UNIT IV

UASB Units

UASB type units are one in which no special media have to be used since the sludge granules themselves act as the 'media' and stay in suspension. UASB system

is not patented. A typical arrangement of a UASB type treatment plant for municipal sewage would be as follows:

1. Initial pumping
2. Screening and degritting
3. Main UASB reactor
4. Gas collection and conversion or conveyance
5. Sludge drying bed
6. Post treatment facility

In the UASB process, the whole waste is passed through the anaerobic reactor in an upflow mode, with a hydraulic retention time (HRT) of only about 8-10
hours at average flow. No prior sedimentation is required. The anaerobic unit does not need to be filled with stones or any other media; the upflowing sewage
itself forms millions of small "granules" or particles of sludge which are held in suspension and provide a large surface area on which organic matter can
attach and undergo biodegradation. A high solid retention time (SRT) of 30-50 or more days occurs within the unit. No mixers or aerators are required. The
gas produced can be collected and used if desired. Anaerobic systems function satisfactorily when temperatures inside the reactor are above 18-20°C. Excess
sludge is removed from time to time through a separate pipe and sent to a simple sand bed for drying.

Design Approach

Size of Reactor: Generally, UASBs are considered where temperature in the reactors will be above 20°C. At equilibrium condition, sludge withdrawn has to be

equal to sludge produced daily. The sludge produced daily depends on the characteristics of the raw wastewater since it is the sum total of (i) the new VSS

produced as a result of BOD removal, the yield coefficient being assumed as 0.1 g VSS/ g BOD removed, (ii) the non-degradable residue of the VSS coming in

the inflow assuming 40% of the VSS are degraded and residue is 60%, and (iii) Ash received in the inflow, namely TSS-VSS mg/l. Thus, at steady state

conditions,

SRT= Total sludge present in reactor, kg Sludge withdrawn per day, kg/d

= 30 to 50 days.

Another parameter is HRT which is given by: HRT= Reactor volume, m3

Flow rate, m3/h

= 8 to 10 h or more at average flow.

The reactor volume has to be so chosen that the desired SRT value is achieved. This is done by solving for HRT from SRT equation assuming (i) depth of

reactor (ii) the effective depth of the sludge blanket, and (iii) the average concentration of sludge in the blanket (70 kg/m3). The full depth of the

reactor for treating low BOD municipal sewage is often 4.5 to 5.0 m of which the sludge blanket itself may be 2.0 to 2.5 m depth. For high BOD wastes, the

depth of both the sludge blanket and the reactor may have to be increased so that the organic loading on solids may be kept within the prescribed range.

Once the size of the reactor is fixed, the upflow velocity can be determined from Upflow velocity m/h = Reactor height

HRT, h

Using average flow rate one gets the average HRT while the peak flow rate gives the minimum HRT at which minimum exposure to treatment occurs. In order to

retain any flocculent sludge in reactor at all times, experience has shown that the upflow velocity should not be more than 0.5 m/h at average flow and not

more than 1.2 m/h at peak flow. At higher velocities, carry over of solids might occur and effluent quality may be deteriorated. The feed inlet system is

next designed so that the required length and width of the UASB reactor are determined.

The settling compartment is formed by the sloping hoods for gas collection. The depth of the compartment is 2.0 to 2.5 m and the surface overflow rate kept

at 20 to 28 m3/m2-day (1 to 1.2 m/h) at peak flow. The flow velocity through the aperture connecting the reaction zone with the settling compartment is

limited to not more than 5 m/h at peak flow. Due attention has to be paid to the geometry of the unit and to its hydraulics to ensure proper working of the

"Gas-Liquid-Solid-Separator (GLSS)" the gas collection hood, the incoming flow distribution to get spatial uniformity and the outflowing effluent.

Physical Parameters

A single module can handle 10 to 15 MLD of sewage. For large flows a number of modules could be provided. Some physical details of a typical UASB reactor

module are given below:

Reactor configuration

Depth

Width or diameter

Length

Inlet feed

Rectangular or circular. Rectangular shape is preferred 4.5 to 5.0 m for sewage.

To limit lengths of inlet laterals to around 10-12 m for facilitating uniform flow distribution and sludge withdrawal.

As necessary.

gravity feed from top (preferred for municipal sewage) or pumped feed from bottom through manifold and laterals (preferred in case of soluble industrial

wastewaters).
4.

Sludge blanket depth 2 to 2.5 m for sewage. More depth is needed for stronger wastes.

This is a deflector beam which together with the gas hood (slope 60) forms a "gas-liquid-solid-separator" (GLSS) letting the gas go to the gas collection

channel at top, while the liquid rises into the settler

Deflector/GLSS compartment and the sludge solids fall back into the sludge compartment. The flow velocity through the aperture connecting the reaction zone

with the settling compartmentt is generally limited to about 5m/h at peak flow.

Settler compartment 2.0-2.5 m in depth. Surface overflow rate equals 20-28 m3/m2/d at peak flow.

Process Design Parameters

A few process design parameters for UASBs are listed below for municipal sewages with BOD about 200-300 mg/l and temperatures above 20°C.
5.

HRT

SRT

Sludge blanket concentration (average)

Organic loading on sludge blanket

Volumetric organic loading

BOD/COD removal efficiency

Inlet points

Flow regime

8-10 hours at average flow (minimum 4 hours at peak flow)

30-50 days or more
15-30 kg VSS per m3. About 70 kg TSS per m3.

0.3-1.0 kg COD/kg VSS day (even upto 10 kg COD/ kg VSS day for agro-industrial wastes).

1-3 kg COD/m3 day for domestic sewage (10-15 kg COD/m3 day for agro-industrial wastes)

Sewage 75-85% for BOD. 74-78% for COD.
Minimum 1 point per 3.7-4.0 m2 floor area.

Either constant rate for pumped inflows or typically
6.

fluctuating flows for gravity systems.

Upflow velocity About 0.5 m/h at average flow, or 1.2 m/h at peak flow, whichever is low.

Sludge production 0.15-0.25 kg TS per m3 sewage treated.

Sludge drying time Seven days (in India)

Gas production Theoretical 0.38 m3/kg COD removed. Actual 0.1-0.3 m3 per kg COD removed.

Gas utilization	Method of use is optional. 1 m3 biogas with 75% methane content is equivalent to 1.4 kWh electricity.
Nutrients nitrogen and phosphorus removal	5 to 10% only.

Nitrification-Denitrification Systems

A certain amount of nitrogen removal (20-30%) occurs in conventional activated sludge systems. Nitrogen removal ranging from 70 to 90 % can be obtained by

use of nitrification-denitrification method in plants based on activated sludge and other suspended growth systems. Biological denitrification requires prior

nitrification of all ammonia and organic nitrogen in the incoming waste.

Nitrification

There are two groups of chemoautotrophic bacteria that can be associated with the process of nitrification. One group (Nitrosomonas) derives its energy

through the oxidation of ammonium to nitrite, whereas the other group (Nitrobacter) obtains energy through the oxidation of nitrite to nitrate. Both the

groups, collectively called Nitrifiers, obtain carbon required, from inorganic carbon forms. Nitrification of ammonia to nitrate is a two step process:

Nitrosomonas Nitrobacter

NH3 NH4 NO2 NO3

Stoichiometrically, 4.6 kg of oxygen is required for nitrifying 1 kg of nitrogen. Under steady state conditions, experimental evidence has shown nitrite

accumulation to be insignificant. This suggests that the rate-limiting step for the conversion of ammonium to nitrate is the oxidation of ammonium to nitrite

by the genus Nitrosomonas.

Combined and Separate Systems of Biological Oxidation & Nitrification

Following figure shows flow sheets for combined and separate systems for biological oxidation and nitrification.

Combined system is favoured method of operation as it is less sensitive to load variations - owing to larger sized aeration tank - generally produces a

smaller volume of surplus sludge
owing to higher values of adopted, and better sludge settleability.
c

Care should be taken to ensure that the oxygenation capacity of aeration tank is sufficient to meet oxygen uptake due to carbonaceous demand and

nitrification. Recycling of sludge must be rapid enough to prevent denitrification (and rising sludge) owing to anoxic conditions in the settling tank.

In separate system, the first tank can be smaller in size since a higher F/M ratio can be used, but this makes the system somewhat more sensitive to load

variations and also tends to produce more sludge for disposal. An additional settling tank is also necessary between the two aeration tanks to keep the two

sludges separate. A principal advantage of this system is its higher efficiency of nitrification and its better performance when toxic substances are feared

to be in the inflow.

. Biological Denitrification

When a treatment plant discharges into receivingstream with lowavailable nitrogen concentration and with a flow much larger than the effluent, the presence

of nitrate in the effluent generally does not adversely affect stream quality. However,if the nitrate concentration in the stream is significant, it may be

desirable to control the nitrogen contentof the effluent, as highly nitrified effluents can stillaccelarate algal blooms. Even more critical is the case

where treatment plant effluent is dischargeddirectly into relatively still bodies of water such as lakes or reservoirs. Another argument for the controlof

nitrogen in the aquatic environment is theoccurence of infantile methemoglobinemia,which results from high concentration of nitrates indrinking water.

The four basic processes that are used are: (1) ammonia stripping,(2) selective ion exchange,

(3) break point chlorination, and (4) biological nitrification/denitrification.

Biological nitrification/denitrification is a two step process. The first step is nitrification, which is conversion of ammonia to nitrate through the action

of nitrifying bacteria. The second step is

nitrate conversion (denitrification), which is carried out by facultative heterotrophic bacteria under anoxic conditions.

Microbiological Aspects of Denitrification

- Nitrate conversion takes place through both assimilatory and dissimilatory cellular functions. In assimilatory denitrification, nitrate is reduced to

ammonia, which then serves as a nitrogen source for cell synthesis. Thus, nitrogen is removed from the liquid stream by incorporating it into cytoplasmic

material.

- In dissimilatory denitrification, nitrate serves as the electron acceptor in energy

metabolism and is converted to various gaseous end products but principally molecular nitrogen, N2, which is then stripped from the liquid stream.

- Because the microbial yield under anoxic conditions is considerably lower than under aerobic conditions, a relatively small fraction of the nitrogen

is removed through assimilation. Dissimilatory denitrification is, therfore, the primary means by which nitrogen removal is achieved.

- A carbon source is also essential as electron donor for denitrification to take place. This source may be in the form of carbon internally available

in sewage or artificially added (eg. as methanol). Since most community wastewaters have a higher ratio of BOD:N, the internally available carbon becomes

attractive and economical for denitrification.

Denitrification releases nitrogen which escapes as an inert gas to the atmosphere while oxygen released stays dissolved in the liquid and thus reduces the

oxygen input needed into the system. Each molecule of nitrogen needs 4 molecules of oxygen during nitrification but releases back 2.5 molecules in

denitrification. Thus, theoretically, 62.5% of the oxygen used is released back in denitrification.

Typical Flowsheets for Denitrification

Denitrification in suspended growth systems can be achieved using anyone of the typical flowsheets shown in the figure.

- The use of methanol or any other artificial carbon source should be avoided as far as possible since it adds to the cost of treatment and also some

operating difficulties may arise fro dosing rate of methanol. Too much would introduce an unnecessary BOD in the effluent while too little would leave some

nitrates undernitrified.

- A more satisfactory arrangement would be to use the carbon contained in the waste itself. However, the anoxic tank has to be of sufficient detention

time for denitrification to occur which, has a slower rate; since the corresponding oxygen uptake rate of the mixed liquor is mainly due to endogenous

respiration and is thus low. The denitrification rate, therefore, in a way also depends on the F/M ratio in the prior aeration tank.

- Consequently, if desired, a portion of the raw waste may be bypassed to enter directly into the anoxic tank and thus contribute to an increased

respiration rate. This reduces

the sizes of both the anoxic and aeration tanks, but the denitrification efficiency is reduced as the bypassed unnitrified ammonia can not be denitrified.

- By reversing the relative positions of anoxic and aerobic tanks, the oxygen requirement of the waste in its anoxic state is met by the release of

oxygen from nitrates in the recycled flow taken from the end of nitrification tank. Primary settling of the raw waste may be omitted so as to bring more

carbon into the anoxic tank.

- More complete nitrification-denitrification can be achieved by Bardenpho arrangement. The first anoxic tank has the advantage of higher

denitrification rate while the nitrates remaining in the liquor passing out of the tank can be denitrified further in a second anoxic tank through endogenous

respiration.

- The flow from anoxic tank is desirable to reaerate for 10-15 minutes to drive off nitrogen gas bubbles and add oxygen prior to sedimentation.

Removal

Phosphorus precipitation is ususally achieved by addition of chemicals like calcium hydroxide, ferrous or ferric chloride, or alum, either in the primary or

the final settling tank.

Alum is more expensive and generates more hydroxide, which creates extra sludge, that is difficult to dewater. Use of lime results in an increase of

approximately 50% in surplus sludge, but the sludge is reported to have good dewatering properties. When using iron salts, a molar ratio of 1.0:1.4 of iron

to phosphorus is reported to give 91-96% removal of total phosphorus using ferrous chloride dosed directly beneath the aerator.

Chemical addition prior to biological treatment is feasible if a primary settling tank exists as in the case of the conventional activated sludge process.

The dose requirement then increases, but chemical precipitation also improves organic removal, thus reducing BOD load on the biological treatment. For

extended aeration plants there is no primary settling; chemical addition has to be done in the final settling tank.

Residual Management

In all biological waste treatment processes some surplus sludge is produced. The objective of residual management is:

- Reduction of water content.
- Stabilization of sludge solids.
- Reduction in sludge solids volume.

In facultative type aerated lagoons and algal waste stabilization ponds, the surplus sludge settles out in the unit itself and is removed only once in a few

years after emptying the unit, exposing the wet sludge to natural drying, and carting away the dried sludge for agricultural use or land filling.

In extended aeration process where aerobic digestion of surplus sludge is done, the sludge can be taken directly for dewatering and disposal.

In case of activated sludge and trickling filter plants, the sludge is taken (along with the primary sludge) to a sludge digester for further

demineralization and thereafter it is dewatered.

Sludge Dewatering Methods

- Natural: sludge drying beds, sludge lagoons
- Mechanical: sludge thickeners, centrifuges, vacuum filters, filter press
- Physical: heat drying, incineration

Disposal of Sludge

Final disposal of sludge is to land and sometimes to the sea, in one of the following ways:

- Agricultural use of dried or wet sludge.
- Use of dried sludge as landfill in absence of agricultural demand.
- Spreading wet sludge on eroded or waste land, contouring the field, so as to gradually build up a top soil of agricultural value.
- Disposing off wet sludge along with solid wastes for (i) composting, or (ii) sanitary landfill.
- Transporting and dumping into the sea.

Sludge Characteristics

For the rational design of sludge drying systems, it is esstential to know a few characteristics of sludges, such as moisture content as affected by the

nature and extent of organic and other matter contained in them, their specific gravity, weight and volume relationships, their dewatering characteristics,

etc. The specific gravity of sludge is very close to that of water itself, 1.01 for biological sludge and 1.02 from alum sludge.

Stepwise reduction in moisture content in dewatering extended aeration sludge

Sludge source	Moisture content	Weight, g/person-day

	% by weight	Solids	Water	Total
Initial moisture content	99	30	2970	3000
After thickening	96	30	720	750
After other mechanical process	90	30	270	300
After natural or physical drying	60	30	45	75

It is evident that the bulk of the water is removed in the thickener. Thereafter, the bulk of the remaining moisture is removed in free drainage. Evaporation

removes the least but, in fact,

takes the longest time. The final "dried" sludge still has considerable moisture in it, but the sludge is now "handleable".

Sand Beds for Sludge Drying

Sand beds are generally constructed as shown in the typical cross-sectional view.

Sludge is generally spread over the sand which is supported on a gravel bed, through which is laid an open-joint earthen pipe 15 cm in diameter spaced about

3 m apart and sloping at a gradient of 1 in 150 towards the filtrate sump. The drying beds are often subdivided into smaller units, each bed 5-8 m wide and

15-50 m long. The drying time averages about 1-2 weeks in warmer climates, and 3-6 or even more in unfavourable ones.

Sludge Digestion

Sludge digestion involves the treatment of highly concentrated organic wastes in the absence of oxygen by anaerobic bacteria.The anaerobic treatment of

organic wastes resulting in the production of carbon dioxide and methane, involves two distinct stages. In the first stage, referred to as "acid

fermentation", complex waste components, including fats, proteins, and polysaccharides are first hydrolyzed by a heterogeneous group of facultative and

anaerobic bacteria. These bacteria then subject the products of hydrolysis to fermentations, b-oxidations, and other metabolic processes leading to the

formation of simple organic compounds, mainly short-chain (volatile) acids and alcohols. However in the second stage, referred to as "methane fermentation",

the end products of the first stage are converted to gases (mainly methane and carbon dioxide) by several different species of strictly anaerobic bacteria.

The bacteria responsible for acid fermentation are relatively tolerant to changes in pH and temperature and have a much higher rate of growth than the

bacteria responsible for methane fermentation. If the pH drops below 6.0, methane formation essentially ceases, and more acid accumulates, thus bringing the

digestion process to a standstill. As a result, methane fermentation is generally assumed to be the rate limiting step in anaerobic wastewater

treatment. The methane bacteria are highly active in mesophilic (27-43°C) with digestion period of four weeks and thermophilic range (35-40°C) with digestion

period of 15-18 days. But thermophilic range is not practised because of odour and operational difficulties.

Digestion Tanks or Digesters

A sludge digestion tank is a RCC or steel tank of cylindrical shape with hopper bottom and is covered with fixed or floating type of roofs.

Types of Anaerobic Digesters

The anaerobic digesters are of two types: standard rate and high rate. In the standard rate digestion process, the digester contents are usually unheated and

unmixed. The digestion period may vary from 30 to 60 d. In a high rate digestion process, the digester contents are heated and completely mixed. The required

detention period is 10 to 20 d.

Often a combination of standard and high rate digestion is achieved in two-stage digestion. The second stage digester mainly separates the digested solids

from the supernatant liquor: although additional digestion and gas recovery may also be achieved.

Design Details

Generally digesters are designed to treat for a capacity upto 4 MLD.

1. Tank sizes are not less than 6 m diameter and not more than 55 m diameter.

2. Liquid depth may be 4.5 to 6 m and not greater than 9 m.

3. The digester capacity may be determined from the relationship

$$V = [V_f - 2/3 (V_f - V_d)]t_1 + V_d t_2$$

where V = capacity of digester in m3, Vf = volume of fresh sludge m3/d, Vd = volume of daily digested sludge accumulation in tank m3/d, t1= digestion time in

days required for digestion, d, and t2 = period of digested sludge storage.

Gas Collection

The amount of sludge gas produced varies from 0.014 to 0.028 m3 per capita. The sludge gas is normally composed of 65% methane and 30% carbondioxide and

remaining 5% of nitrogen and other inert gases, with a calorific value of 5400 to 5850 kcal/m3.

Treatment Plant Layout and Siting

Plant layout is the arrangement of designed treatment units on the selected site. The components that need to be included in a treatment plant, should be so

laid out as to optimize land requirement, minimize lengths of interconnecting pipes and pumping heads. Access for sludge and chemicals transporting, and for

possible repairs, should be provided in the layout.

Siting is the selection of site for treatment plant based on features as character, topography, and shoreline. Site development should take the advantage of

the existing site topography. The following principles are important to consider:

1. A site on a side-hill can facilitate gravity flow that will reduce pumping requirements and locate normal sequence of units without excessive

excavation or fill.

2. When landscaping is utilized it should reflect the character of the surrounding area. Site development should alter existing naturally stabilized

site contours and drainage as little as possible.

3. The developed site should be compatible with the existing land uses and the comprehensive development plan.

Treatment Plant Hydraulics

Hydraulic profile is the graphical representation of the hydraulic grade line through the treatment plant. If the high water level in the receiving water is

known, this level is used as a control point, and the head loss computations are started backward through the plant. The total available head at the

treatment plant is the difference in water surface elevations in the interceptor and the water surface elevation in the receiving water at high flood level.

If the total available head is less than the head loss through the plant, flow by gravity cannot be achieved. In such cases pumping is needed to raise the

head so that flow by gravity can occur.

There are many basic principles that must be considered when preparing the hydraulic profile through the plant. Some are listed below:

1. The hydraulic profiles are prepared at peak and average design flows and at minimum initial flow.

2. The hydraulic profile is generally prepared for all main paths of flow through the plant.

3. The head loss through the treatment plant is the sum of head losses in the treatment units and the connecting piping and appurtenances.

4. The head losses through the treatment unit include the following:

a. Head losses at the influent structure.

b. Head losses at the effluent structure.

c. Head losses through the unit.

d. Miscellaneous and free fall surface allowance.

5. The total loss through the connecting pipings, channels and appurtenances is the sum of following:

a. Head loss due to entrance.

b. Head loss due to exit.

c. Head loss due to contraction and enlargement.

d. Head loss due to friction.

e. Head loss due to bends, fittings, gates, valves, and meters.

f. Head required over weir and other hydraulic controls.

g. Free-fall surface allowance.

Treated Effluent Disposal

The proper disposal of treatment plant effluent or reuse requirements is an essential part of planning and designing wastewater treatment facilities.

Different methods of ultimate disposal of secondary effluents are discussed as follows.

Natural Evaporation

The process involves large impoundments with no discharge. Depending on the climatic conditions large impoundments may be necessary if precipitation exceeds

evaporation. Therefore, considerations must be given to net evaporation, storage requirements, and possible percolation and groundwater pollution. This

method is particularly beneficial where recovery of residues is desirable such as for disposal of brines.

Groundwater Recharge

Methods for groundwater recharge include rapid infiltration by effluent application or impoundment, intermittent percolation, and direct injection. In all

cases risks for groundwater

pollution exists. Furthermore, direct injection implies high costs of treating effluent and injection facilities.

Irrigation

Irrigation has been practiced primarily as a substitute for scarce natural waters or sparse rainfall in arid areas. In most cases food chain crops (i.e.

crops consumed by humans and those animals whose products are consumed by humans) may not be irrigated by effluent. However, field crops such as cotton,

sugar beets, and crops for seed production are grown with wastewater effluent.

Wastewater effluent has been used for watering parks, golf courses and highway medians.

Recreational Lakes

The effluent from the secondary treatment facility is stored in a lagoon for approximately 30 days. The effluent from the lagoon is chlorinated and then

percolated through an area of sand and gravel, through which it travels for approximately 0.5 km and is collected in an interceptor trench. It is discharged

into a series of lakes used for swimming, boating and fishing.

Aquaculture

Aquaculture, or the production of aquatic organisms (both flora and fauna), has been practiced for centuries primarily for production of food, fiber and

fertilizer. Lagoons are used for aquaculture, although artificial and natural wetlands are also being considered. However, the uncontrolled spread of water

hyacinths is itself a great concern because the flora can clog waterways and ruin water bodies.

Municipal Uses

Technology is now available to treat wastewater to the extent that it will meet drinking water quality standards. However, direct reuse of treated wastewater

is practicable only on an emergency basis. Many natural bodies of water that are used for municipal water supply are also used for effluent disposal which is

done to supplement the natural water resources by reusing the effluent many times before it finally flows to the sea.

Industrial Uses

Effluent has been successfully used as a cooling water or boiler feed water. Deciding factors for effluent reuse by the industry include (1) availability of

natural water, (2) quality and quantity of effluent, and cost of processing, (3) pumping and transport cost of effluent, and (4) industrial process water

that does not involve public health considerations.

Discharge into Natural Waters

Discharge into natural waters is the most common disposal practice. The self-purification or assimilative capacity of natural waters is thus utilized to

provide the remaining treatment.

Stabilization Ponds

- The stabilization ponds are open flow through basins specifically designed and constructed to treat sewage and biodegradable industrial wastes. They

provide long detention periods extending from a few to several days.

- Pond systems, in which oxygen is provided through mechanical aeration rather than algal photosynthesis are called aerated lagoons.

- Lightly loaded ponds used as tertiary step in waste treatment for polishing of secondary effluents and removal of bacteria are called maturation

ponds.

Classification of Stabilization Ponds

Stabilization ponds may be aerobic, anaerobic or facultative.

- Aerobic ponds are shallow ponds with depth less than 0.5 m and BOD loading of 40-120 kg/ha.d so as to maximize penetration of light throughout the

liquid depth. Such ponds develop intense algal growth.

- Anaerobic ponds are used as pretreatment of high strength wastes with BOD load of 400-3000 kg/ha.d Such ponds are constructed with a depth of 2.5-5m

as light penetration is unimportant.

- Facultative pond functions aerobically at the surface while anaerobic conditions prevail at the bottom.They are often about 1 to 2 m in depth. The

aerobic layer acts as a good check against odour evolution from the pond.

Mechanism of Purification

The functioning of a facultative stabilization pond and symbiotic relationship in the pond are shown below. Sewage organics are stabilized by both aerobic

and anaerobic reactions. In the top aerobic layer, where oxygen is supplied through algal photosynthesis, the non-settleable and dissolved organic matter is

oxidized to CO_2 and water. In addition, some of the end products of partial anaerobic decomposition such as volatile acids and alcohols, which may permeate

to upper layers are also oxidized periodically. The settled sludge mass originating from raw waste and microbial synthesis in the aerobic layer and dissolved

and suspended organics in the lower layers undergo stabilization through conversion to methane which escapes the pond in form of bubbles.

Factors Affecting Pond Reactions

Various factors affect pond design:

- wastewater characteristics and fluctuations.
- environmental factors (solar radiation, light, temperature)
- algal growth patterns and their diurnal and seasonal variation)
- bacterial growth patterns and decay rates.
- solids settlement, gasification, upward diffusion, sludge accumulation.

The depth of aerobic layer in a facultative pond is a function of solar radiation, waste characteristics, loading and temperature. As the organic loading is

increased, oxygen production by algae falls short of the oxygen requirement and the depth of aerobic layer decreases. Further, there is a decrease in the

photosynthetic activity of algae because of greater turbidity and inhibitory effect of higher concentration of organic matter.

Gasification of organic matter to methane is carried out in distinct steps of acid production by acid forming bacteria and acid utilization by methane

bacteria. If the second step does not proceed satisfactorily, there is an accumulation of organic acids resulting in decrease of pH which would result in

complete inhibition of methane bacteria. Two possible reasons for imbalance between activities of methane bacteria are: (1) the waste may contain inhibitory

substances which would retard the activity of methane bacteria and not affect the activity of acid producers to the same extent. (2) The activity of methane

bacteria decreases much more rapidly with fall in temperature as compared to the acid formers.

Thus, year round warm temperature and sunshine provide an ideal environment for operation of facultative ponds.

Algal Growth and Oxygen Production

Algal growth converts solar energy to chemical energy in the organic form. Empirical studies have shown that generally about 6% of visible light energy can

be converted to algal energy.

The chemical energy contained in an algal cell averages 6000 calories per gram of algae.

Depending on the sky clearance factor for an area, the average visible radiation received can be estimated as follows:

Avg. radiation= Min. radiation + [(Max. radiation - Min. radiation)x sky clearance

factor]

Oxygen production occurs concurrently with algal production in accordance with following equation:

$106C02 + 16NO3 + HPO4 + 122H2O + 18H+ \; C106H263O110N16P1 + 138O2$

On weight basis, the oxygen production is 1.3 times the algal production.

Areal Organic Loading

The permissible areal organic loading for the pond expressed as kg BOD/ha.d will depend on the minimum incidence of sunlight that can be expected at a

location and also on the percentage of influent BOD that would have to be satisfied aerobically. The Bureau of Indian Standards has related the permissible

loading to the latitude of the pond location to aerobically stabilize the organic matter and keep the pond odour free. The values are applicable to towns at

sea levels and where sky is clear for nearly 75% of the days in a year. The values may be modified for elevations above sea level by dividing by a factor (1

+ 0.003 EL) where EL is the elevation of the pond site above MSL in hundred meters.

Detention Time

The flow of sewage can approximate either plug flow or complete mixing or dispersed flow. If BOD exertion is described by first order reaction, the pond

efficiency is given by:
for plug flow: $Le/Li = e-k1t$

for complete mixing: $Le/Li = 1$

1+k1t

For dispersed flow the efficiency of treatment for different dgrees of intermixing is characterized by dispersion numbers.Choice of a larger value for

dispersion number or assumption of complete mixing would give a conservative design and is recommended.

Depth

Having determined the surface area and detention capacity, it becomes necessary to consider the depth of the pond only in regard to its limiting value. The

optimum range of depth for facultative ponds is 1.0 - 1.5 m.

Aquatic Plant Systems

Aquatic systems in waste treatment are either free floating growths harnessed in the form of built-up ponds for waste treatment such as duckweed and hyacinth

ponds or rooted vegetations (reeds) which emerge out of shallow waters cultivated in constructed wetlands.

- Natural wetlands exists all over the world. They generally have saturated soil conditions and abound in rooted vegetation which emerges out of

shallow waters in the euphotic zone. They may also have phytoplankton. Natural wetlands can be integrated with wastewater treatment systems.

- Constructed wetlands are man-made for treatment of wastewater, mine drainage, storm drainage, etc. They have rooted vegetation.

Longitudinal Section Through a Typical Reed Bed With Gravel, Sand or Selected Soil With Horizontal Flow of Wastewater

- Aquatic plant ponds consisting of free floating macrophytes, such as water hyacinths, duckweeds, etc. have been cultured in ponds either for their

ability to remove heavy metals, phenols, nutrients, etc. from wastewaters or to assist in giving further treatment to pretreated wastewaters to meet

stringent discharge standards while at the same time producing new plant growths for their gas production or food value.
Conceptual flowsheet showing waste treatment using an aquatic plant pond

Septic Tank

Septic tanks are horizontal continuous flow, small sedimentation tanks through which sewage is allowed to flow slowly to enable the sewage solids to settle

to the bottom of the tank, where they are digested anaerobically. The tank is de-sludged at regular intervals usually once every 1-5 years.

Cesspool

It is a pit excavated in soil with water tight lining and loose lining by stone or brick to provide for leaching of wastewater by sides and the pit is

covered. The leaching type is suitable for porous soils. The capacity should not be less than one day's flow into the pit. If all the water in a test pit of

one meter diameter and 2 m deep, disappears in 24 hours, such soil is best suitable for cesspools. The bottom of the cesspool must be well above the ground

water level. After sometime the sides of pit get clogged by the sewage solids, reducing the leaching capacity. At overflow level, an outlet is provided to

take-off unleached liquid into a seepage pit. The settled matter is removed at intervals. Water tight cesspools are cleaned every 6 months and their capacity

must not be less than 70 l/person/month.

Seepage Pit

The seepage pit is needed to discharge the effluent of cesspool, aquaprivy, septic tank or sullage from bathrooms and kitchens. The difference between

seepage pit and cesspool is that the seepage pit is completely filled up with stones. The fine suspended solids adhere to the

surface of stones and get decomposed by the zoogleal film, which are on the stones and the effluent is leached into the sidewalls.