively more useful information and references can be expected there on the use of vetiver grass in constructed wetlands.

APPENDIX 10.1

ENVIRONMENTAL IMPACT OF SEWAGE TREATMENT PLANTS

This Annex reproduces, a checklist for the Environmental Impact Assessment of sewage treatment plants such as WSP systems. Section 4.3 and 4.4 of the (UNEP Regional Seas Reports and Studies No. 112).

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

A. Description of the proposed project.

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

(a) Types of sewage to be treated (industrial, domestic, agricultural).

(b) Number of inhabitants to be served by the plant.

(c) Types of clients to be served, e.g. industrial, residential, commercial, hospitals.

(d) Quantity of sewage (cubic metres per day or per year).

(e) Quality of sewage to be treated, including suspended solids (mg/1), settleable solids (mg/1), pH, turbidity, conductivity, BOD (mg/1), COD (mg/l), nitrogen, ammonia phosphate, oil, surfactants, and heavy metals such as arsenic, cadmium, copper, lead, nickel and mercury.

(f) Method to be used in treatment of sewage.

(g) Layout of the plant (including treatment facilities and service area).

(h) Use of effluent (agriculture, recharging aquifer, disposal to sea or to nearest river).

(i) Description of the plant's recipient body of water, if any.

(j) Sludge quantity and quality.

(k) Method of sludge treatment and disposal.

(l) Chemical, physical and bacteriological characteristics of effluent such as suspended solids, settleable solids, pH, turbidity, conductivity, BOD, COD, nitrogen, ammonia, phosphate, oil, surfactants, and heavy metals such as arsenic, cadmium, copper, lead, nickel and mercury, total coliforms, faecal coliforms and faecal streptococci.

(m) Programme for operation and maintenance of the sewage treatment plant.

B. Reasons for selecting the proposed site and the technologies.

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

C. Description of the Environment

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

(a) Physical site characteristics.

   (i) Site location on a map at a scale of 1:10,000 or 1:50,000 including residential areas, industrial areas and access roads.

(b) Climatological and meteorological conditions.

   (i) Basic meteorological data such as wind direction and wind velocity.

   (ii) Special climatic conditions such as storms, inversions, trapping and fumigation, proximity to seashore, average yearly rainfall and number of rainy days per year.

   (iii) Existing sources of air pollution, especially of particulates and odorous.

(c) Geological and hydrological conditions

   (i) Geological structure of proposed area, including hydrology and aquifers.

   (ii) Existing uses of water bodies around the proposed site and the quality of the water.

(d) Present land use of the site and its surroundings.

(e) Characteristics of sea area which will be recipient of discharged treated sewage.

   (i) Sea circulation, existence and characteristics of the thermocline, thermohaline structure, dissolved oxygen and nutrients concentration, microbial pollution, fishing grounds, aquaculture sites, marine habitats.

(f) Existence of endemic water borne diseases.

D. Identification of possible impacts

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

(a) Odorous and air pollution from the plant and from the disposal of effluents and sludge.

(b) Infiltration of sewage into topsoil, aquifer or water supply and impact on drinking water quality.

(c) Mosquito breeding and diseases transmitted by mosquitoes.

(d) Pollution of water bodies such as rivers, lakes or sea by effluents and impact on bathing water quality.

(e) Flora and fauna.

(f) Fruit and vegetable safety, if land disposal of effluent or sludge.

(g) Noise levels around plant and its sources.

(h) Solid waste disposal of sludge other wastes.

(i) Devaluation of property values.

(j) Tourist and recreation areas such as natural reserves, forests, parks, monuments, sports centres, beaches, and other open areas which could be impacted.

(k) Possibly emergencies and plant failure, the frequently at which they may occur, and possible consequences of such emergencies.

(l) Anticipated or foreseeable impact on the area outside of national jurisdiction.

E. Proposed measures to prevent, reduce or mitigate the negative effects of the proposed plant.

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

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

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

Section A

Item (e) are less extensive, as follows:

(e) Quality of sewage to be treated, including suspended solids (mg/1), settleable solids (mg/1), pH, turbidity, conductivity, BOD (mg/1), COD (mg/litre), nitrogen, and oil.

(l) Chemical, physical and bacteriological characteristics of effluent such as suspended solids, settleable solids, pH, turbidity, BOD, COD, nitrogen and oil.

Section C

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

Section D

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

APPENDIX 10.2

WASTE STABILISATION POND DESIGN EXAMPLES

1. **Constant Population**

Design a WSP system to treat 10,000 m^3/d of a wastewater which has a BOD of 350 mg/1 and 1×10^8 FC per 100 ml. The effluent should contain no more than 1000 FC per 100 ml, and the design temperature and evaporation rate are 18ºC and 6 mm/d, respectively.

Solution:

(a) Anaerobic ponds

From Table 10.4 the design loading is given by:

$$v = 20T - 100 = (20 \times 18)\text{-}100 = 260 \text{ g/m}^3\text{/d}$$

The pond volume is given by equation 10.2 as:

$$V_a = L_i \times Q/v = 350 \times 10{,}000/260 = 13{,}462 \text{ m}^3$$

The retention time is given by equation 10.3 as:

$$\Theta_a = V_a/Q = 13{,}462/10{,}000 = 1.35 \text{ d}$$

The BOD removal is given in Table 10.4 as:

$$R = 2T + 20 = (2 \times 18) + 20 = 56 \text{ percent}$$

(b) Facultative ponds

The design loading is given by equation 10.5 as:

$$s = 350\,(1.107 - 0.002T)^{T-25}$$

$$= 350\,[1.107\text{-}(0.002 \times 18)^{18-25} = 216 \text{ kg/ha d}$$

Thus the area is given by a re-arranged equation 10.4 as:

$$A_f = 10 \times L_i \times Q/s = 10 \times 0.44 \times 350 \times 10{,}000/216 = 71{,}300 \text{ m}^2$$

The retention time is given by the equation 10.7 as:

$$\Theta_f = 2 \times A_f \times D/(2Q - 0.001\,A_f \times e)$$

Taking a depth of 1.5m, this becomes:

$$\Theta_f = 2 \times 71{,}300 \times 1.5\,[(2 \times 10{,}000) - (0.001 \times 71{,}300 \times 6)] = 10.9 \text{ days}$$

The effluent flow is given by:

$$Q_e = Q_i - 0.001\,A_f e = 10{,}000 - (0.001 \times 71{,}300 \times 6) = 9572 \text{m}^3\text{/d}$$

(c) Maturation ponds

For 18ºC the value of K_T can be calculated from equation 10.11:

$$K_T = 2.6\,(1.19)^{T-20} = 1.84 \text{ d}^{-1}$$

Equation 10.10 can be rearranged as follows:

$$\Theta_m = \{[N_i \times N_e\,(1 + K_T \times \Theta_a)\,(1 + K_T\ \Theta_f)^{1/m} - 1\}/K_T$$

$= \{[10^8/10^3 (1 + (1.84 \times 1.35)(1 + (1.84 \times 10.9)]^{1/m} - 1\}/1.84$

$= 74.0$ d for $n = 1$

$= 19.5$ d for $n = 2$

$= 5.5$ d for $n = 3$

$= 2.8$ d for $n = 4$

The first two combinations of Θ_m and n are rejected as $\Theta_m > \Theta_f$. The fourth combination is also rejected as $\Theta_m < \Theta_m^{min} = 3$ d, and $n = 4$, and the latter has a smaller product (11.2) than the former (16.5), which is therefore chosen.

Check the loading on the first maturation pond from equation 10.13:

$$s(ml) = 10 \times 0.03 \times 350 \times 1.5/3 = 525 \text{ kg/ha d}$$

The value is higher than 75 percent of the load on the facultative pond (= 0.75 × 216, = 162 kg/ha d). Thus s(ml) is taken as 162 kg/ha d and Θ_{ml} calculated from:

$$\Theta_{ml} = 10\, L_i D/ \; ml = 10 \times 0.03 \times 350 \times 1.5/162 = 9.7 \text{ d}$$

The retention times in the subsequent maturation ponds are calculated from:

$$\Theta_m = \{[N_i\, N_e\, (1 + K_T \times \Theta_a)\, (1 + K_T \times \Theta_f)](\, 1 + K_T \times \Theta_{ml})]^{1/n} - 1\}/K_T$$

$$= \{\, [\, 10^8/10^3\, (1 + 1.84 \times 1.35)]\, (1 + 1.84 \times 10.9)\, (1 +(1.84 \times 9.7)]^{1/n} - 1\}/1.84$$

$= 39$ d for $n = 1$

$= 4.1$ d for $n = 2$

The second combination is chosen as its product is 8.2, which is less than that for Θ_m

$= \quad \Theta_m \text{ min} = 3$ d and $n = 3$

For a depth of 1.5m, the area of the first maturation is given by equation 10.14 as:

$$A_{ml} = 2Q_i \times \Theta_m/(2D + 0.001e\, \Theta_m)$$

$$= 2 \times 9572 \times 9.7/[(2 \times 1.5) + (0.001 \times 6 \times 9.7)] = 60.721 m^2$$

The effluent flow is given by:

$$Q_e = Q_i - 0.991 A_{ml} e = 9572 - 0.001 \times 60{,}721 \times 6) = 9208 m^3/d$$

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

$$A_{ml} = 2 \times 9208 \times 4.1/(2 \times 1.5) + (0.001 \times 6 \times 4.1)] = 24{,}964 m^2$$

$$Q_e = 9208 - (0.001 \times 24{,}964 \times 6) = 9058 m^3/d$$

And the third maturation pond:

$$A_{m3} = 2 \times 9058 \times 4.1/(2 \times 1.5) + (0.001 \times 6 \times 4.1)] = 24{,}557 m^2$$

$$Q_e = 9058 - (0.001 \times 24{,}557 \times 6) = 8911 m^3/d$$

BOD Removal

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

$$350 \times 0.1 \times 0.75 \times 0.75 \times 0.75 = 15\text{mg/1, which is satisfactory.}$$

Summary

The design thus comprises:

Anaerobic pond(s)	:	volume 13,462 m^3
retention time		1.35 d
Facultative pond(s)	:	area 71,300 m^3
retention time		10.9 d
First maturation pond(s)	:	area 60,721 m^2
Second maturation pond(s)	:	area 24,964 m^2
Third maturation pond(s)	:	area 24,557 m^2
retention time		4.1 d

The overall retention time is thus 30.14 days, and the removal of filtered BOD and FC throughout the pond series is expected to be as follows:

	BOD (mg/1)	FC (per 100ml)
Raw wastewater	350*	1.0×10^8
Anaerobic pond effluent	154*	2.9×10^7
Facultative pond effluent	35	1.4×10^8
1st maturation pond effluent	26	7.2×10^6
2nd maturation pond effluent	20	8.5×10^2
3rd maturation pond effluent	15	9.9×10^2

*unfiltered BOD

The effluent flow is 8,911 m^3/d, so evaporative losses are 10.9 percent.

Note: If the above design were done without anaerobic ponds, the result would be a primary facultative pond and four maturation ponds, as follows:

Facultative pond(s)	:	area 162,037 m^2
retention		25.5 d
First maturation pond(s)	:	area 57,270 m^2
retention time		9.7 d
Second maturation pond(s)	:	area 17,057 m^2
retention time		3 d

Third maturation pond(s) : area 16,854 m^2

retention time 3 d

The overall retention time is thus 44.2 days, which is 46.6 percent greater than when anaerobic ponds are included and evaporative losses are 16.2 percent, which is 48.6 percent more than the design with anaerobic ponds. This clearly shows the advantages of including anaerobic ponds as their inclusion substantially reduces retention times, and thus land area requirements, and also losses due to evaporation (which is important if the effluent is to be used for crop irrigation).

2. **Seasonally Varying Population**

The design is to be the same as for design example no.1 but, due to seasonal effects of tourism, the cool and hot season flows, temperatures and evaporation rates are as follows:

Cool season: 10,000 m^3/d @ 18ºC 6 mm/d

Hot season: 30,000 m^3/d @ 28ºC 11 mm/d

Solution

(a) Anaerobic ponds

Design the anaerobic pond for the hot season and compare this with the cool season design given in design example No. 1.

For 28ºC, v = 300 g/m^3d. Therefore:

$$V_a = 350 \times 30{,}000/300 = 35{,}000 \text{ m}^3$$

This V_a is higher than that for 18ºC and is therefore chosen.

$$\Theta_a = 35{,}000/30{,}000 = 1.17 \text{ d (hot season)}$$

$$= 35{,}000/10{,}000 = 3.5 \text{ d (cool season)}$$

$$= 350 \times 10{,}000/35{,}000 = 100\text{g/m}^3\text{d (cool season) which is satisfactory}$$

(b) Facultative ponds

For 28ºC, the permissible surface BOD loading is given by:

$$= 350\{1.107-(0.002 \times 28)]^{28-25} = 406 \text{ kg/ha d}$$

The BOD removal in the anaerobic pond is 60 percent at 28ºC. Therefore

$$A_f = 10 \times 0.4 \times 350 \times 30{,}000/406 = 103{,}448 \text{ m}^2$$

This is greater than that required in cool season, and therefore chosen.

$$\Theta_f = 2 \times 103{,}488 \times 1.5/[2 \times 30{,}000) - (0.001 \times 103{,}488 \times 11)] = 5.3 \text{ d (hot season)}$$

$$= 2 \times 103{,}488 \times 1.5/[2 \times 10{,}000) - (0.001 \times 103{,}488 \times 6)] = 16.0 \text{ d (cool season)}$$

$$Q_e = 30{,}000 - (0.001 \times 103{,}488 \times 11) = 28{,}862 \text{ (hot season)}$$

$$= 10{,}000 - (0.001 \times 103{,}488 \times 6) = 9{,}379 \text{ m}^3\text{/day (cool season)}$$

(c) Maturation ponds

These will be designed first for the hot season and then checked for performance in the cool season.

At 28ºC, K_T is given by:

$$K_T = 2.6(1.19)^{28-20}$$

$$\Theta_m = \{[10^8/10^3(1 + (10.5 \times 1.17)(1 + (10.5 \times 5.3))]^{1/n} - 1\}/10.5$$

$$= 12.6 \quad \text{for} \quad n = 1$$

$$= 1.0 \quad \text{for} \quad n = 2$$

Consider 2 ponds, each with a retention time of 3 d ($=\theta_m{}^{min}$) and then calculate the loading on the first:

$$s(min) = 10 \times 0.3 \times 350 \times 1.5/3 = 525 \text{ kg/ha d}$$

Which is higher than (0.75 × 406), =304kg/ha d. Thus the retention time in the first maturation pond is:

$$\theta_{ml} = 10 \times 0.3 \times 350 \times 1.5/304 = 5.2 \text{ d}$$

The retention time is the subsequent maturation ponds(s) is:

$$\theta_m = \{10^8/10^3 (1+(10.5 \times 1.17)(1+10.5 \times 5.3)(1+(10.5 \times 5.2)]^{1/n}-1)\}/10.5$$

$$= 0.13 \text{ d} \quad \text{for} \quad n = 1$$

So choose a single pond with a 3 d retention time.

Check performance in the cool season (when the retention in the maturation ponds will be three times greater):

$$N_e = 10^8/[1+(1.84 \times 3.5)(1+(1.84 \times 16)(1+(1.84 \times 15.6)(1+(1.84 \times 9)]$$

$$= 847 \text{ per 100ml, which is satisfactory.}$$

The maturation ponds areas are now calculated:

$$A_{ml} = 2 \times 28{,}862 \times 5.2/[(2 \times 1.5)+(0.001 \times 115.2)] = 98{,}183\text{m}^2$$

$$Q_e = 28{,}862 - (0.001 \times 98{,}183 \times 11) = 27{,}782\text{m}^3\text{/d}$$

$$A_{m2} = 2 \times 27{,}782 \times 3/[(2 \times 1.5)+(0.001 \times 11 \times 3)] = 54{,}960\text{m}^2$$

$$Q_e = 27{,}782 - (0.001 \times 54{,}960 \times 11) = 27{,}177\text{m}^3\text{/d}$$

Calculate actual retention times in the cool season:

$$\Theta_{ml} = 98{,}183 \times 1.5/9{,}379 = 15.7 \text{ d}$$

$$Q_e = 9{,}379 - (0.001 \times 98{,}186 \times 6) = 8790 \text{ m}^3\text{/d}$$

$$\Theta_{m2} = 54{,}960 \times 1.5/8790 = 9.4 \text{ d}$$

$$Q_e = 8790 - (0.001 \times 57{,}960 \times 6) = 8460 \text{ m}^3\text{/d}$$

The retention times are slightly greater than those assumed above in the check on the cool season performance, so N_e will be slightly less than 847 per 100 ml but still satisfactory.

Summary

The design thus comprises:

Anaerobic pond(s)	:	volume	35,000 m^3
		retention time	1.17 d (3.5 d)*
Facultative pond(s)	:	area	103,448 m^2
		retention time	5.3 d (16.0 d)
First maturation pond(s)	:	area	98,183m^2
		retention time	5.2 d (15.7 d)
Second maturation pond(s)	:	area	54,960 m^2
		retention time	3 d (9.4 d)

* Cool season retention times given in parentheses.

Thus the overall retention time is 14.7 d in the hot season and 44.6 d in the cool season.

3. **For Unrestricted Irrigation Use**

The design is to be the same as designed example No.1, except that the effluent is to be used for unrestricted irrigation only in those months in which the temperature is above 25ºC (when the evaporation rate is 9 mm/d).

Solution

(a) Anaerobic and facultative ponds

These must operate satisfactorily at all times and therefore they have to be designed for 18ºC; so they are as in design example No.1. The retention time in the facultative pond when the evaporation rate is 9mm/d is given by:

$\Theta_f = 2 \times 71{,}300 \times 1.5/[(2 \times 10{,}000) - (0.001 \times 71{,}300 \times 9)] = 11.0$ d,
and the corresponding effluent flow from the facultative pond is:

$\Theta_e = 10{,}000 - (0.001 \times 71{,}300 \times 9) = 9358 m^3/d$

(b) Maturation ponds

The value of K_T at 25ºC = $2.6(1.19)^{25-20} = 6.20 d^{-1}$

The Retention time is:

$$\Theta_m = \{[10^8/10^3(1+(6.2 \times 1.35)(1+(6.2 \times 11)]^{1/n}-1\}/6.2$$
$$= 24.7 \text{ d for } n = 1$$
$$= 1.8 \text{ d for } n = 2$$

Choose 2 ponds each with $\Theta_m = 3$ ($= \Theta_m^{\ min}$) and check loading on first pond:

$s(ml) = 10 \times 0.3 \times 350 \times 1.5/3 = 1675$ kg/ha d

This is greater than 75 percent of the permissible loading on facultative, which at 25ºC is (0.75 × 350), = 262kg/ha d. Thus the retention time in the first maturation pond is calculated from:

$\Theta_{ml} = 10 \times 0.3 \times 350 \times 1.5/262 = 6.0$ d

The retention time in the subsequent maturation pond(s) is:

$\Theta_{ml} = \{10^8/10^3(1+(6.2 \times 1.35)(1+(6.2 \times 11)(1+(6.2 \times 6)]^{i/n}-1\}/6.2$

So choose a single secondary maturation pond with a retention time of 3 d (= $\Theta_m{}^{min}$)

The maturation pond areas are calculated as follows:

$A_{ml} = 2 \times 9358 \times 6/[(2 \times 1.5)+(0.001 \times 9 \times 6)] = 36{,}770$ m^2

$Q_e = 9358 = (0.001 \times 36{,}770) = 9{,}027$ m^3/d

$A_{m2} = 2 \times 9027 \times 3/\{2 \times 1.5)+(0.001 \times 9 \times 3)] = 17{,}893$m^2

$Q_e = 9027 – (0.001 \times 17{,}893 \times 9) = 8{,}866$ m^3/day

Summary

The design thus comprises:

Anaerobic pond(s)	:	volume	13,462 m^3
		retention time	1.35 d
Facultative pond(s)	:	area	71,300 m^2
		retention time	11 d
First maturation pond(s)	:	area	36,770 m^2
		retention time	6 d
Second maturation pond(s)	:	area	17,893 m^2
		retention time	3 d

Thus overall retention time is 21.35 d, which is 30 percent less than in design example No. 1. This illustrates the advantage of designing a pond system to produce an effluent containing > 1000 FC per 100 ml only in those months when it will be actually used for unrestricted crop irrigation.

4. **Restricted Irrigation**

The design is to be the same as design example No.3 except that the effluent is to be used for restricted irrigation. The raw wastewater contains 600 human intestinal nematode eggs per litre.

Solution

(a) Anaerobic pond

As in design example No. 3, Θ_a will be 1.35 d. For this retention time, the percentage egg removal is given by equation 4.19 as:

$R = 100[1 – 0.4 \exp(- 0.49\ \Theta_a + 0.0085\ \Theta_a{}^2)]$

$= 100[- 0.41 \exp(- 0.49 \times 1.35) + (0.0085 \times 1.8225)] = 78.5$ percent

So the number of eggs per litre of anaerobic pond effluent is (600 × 0.215) = 129

(b) Facultative pond

Θ_f = 10.9 d, so R is given by:

$$R = 100[1 - 0.41 \exp(-0.49 \times 10.9) + (0.0085 \times 118.81)] = 99.4 \text{ percent}$$

So the number of eggs per litre of facultative pond effluent is (0.006 × 129), = 0.8. Thus no further treatment is necessary as the facultative pond effluent complies with the WHO guideline value of > 1 egg per litre (Table 10.1).

Suppose, however, that the facultative pond effluent contained 10 eggs per litre. A maturation pond would then be required to reduce the number to 1. This is equivalent to an egg removal of 90 percent. From Table 4.4 this requires a retention time of 3.1 days, and so a single maturation pond of this size would be provided.

5. **Partially aerated facultative pond**

The WSP system designed in design example No. 1 is now receiving twice its design flow. The anaerobic ponds are to be duplicated and floating aerators installed on the facultative pond. What is the required aerator power input? The temperature of the anaerobic pond effluent in the coldest month is 24ºC.

Solution

The extended anaerobic ponds will achieve the same BOD removal as before (56 percent). So the influent to the facultative ponds still has a BOD of 154 mg/l.

The current retention time in the facultative ponds is half that in the original design, i.e. 5.45 d.

The temperature T is the temperature of the partially aerated pond and is given by the equation:

$$T = Ta + (T_o - T_a)/3 = 18 + (24 - 18)/3 = 20ºC$$

The required retention time in the partially aerated facultative pond for 70 percent BOD removal is:

$$\Theta = 8.45\ (1.036)^{20-T}$$

For T - 20ºC, Θ = 8.45 d. This is greater than what is available (5.45 d) and so the BOD removal is given by:

$$L_c = L_i\ (1 + K_{pm(T)}\Theta) = 154/[1 + (0.276 \times 5.45)] = 62 mg/l$$

Thus the BOD removal is 60 percent, and the oxygen requirement is given as:

$$R = 1.5 \times 10^{-3}\ (L_i - L_c)\ Q/24 = 1.5 \times 10^{-3}\ (154 - 62)20{,}000/24 = 155 kg/hr$$

Assume that the manufacturer's rating for aerators (no.) is 2 kg 0_2/kWh. This is corrected for field conditions (assuming the pond system is at seal level):

$$N = N \text{ x } [(1.024)^{T-20}]\ [(\beta C_{S(T,A)} - C_L)/C_{S(20,0)}]$$

$$= 2 \times 0.7 \times \{(0.9 \times 9.08) - 2]/9.08\} = 0.95 kg\ 0_2/kWh$$

The aerator power requirement (P) is given by the equation:

$$P = R/N_f = 115/095 = 121 \text{ kW}$$

As the area of the facultative ponds is large (71,300 m^2. Four ponds, say, each of 17,825 m^2 or 77 m × 231 m, would be realistic requiring twelve 10 kW aerators (three per pond).

6. **Septage ponds**

Design a WSP system to receive septage from fifty 5,000-litre tankers a day. The septage BOD and FC count are 5000mg/l and 1×10^8 per 100ml respectively, and the design temperature and evaporation rate are 20 ºC and 6 mm/d.

Solution

Facultative pond

The daily BOD load (L, kg/d) is calculated from:

$$L = 10^{-3} L_i\, Q = 10^{-3} \times 5000 \times (50 \times 5000 \times 10^{-3})$$

$$= 1250\text{kg/d}$$

The permissible BOD surface loading at 20 ºC is 350 kg/ha d, so the facultative pond area (A_f, ha) is given by:

$$A_f = 1250/350 = 3.56 \text{ ha}$$

This area now compared with that calculated on the basis of evaporative losses equalling the septage inflow:

$$A_f e = Q$$

So that: $A_f = 250/(0.001 \times 6) = 41{,}700 \text{ m}^2$

This is greater than that based on the permissible loading and so adopted. In practice one would have say, four ponds in parallel. Make-up water is added to each pond in those months when there is positive net evaporation (e, mm/d) at the following rate (Q_{m9} m^3/d):

$$Q_m = (0.001 \times 10{,}425 \times e) - 250/4$$

APPENDIX 10.3

EXAMPLE OF A CONSTRUCTED WETLAND

The Splash constructed wetland consists of a combination of a sub-surface horizontal flow system (VSB) planted with *Typha*, followed by a series of three pond systems planted with a variety of plants, including ornamental plants. The plant species include *Typha*, *Cyperus latifolius*, *Cyperus papyrus*, *Hydrocotyle*, *Hydrocleis* and *Pontederia*.

The performance of this constructed wetland was examined for four months (Dec' 1995 to Mar' 1996). The study area is approximately 0•5 ha, and is located in the southern part of Nairobi city. Splash wetland continuously receives domestic sewage from two busy restaurants. Treated wastewater is recycled for re-use for various purposes in the restaurants. Both wet and dry season data were analysed with a view of determining the impact of seasonal variation on the system performance.

The physical and chemical properties of water were measured at a common intake and at series of seven other points established along the wetland gradient and at the outlet where the water is collected and pumped for re-use at the restaurants. The physico-chemical characteristics of the wastewater changed significantly as the wastewater flowed through the respective wetland cells.

A comparison of wastewater influent versus the effluent from the wetland revealed the system's apparent success in water treatment, especially in pH modification, removal of suspended solids, organic load and nutrients mean influent pH = 5.7 ± 0.5, mean effluent pH 7.7 ± 0.3; mean influent BOD_5 = 1603.0 ± 397.6 mg/l, mean effluent BOD_5 = 15.1 ± 2.5 mg/l; mean influent COD = 3749.8 ± 206.8 mg/l, mean effluent COD = 95.6 ± 7•2 mg/l; mean influent TSS = 195.4 ± 58.7 mg/l, mean effluent TSS = 4.7 ± 1.9 mg/l.

As the wastewater flows through the wetland system dissolved free and saline ammonia, NH_4+, decreased from 14.6 ± 4.1 mg/l to undetectable levels at the outlet. Dissolved oxygen increased progressively through the wetland system. Analysis of the data available did not reveal temporal variation in the system's performance. However, significant spatial variation was evident as the wetland removed most of the common pollutants and considerably improved the quality of the water, making it safe for re-use at the restaurants for watering.

The wetland is designed to treat wastewater as well as to provide an aesthetically pleasing and environmentally sensitive landscape with ponds and ornamental plants for recreation. The 0.5 ha wetland is designed for 1,200 population equivalents, and has a high treatment efficiencies for BOD_5, SS, COD, faecal coliforms, Kj-N, NH4-N and o-PO_4 of 98, 85, 96, 99, 90, 92 and 88% removal. Most of the nitrogen and phosphorus is deposited in the soil of both the subsurface horizontal flow constructed wetland and the ponds. Some nitrogen is removed by denitrification, only 6% of it is retained by the plants. The sediments play therefore, a major role in the removal of the nutrients. The constructed wetland attracts many birds (128 bird species have been reported) as well as amphibians. The overall impression of the wetland is that the dual function of beauty and wastewater treatment is achieved.