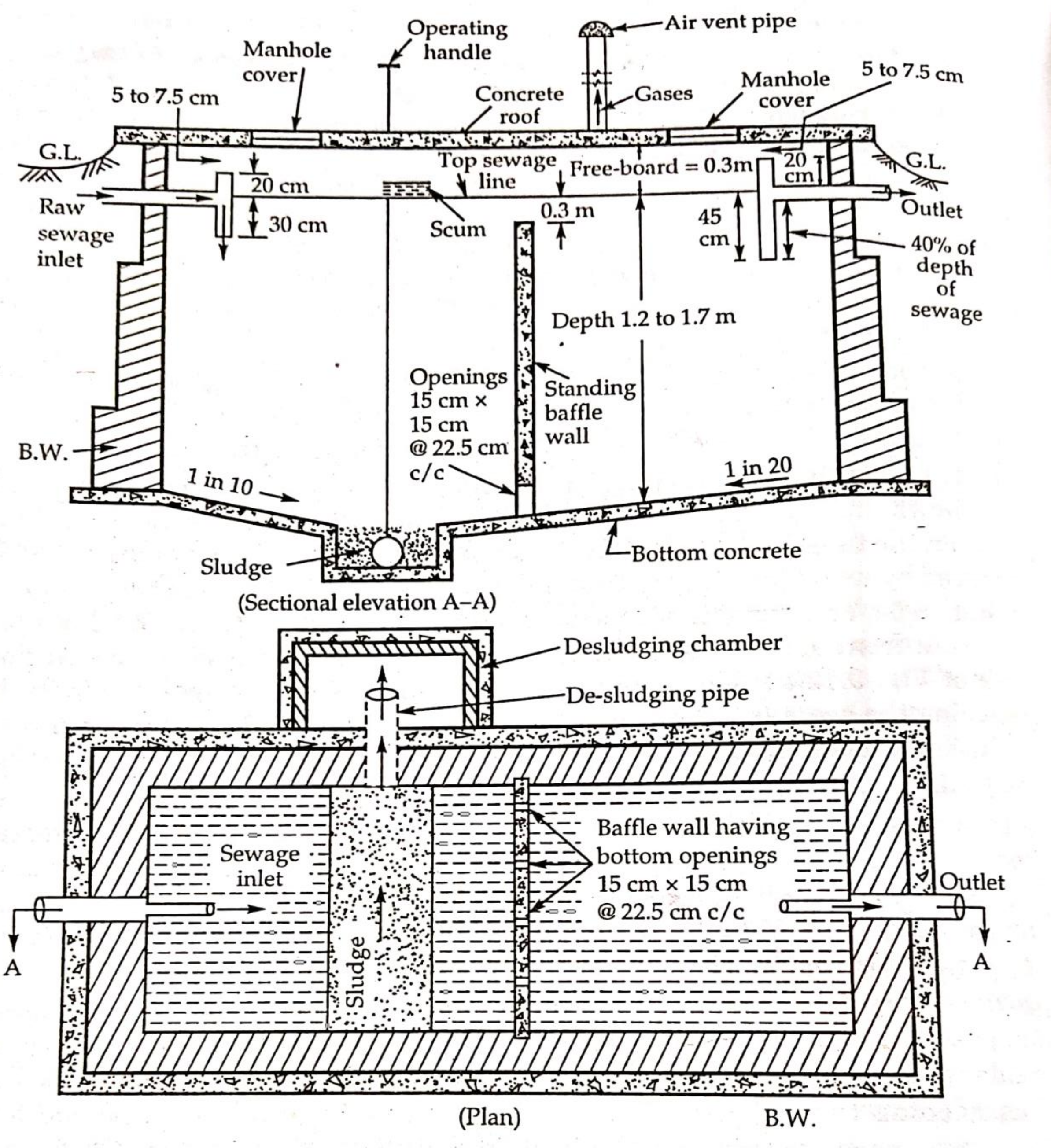
SEPTIC TANK

A septic tank consists of one or more concrete or plastic tanks of between 4000 and 7500 litres; one end is connected to an inlet wastewater pipe and the other to a septic drain field. Generally these pipe connections are made with a T pipe, allowing liquid to enter and exit without disturbing any crust on the surface.

- Waste water flows from the house to the septic tank. The tank is designed to retain wastewater and allow heavy solids to settle to the bottom.
- These solids are partially decomposed by bacteria to form sludge.
- Grease and light particles float, forming a layer of scum on top of the wastewater.
- Baffles installed at the inlet and outlet of the tank to help prevent scum and solids from escaping.

The design of the tank usually incorporates two chambers, each equipped with an access opening and cover, and separated by a dividing wall with openings located about midway between the floor and roof of the tank. Wastewater enters the first chamber of the tank, allowing solids to settle and scum to float. The settled solids are anaerobically digested, reducing the volume of solids. The liquid component flows through the dividing wall into the second chamber where further settlement takes place with the excess liquid then draining in a relatively clear condition from the outlet into the leach field, also referred to as a drain field, or seepage field, depending upon locality.


Operating handle
Air vent pipe
Manhole cover
Gases
Manhole cover
5 to 7.5 cm
Concrete roof
5 to 7.5 cm
G.L.
Top sewage line
Free-board = 0.3m
20 cm
G.L.
20 cm
Raw sewage inlet
30 cm
Scum
0.3 m
45 cm
Outlet
40% of depth of sewage
Depth 1.2 to 1.7 m
Openings 15 cm × 15 cm @ 22.5 cm c/c
Standing baffle wall
B.W.
1 in 10
1 in 20
Sludge
Bottom concrete
(Sectional elevation A–A)
Desludging chamber
De-sludging pipe
Sewage inlet
Sludge
Baffle wall having bottom openings 15 cm × 15 cm @ 22.5 cm c/c
Outlet
A
A
(Plan)
B.W.

SEPTIC TANK

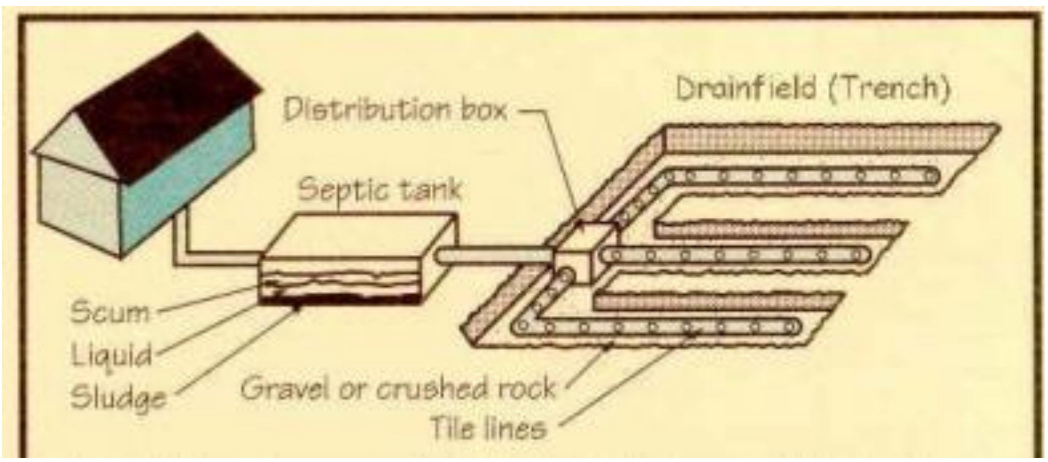
Drainfield (Trench):

• A solid pipe leads from the septic tank to a distribution box where the wastewater is channeled into one or more perforated pipes set in trenches of gravel.

• The water slowly infiltrates (seeps) into the underlying soil.

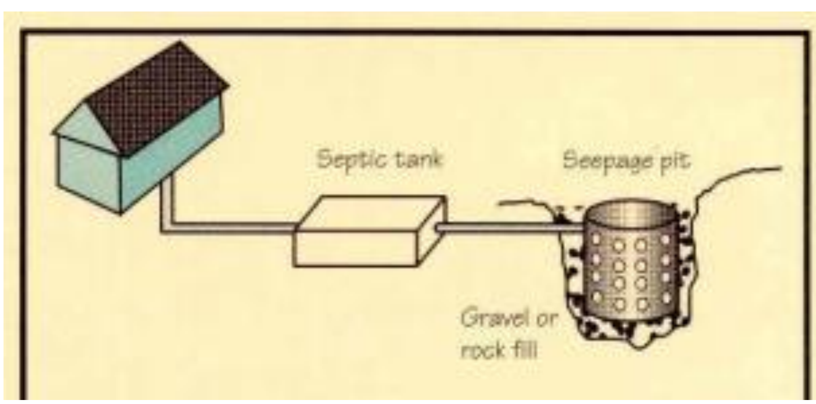
• Dissolved wastes and bacteria in the water are trapped or adsorbed to soil particles or decomposed by microorganisms.

• This process removes disease-causing organisms, organic matter, and most nutrients (except nitrogen and some salts).



Seepage pit(Soak pit):

- An alternative to the common drain field is the Seepage Pit (Dry Well).
- In this type, liquid flows to a pre-cast tank with sidewall holes, surrounded by gravel.
- Liquid seeps through the holes or joints to the surrounding soil.
-



SEPTIC TANK

★ Design Considerations

★ **Capacity** (8 – 10 persons)

- Only water closets are connected → 40 – 70 lpcd (1400 l)
- Sullage is also discharged ⇒ 90 - 150 lpcd (2250 l)
- Rate of accumulation of sludge = 30 lit / person/year.
- Free board = 0.3 – 0.5 m.

★ **Detention time** = 12 – 36 hrs (usually 24 hrs)

★ **Length / width ratio**

L/B ⇒ 2 to 3 .
B ≮ 90 cm
D = 1.2 to 1.8 m

★ **Inlet & outlet Baffles**

★ Baffles / Tees should extend upto top level of scum (20 – 22 cm above top of sewage line), but must stop a little below the bottom of the covering slab (7.5 cm)
⇒ For free movement of gases

★ Inlet should penetrate by 30 cm below top sewage line
Outlet should penetrate to ~ 40 % of depth of sewage.

★ Outlet invert level should be kept 5 – 7.5 cm below Inlet invert level.

Scanned by CamScanner

1. Design a septic tank for 200 users. Water allowance is 120 l/head/day. Detention period = 18 hrs. What would be the size of soak pit if the effluent from this septic tank is to be discharged in it.

Ans. Flow of sewage/day = 200 x 120
= 24000 l/day.

Detention period = 18 hrs.

∴ Tank Capacity = $\frac{24000 \times 18}{24}$ (V = Qt)
= 18000 l

Assume that the tank is to be cleared every year;
Let sludge storage capacity be 30 lit/person/year.

Total quantity of sludge generated = 30 x 200
= 6000 lit.

∴ Total tank capacity = 18000 + 6000
= 24000 lit
≃ 25 m^3 (considering future expansion)

Assume Depth of liquid as 1.7 m.

plan area of tank = 25/1.7
= 14.7 ≃ 15 m^2

Let L/B = 3

Area = LB = $3B^2$ = 15 m^2 B = 2.24 m

∴ B ≃ 2.3 m

L = 6.9 m ≈ 7 m

Assuming a freeboard of 30 cm.

Total depth = 1.7 + 0.3 = 2 m.

Hence provide a septic tank of size

7 m x 2.3 m x 2 m

Design of soakpit-

Assuming percolating capacity of filtering media as 1250 l/m³/day

Sewage outflow = 24000 l/day.

$$\text{Volume of soakpit} = \frac{\text{Sewage outflow}}{\text{Percolation rate}}$$

$$= \frac{24000 \ l/day}{1250 \ l/m^3/day}$$

$$= 19.2 \ m^3$$

Assume depth of soakpit = 3 m.

Plan area = $19.2/3 = 6.4 m^2$.

$$\pi/4 \, d^2 = 6.4$$

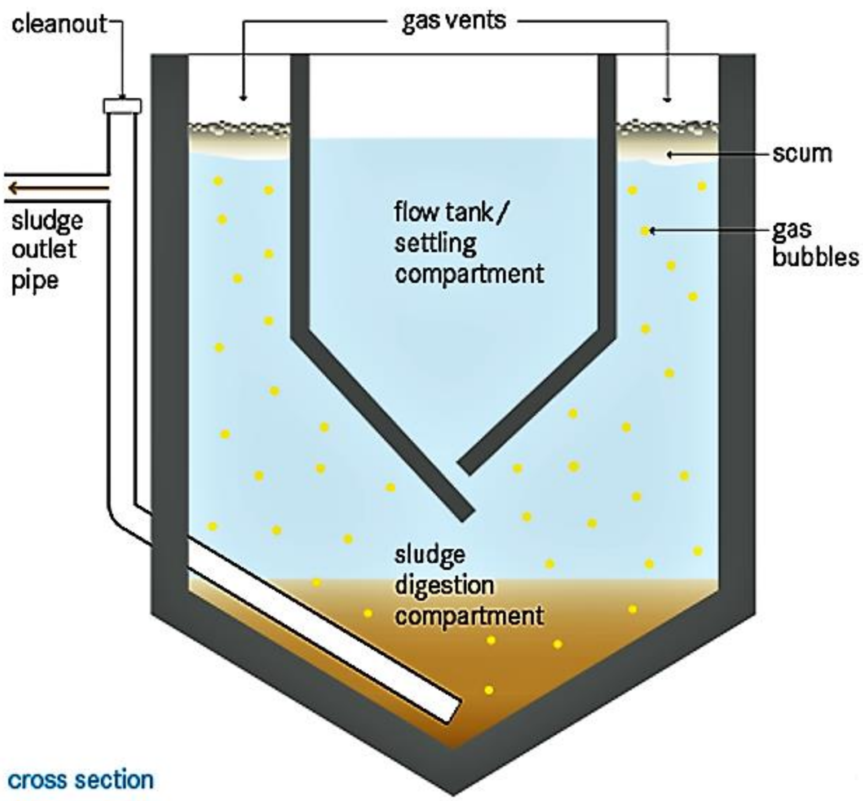
$$d = 2.85 \ m$$

$$d \simeq 2.9 \ m$$

Dia. of soakpit = 2.9m

IMHOFF TANK

Imhoff tanks are the improvements over the septic tanks. Imhoff tanks are two-storeyed tanks which have large settling tanks and below it are sludge digestion chambers. Imhoff tanks are used by small communities and due to the underground construction, land use is very limited. Investment costs are low and operation and maintenance simple. But the treatment efficiency is low and a secondary treatment of the effluent is required. Moreover, the tanks must be desludged regularly.



Process

First the sewage enters the upper sedimentation tank whose bottom has sharp inclinations. The solids are allowed to settle in the upper tank from where they slip in the lower hoppers through the slots. In the hoppers the settled solids remain stored for a long period – about 30-45 days. During this period, they are acted upon by the bacteria and are converted into stable solids, organic acids and gases.

The gas is deflected by baffles to the gas vent channels to prevent it from disturbing the settling process. The gases are allowed to escape in the atmosphere. The stabilized solids are taken out by means of a sludge pipe under hydrostatic pressure. The flow of solids in the lower hoppers is regulated by means of a triangular beam. When one hopper is filled up with the solids its top is closed by means of he triangular beam and in that hopper digestion starts.

The effluent of Imhoff tank is similar to the primary settling tank. The organic matters are digested in the lower compartments. The digested sludge has black colour and has no odour. The moisture contents of this sludge is 90-95%, therefore it can easily flow in the pipes. This moisture can be removed by passing it through sand beds and sun-drying.

These tanks are simple in operation and the process is automatic, uniform and continuous. The sludge can be easily removed under hydrostatic pressure, therefore no pumping is required. The disadvantage of these tanks being more depth (8- 10 m), on operational control and the fouling of the atmosphere due to the developed gases, which are allowed to escape in the atmosphere.

Advantages

- Solid-liquid separation and sludge stabilisation are combined in one single unit
- Resistant against organic shock loads
- Small space requirements
- The effluent remains fresh (i.e., not septic)
- Low operating costs
- Suitable for small settlements and house clusters
- Standardised designs available
- Simple operation and maintenance

Disadvantages

- Very high (or deep) infrastructure; depth may be a problem in case of high groundwater table
- Requires expert design and construction
- Low reduction of pathogens
- Requires desludging

- Inefficient treatment option if not regularly desludged
- Odour occurs from escaping gases
- Effluent, sludge and scum require further treatment
- Less simple than septic tank

IMHOFF TANK

★ Design considerations.

1) Sedimentation chamber : Rectangular

Detention Period = 2-4 hrs.

Surface loading ≯ 30000 $l/m^2/day$.

Flow through velocity ≯ 0·3 m/min

Length ≯ 30 m

L/B = 3 to 5

Depth = 3 to 3·5m. | Total 9 – 11 m |

Freeboard = 45 cm

2) Digestion chamber

Capacity ≐ 57 litres per capita

★ Warmer climate ⇒ 35 to 40 litres per capita

3) Gas vent / Scum chamber

- Surface area of scum chamber = 25-30% of the area of horizontal projection of the top of digestion chamber
- Width of vent ≥ 60 cm .

★ Problem

1. Design an Imhoff tank to treat the sewage from a small town with 30,000 population. The rate of sewage may be assumed as 150 lpcd.

★ a) Design of sedimentation chamber

Total quantity of sewage = 30,000 x 150

= 4500 m^3/day

Assume Detention period as 2 hrs.

Capacity $V = Qt = \frac{4500}{24} \times 2 = 375\ m^3$

Assume effective depth as 2·2 m (Including bottom sloping walls of the chamber) & width as 4·3 m

$\therefore$ Length = $\frac{\text{volume}}{D \times B}$ = $\frac{375}{2.2 \times 4.3}$ = 39·6 m ≃ 40 m

But L ≯ 30 m

$\therefore$ Provide 2 tanks of length 20 m & width 4·3 m.

★ Check for L/B ratio

L/B = 20/4·3 = 4·65 in the range 3 to 5

Hence ok.

Discharge through each unit = $\frac{4500}{2}$ = 2250 m^3/day

★ Check for velocity

Velocity = $\frac{\text{Length}}{\text{Detention time}}$ = $\frac{20}{2 \times 60}$ (hr) = 0·167 m/min < 0·3 m/min.

Hence OK

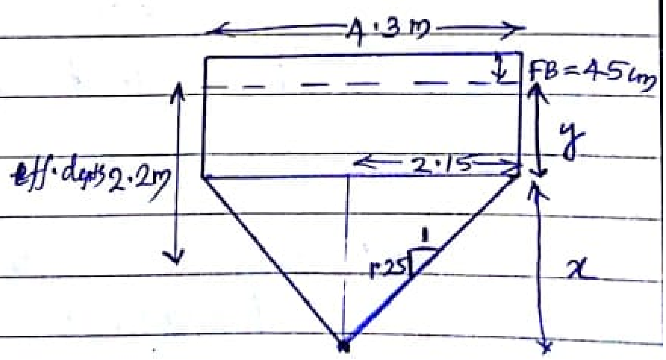
★ Check for Surface loading

Surface loading = $\frac{Q}{L \times B}$ = $\frac{2250 \times 10^3}{20 \times 4.3}$ L/day

= 26162·79 L/m²/day

< 30 000 l/m²/day

Hence OK.



Let the bottom sides be sloping at 1H to 1·25 V

Height of sloping bottom be x

$\frac{1}{1.25} = \frac{2.15}{x}$

x = 1·25 × 2·15

= 2·69 m

∴ Height of vertical portion below liquid surface

$y = 2.2 - x/2 = 2.2 - \frac{2.69}{2}$

$\simeq 0.86$ m

Provide a freeboard of 45 cm.

∴ Total depth of sedimentation chamber upto bottom at the entrance of the slot $= 0.45 + 0.86 + 2.69$

$(FB + y + x) = 4$ m

b) Design of Gas vent & Neutral Zone

Provide a neutral zone of 0.45 m below this depth of 4m. The tank is of 20m length, but below this 4m depth, it shall be divided into a no. of compartments, Let it be of 4 compartment.

∴ Length of each compartment $= 20/4 = 5$m

Let's assume an overall width of 6.5 m & thickness of chamber walls as 0.15 m.

∴ Total width of Gas vent $= 6.5 - 4.3 - (2 \times .15)$

$= 1.9$ m

* Check for width of Gas vent

Width should be about 25-30% of total tank width

ie. $\frac{1.9}{6.5} \times 100 = 29.2\%$ b/w (25-30%)

Hence OK.

Also width of gas vent $= \frac{1.9}{2}$

$= 0.95$m > 0.60 m

Hence OK.

∴ Provide 0.95 m gas vent on either side of the sedimentation chamber.

c) Design of Digestion chamber

Assuming the capacity of digestion chamber as 40 lit/capita

∴ Total capacity = 40 x 30,000 (popln)

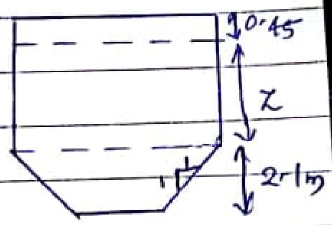
= 1200 m^3

In Total we have 2 tanks having 4 compartments each.

∴ Capacity of each unit = $\frac{1200}{8}$

= 150 m^3

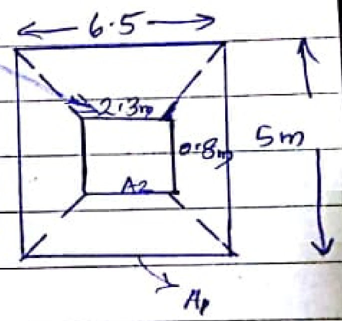
Assuming Depth of each hopper as 2·1 m & side slope 1:1



Bottom Dimensions

5 − (2×2·1) = 0·8 m

6·5 (2×2.1) = 2·3 m



Capacity of each hopper

$= \frac{h}{3}\left[A_1 + A_2 + \sqrt{A_1A_2}\right]$

$= \frac{2 \cdot 1}{3}\left[(6 \cdot 5 \times 5) + (2 \cdot 3 \times \cdot 8) + \sqrt{6 \cdot 5 \times 5 \times 2 \cdot 3 \times 0 \cdot 8}\right]$

= 29·45 m^3

Capacity of rectangular portion = Capacity of each unit − Capacity of each hopper

= 150 − 29·45

= 120·55 m^3

∴ Ht. of rectangular portion $x = \frac{\text{volume}}{\text{area}}$

$= \frac{120 \cdot 55}{6 \cdot 5 \times 5}$ = 3·71 m

∴ Total height of digestion chamber including neutral zone

= 0·45 + 3·71 + 2·1 = 6·26 m

Total height of tank = Height of sedimentation chamber + Height of digestion chamber

$= 4 + 6.26$

$= 10.26$ m

b/w 9 to 11 m

Hence OK

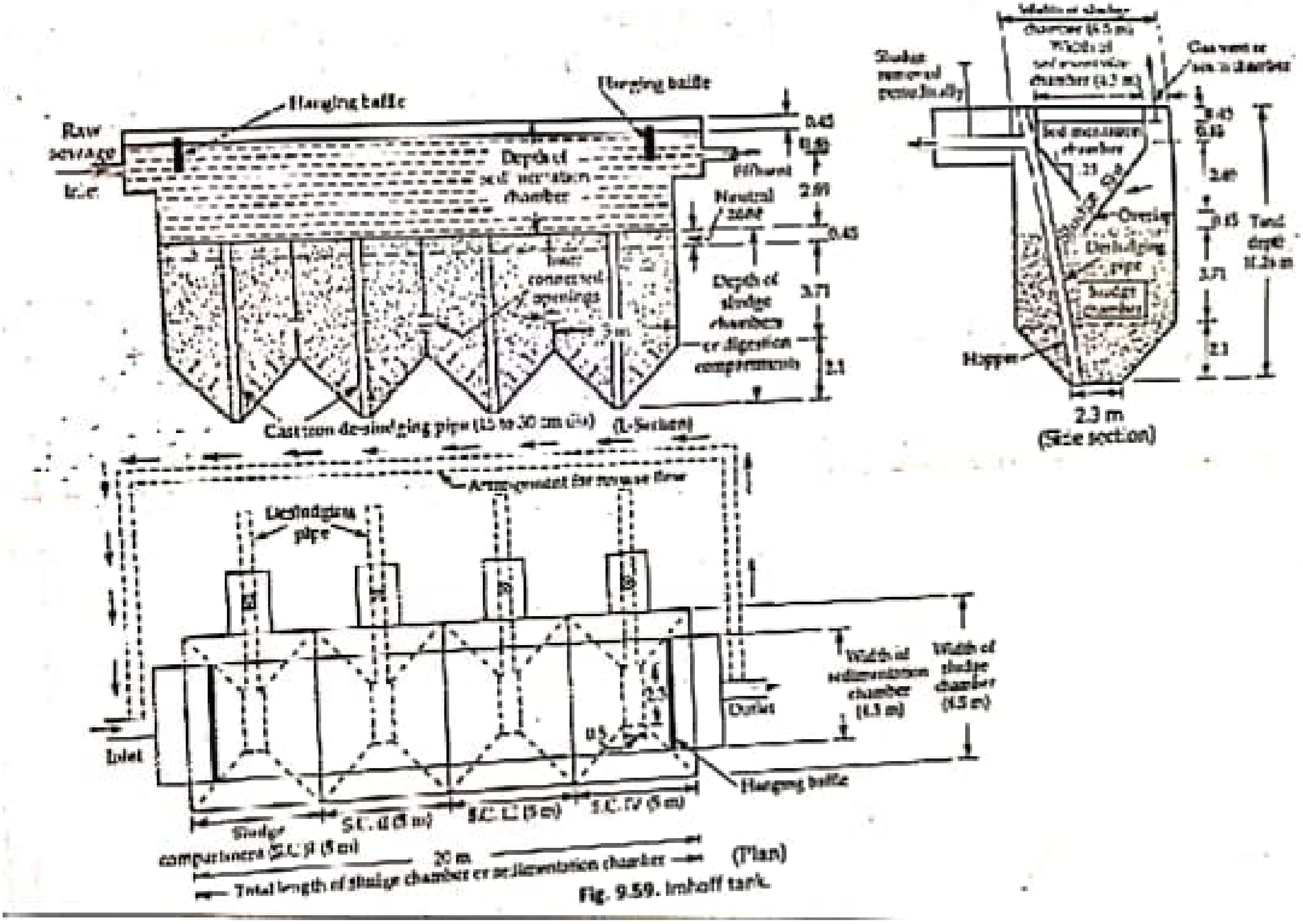


Fig. 9.59. Imhoff tank.

9.11 OXIDATION DITCH

The oxidation ditch is essentially an extended-aeration activated sludge process. An oxidation ditch consists of an endless ditch for the aeration tank and a rotor for aeration mechanism. The ditch consists of a long continuous channel, usually ovel in plan. The channel may be earthen with lined sloping sides and lined floor or it may be built in concrete or brick with vertical walls. There is normally no primary tank used in the oxidation ditch process. Raw sewage passes directly through a bar screen to the ditch.

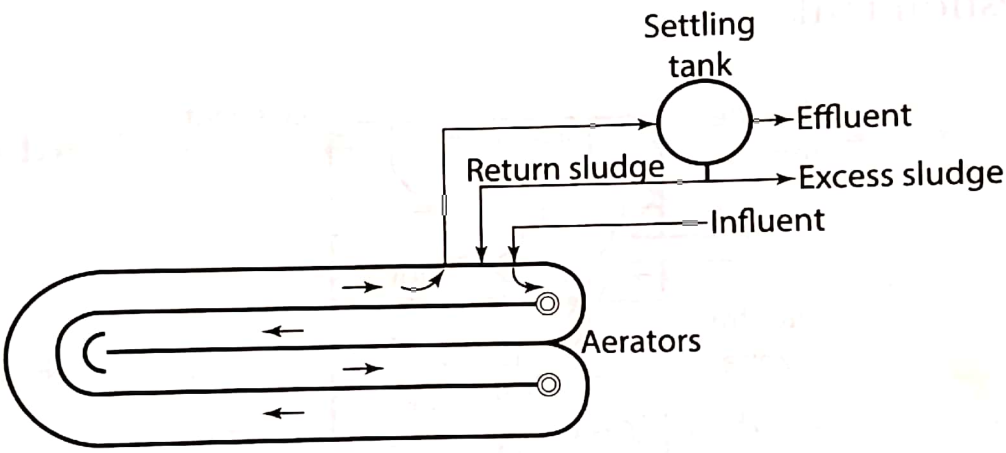


Figure 9.9 Oxidation Ditch

The sewage is aerated by surface rotar placed across the channel, the rotar entrains the necessary oxygen into the liquid and the contents of the ditch mixed and moving. They are designed to impart a velocity of 0.3 to 0.4 m/s to the mixed liquor, preventing the biological sludge from settling out. The width of the ditch divided by the rotor length should give a ratio between 1.5 and 2.8. The longer ratios are normally used for short length of 0.9 to 1.2 m. Oxidation ditches are constructed in two types.

i. Continuous flow type
ii. Intermittent flow type

(i) Continuous flow type

In the continious flow type oxidation ditch the operation is kept continous by allowing the mixed liquor to settle in a separate settling tank. Quiescent conditions in the clarified liquid passes over the effluent weir for final disposal. The settled sludge is removed from the bottom of the clarifier by an air lift or pump and returned to the ditch. The oxidation ditch is operated as a closed system, and the net growth of the volatile suspended solids will require periodic removal of some sludge from the process

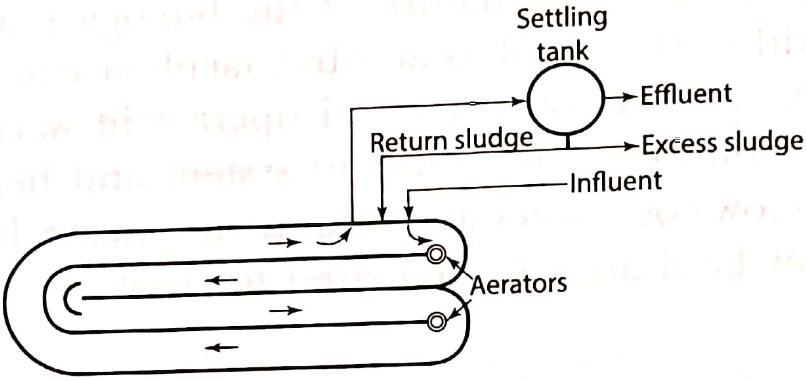


Figure 9.10 Oxidation Ditch continuous flow type

(ii) Intermittent flow type

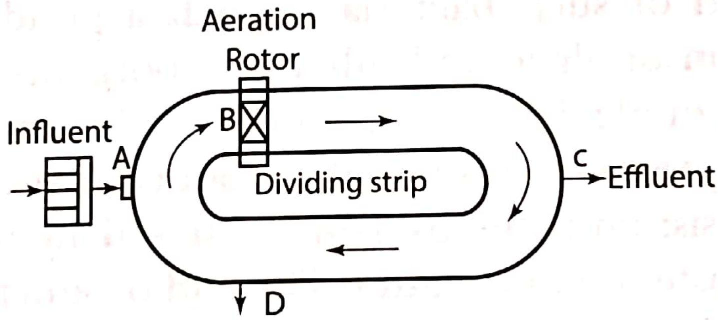


Figure 9.11 Oxidation Ditch intermittent flow type

In the intermittent type oxidation ditch, the numbers of separate settling tanks are used. The flow in the ditch remains suspended during a predetermined period, by stopping the rotar and the ditch itself is used for settling. The supernatant is withdrawn through the outlet. The surplus sludge, settled in the ditch is removed with the aid of a sludge trap. For intermittent operation, the cycle consists of:

i. Closing the inlet valve (A) and aerating the waste water.

ii. Stopping the rotor and lefting the contents settlle.

iii. Opening both inlet, and outlet valves, thereby allowing the incoming wastewater to displace on equal volume of clarified effluent.

...uent. ... displace an equal volume of clarified

9.12 OXIDATION PONDS (STABILIZATION PONDS)

It is a shallow body of water contained in an earthen basin, open to sun and air. Longer time of retention from few days to weeks is provided in the pond. The purification of wastewater occurs due to symbiotic relationship of bacteria and algae. The ponds are classified according to the nature of the biological activity which takes place within the pond as aerobic, facultative and anaerobic. These are cheaper to construct and operate in warm climate as compared to conventional treatment system and hence they are considered as low cost wastewater treatment systems. However, they require higher land area as compared to conventional treatment system.

Aerobic pond

In aerobic pond, the stabilisation of waste is brought about by aerobic bacteria, which flourish in the presence of oxygen. The oxygen demand of such bacteria in such a pond is met by the combined action of algae and other microorganisms, called algal photosynthesis, or algal-symbiosis. In this symbiosis, the algae while growing in the presence of sun light produce oxygen by the action of photosynthesis; and this oxygen is utilised by the bacteria for oxidising the waste organic matter. The end products of the process are carbon dioxide, amonia and phosphates, which are required by the algae to grow and continue to produce oxygen.

Anaerobic pond

In anaerobic pond, the entire depth is under anaerobic condition except an extremely shallow top layer. Normally these ponds are used in series followed by facultative or aerobic pond for complete treatment. The depth of these ponds is in the range of 2.5 to 6 m. They are generally used for the treatment of high strength industrial wastewaters and sometimes for municipal wastewater and sludges. Depending upon the strength of the wastewater, longer retention time up to 50 days is maintained in the anaerobic ponds.

Anaerobic lagoons are covered these days by polyethylene sheet for biogas recovery and eliminating smell problem and green house gas emission in atmosphere.

Faculative pond

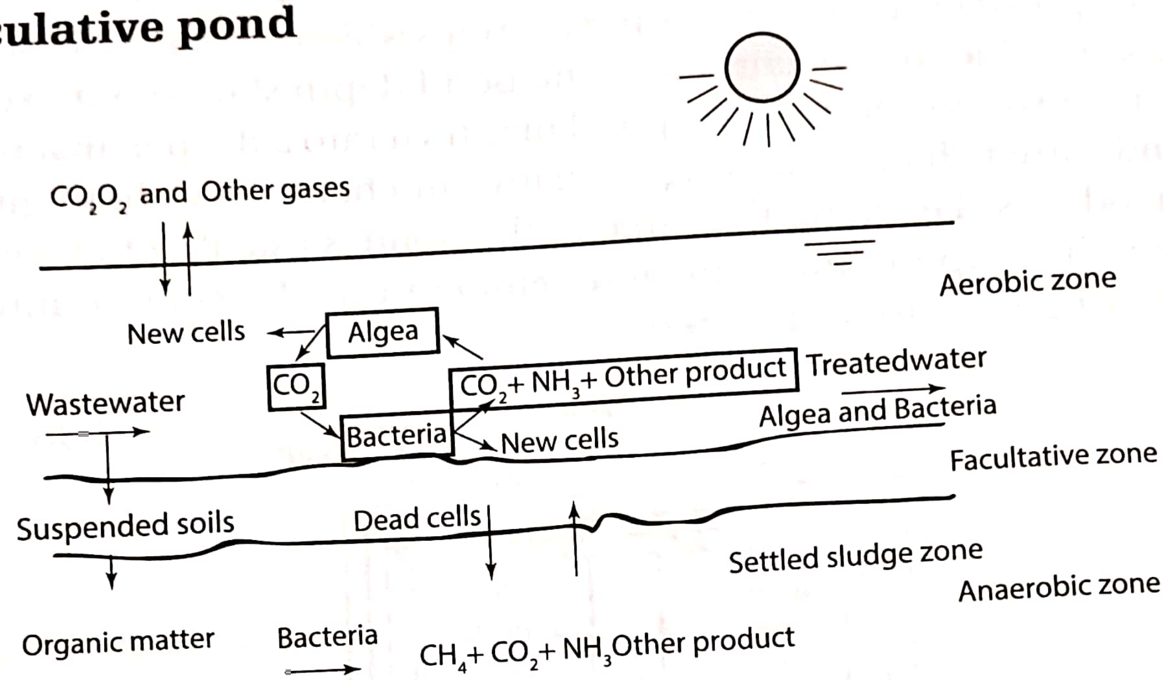


Figure 9.12 Oxidation Ditch intermittent flow type

Most of the ponds exist in facultative nature. Three zones exist in this type of ponds (Figure 9.12). The top zone is an aerobic zone in which the algal photosynthesis and aerobic biodegradation takes place. In the bottom zone, the organic matter present in wastewater and cells generated in aerobic zone settle down and undergo anaerobic decomposition. The intermediate zone is partly aerobic and partly anaerobic. The decomposition of organic waste in this zone is carried out by facultative bacteria. The nuisance associated with the anaerobic reaction is eliminated due to the presence of top aerobic zone. Maintenance of an aerobic condition at top layer is important for proper functioning of facultative stabilization pond, and it depends on solar radiation, wastewater characteristics, BOD loading and temperature. Performance of these ponds is comparable with conventional wastewater treatment.

Constructional details

A typical plan of an oxidation pond is shown in figure 9.13. It is an earthen pond, dug into the ground, with shallow depth. Oxidation ponds are rectangular in shape (L/B=2-3/1) having side slopes (1:1.5) and are constructed by building embankments of earth. They are of shallow depth usually 0.9-1.5m and as such effective in permitting penetration of sunlight to all parts of the waste water encouraging algal growth. Influents is applied in the middle of pond and allowed to be spread by the action of wind currents which prevents any odour nuisance due to concentration.The pond depth should not exceed 1.8 m or so, as otherwise the pond may turn into a deeper anaerobic pond rather than remaining faculative in character without giving foul odours. The detention time in the pond is usually 2 to 6 weeks, depending upon sun light and temperature. In cold countries, higher figure is to be adopted.

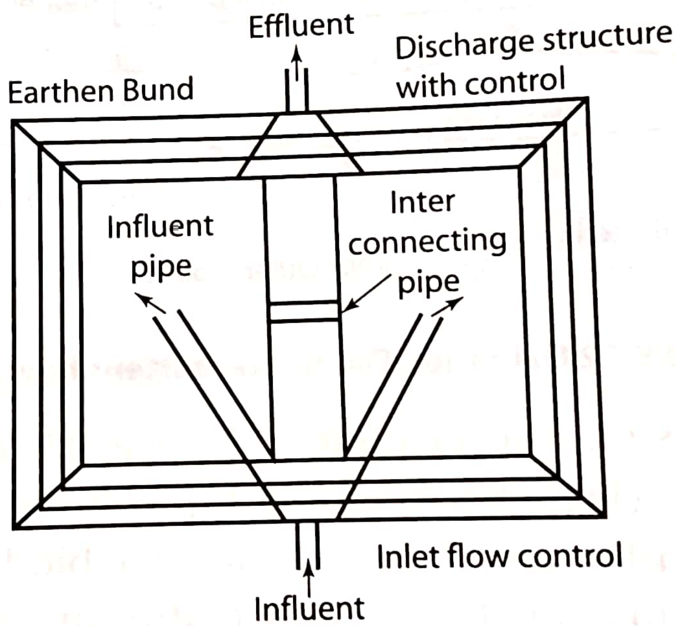


Figure 9.13 Plan of oxidation pond

Advantages of oxidation ponds

1. Low cost
2. Quickness of construction
3. Easy maintenance
4. High efficiency of BOD removal

Disadvantages of oxidation ponds

Nuisance due to mosquito breeding and odours.

Design considerations

(1) Organic loading

- 300 to 150 kg/ha/day [hot tropical countries like India]
- 90 to 60 kg/ha/day [colder countries situated at higher latitude

(2) Area of one unit - 0.5 to 1.0 ha.

(3) $\frac{L}{B}$ ratio = 2

(4) Depth of the tank = 1 to 1.5m

(5) Free board = 1m

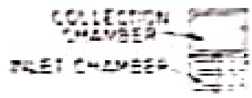
(6) Detention time - 70 to 42 days.

$$DT = \frac{1}{K_D} \log_{10}\left[\frac{L}{L-Y}\right]$$

where L = BOD of effluent entering the pond.

Y = BOD removed (90 to 95% of L)

K_D = Deoxygenation constant

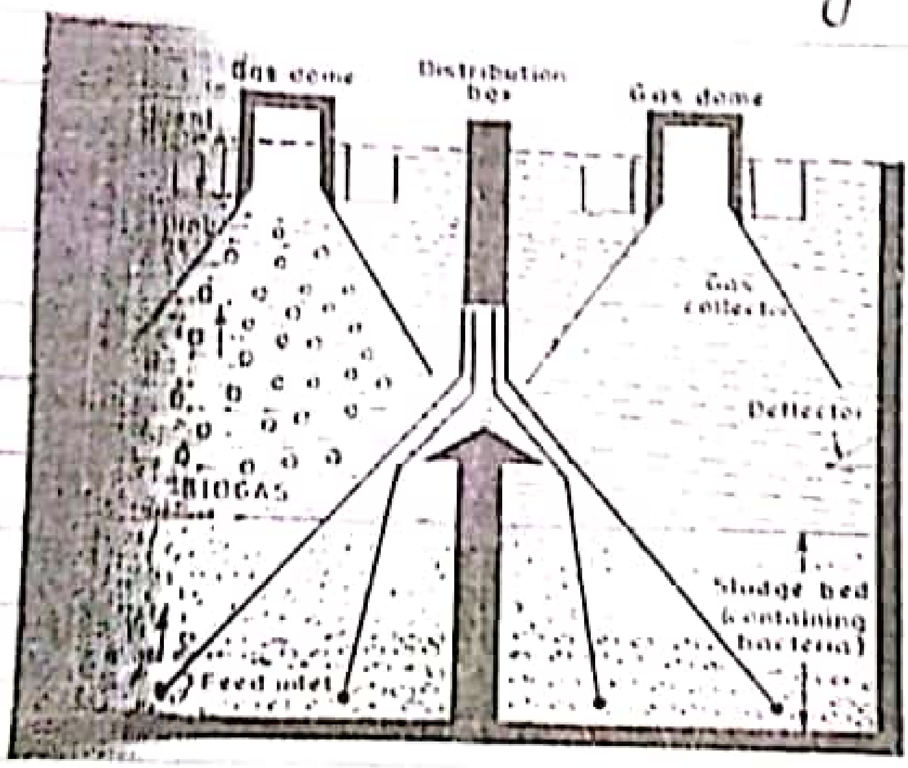


Upflow Anaerobic sludge blanket reactors (UASB reactor)

→ The wastewater flows upwards through a layer of very active sludge to cause anaerobic digestion of organics of the wastewater.

→ At the top of the reactor, three-phase separation between gas-liquid-solid takes place.

→ Any biomass leaving the reaction zone is directly recirculated from the settling zone.



This reactor consists of an upflowing treatment tank, provided with a feed inlet distribution system at the tank bottom.

A gas-solid-liquid separator device is provided at the top to help to provide a quiescent zone at the top of the reactor.

reacto...

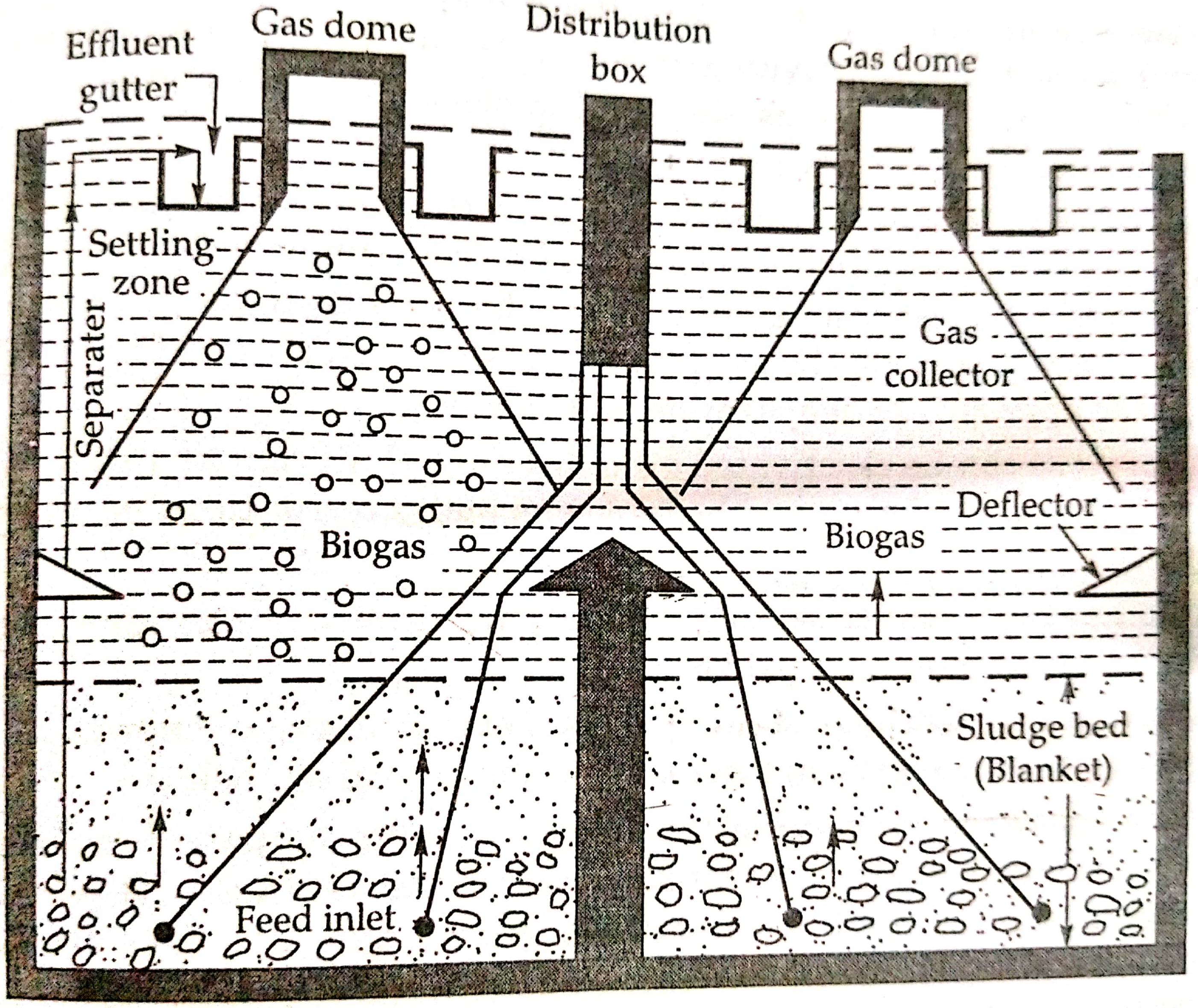


Fig. 9.66. X-section of a typical UASB reactor.

- The wastewater enters the tank from the bottom and flows upwards through the sludge bed, which gets formed during the process itself.
- The sludge bed develops micro-organisms capable of flourishing in an oxygen deficient environment.
- The sludge bed traps the suspended organics of the ~~upcoming~~ upmoving wastewater.
- The suspended solids trapped in the sludge bed are degraded by the anaerobic bacteria producing methane and carbondioxide.
- The biogas produced during the anaerobic decomposition helps in providing gentle mixing and stirring of the biomass. This increase the efficiency of decomposition, reducing the BOD and suspended solids of the wastewater.
- The methane or biogas is collected at the top of tank in a gas collector. It can be used as a gas for domestic or industrial use.
- The treated effluent is collected in gutters and discharged out of the reactor. ~~The~~
- The sludge is periodically shifted into the drying beds to be used as a soil enricher.

Design considerations.

→ Upflow velocity (Superficial velocity)

→ It is based on the flow rate and reactor area

→ [redacted]

→ The upflow velocity is equal to the feed rate divided by the reactor cross-section area:

$$v = \frac{Q}{A}$$

where v = design upflow superficial velocity, m/h

A = reactor cross section area, m^2

Q = Influent flow rate, m^3/h

II) Reactor volume and dimensions.

* The nominal liquid volume of the reactor,

$$V_n = \frac{Q S_o}{L_{org}}$$

Where Q = Influent flow rate, m^3/h

S_o = Influent COD, kg COD/m^3

L_{org} = organic loading rate, kg COD/$m^3 \cdot d$

* Total liquid volume of the reactor exclusive of the gas storage area, $V_L = \frac{V_n}{E}$

where, V_n = nominal liquid volume of reactor, m^3
E = Effectiveness factor [0.8 to 0.9]

* Area of the reactor, $A = \frac{Q}{v}$

* Height of the reactor, $H_L = \frac{V_L}{A}$

* Total height of the reactor

$$H_T = H_L + H_G$$

where H_L = reactor height based on liquid volume
H_G = reactor height to accomodate gas collection and storage, m
[2.5m to 3m]

III Physical features:

(i) Feed inlet
(ii) Gas separation
(iii) Gas collection
(iv) Effluent withdrawal.

Advantages of UASB System. The various advantages offered by UASB system over the Conventional Aerobic systems are given below :

(*i*) The space requirement of the system is quite comparable to that of an Activated sludge system ; *i.e.* about 0.5 acres per MLD, as compared to 2.5 acres per MLD required for Oxidation ponds, and 1.5 acres for Aerated lagoons.

(*ii*) The capital cost investment of such a plant is about ₹ 20 lakh/MLD as compared to about ₹ 35 lakh/MLD for an Activated sludge plant, ₹ 7.5 lakh/MLD for Oxidation ponds, and ₹ 15 lakh/MLD for Aerated lagoons.

(*iii*) The system requires lesser and simpler electromagnetic parts as compared to the ones required in an Activated sludge plant, leading to lower O and M (Operation and Maintenance) Costs.

(*iv*) Electricity consumption in this system, like all anaerobic systems, is quite low, and the system is quite capable of withstanding long power failures.

(*v*) The sludge production in this system is low, and the produced sludge is having quick dewatering characteristics.

(*vi*) The system enables quicker sludge digestion, as compared to the conventional digestors.

(*vii*) Biogas is produced in the system as a by-product, which can be used to produce electricity to run the system.

Limitations or Drawbacks of UASB System. The various drawbacks of the UASB system as compared to the Conventional Aerobic system are given below :

(*i*) The system helps to lower only two parameters of wastewaters; *i.e.* (*a*) BOD ; and (*b*) Suspended solids. Eventually, the system does not help in the removal of toxic pollutants, like heavy metals, which may be present in some of the wastewaters. The USAB system will therefore have to be supported by subsidiary disposal system to remove the toxic pollutants, if present in the wastewater.

(*ii*) Like all other anaerobic high rate systems, UASB reactor also requires larger quantity of organic matter as compared to the aerobic reactors, because the growth of aerobic bacteria per unit of organic matter is about 10—20 times the growth of anaerobes. In order to support microbial growth and metabolism in UASB system, therefore, 20 to 30 times more of organic matter has to be metabolised, as compared to that in Aerobic systems. For the success of UASB, it therefore becomes necessary to ensure the presence of at least 10% of suspended solids in the wastewater. This requirement factor can not always be met by all types of wastewaters.

(*iii*) Some of the wastewaters may contain minerals, which may interfere with the efficiency of the anaerobic microbes. The system also does not respond well to the wastewaters of tanneries, which contain more than 500 auxiliary chemicals, offering varying response to the UASB technology.

(*iv*) The acids produced during the breakdown of organic matter in a UASB reactor, may cause corrosion of the reactor.

(*v*) The efficiency of BOD and S.S. removal is a little bit low, as compared to that in an Activated sludge plant. With generally adopted organic loadings of 1.0—2.0 kg COD/m^3.d in UASB reactors, the achieved efficiency varies between 50 to 70% only. The effluent BOD of municipal wastewaters treated in UASB reactor system, will therefore be higher.

Say for example, the effluent BOD may be about 50 mg/l for influent BOD of 200 mg/l. For concentrated influents, the effluents BOD may still be higher. Direct disposal of effluent containing such high BOD may not always be permissible. Depending upon the situation, the effluent from a UASB system may have to be given further aerobic treatment in Aerated lagoons, or Oxidation ponds, or Filters. Where, enough space is not available, the post treatment may consist of using a *holding pond* of 1 day detention time followed by **fish pond/aqua culture pond.**

(*vi*) Pre-treatment of wastewater with *screening* and *grit removal*, are usually found necessary for direct anaerobic treatment.

(*vii*) The system responds well in high temperature climate areas, because the activity of methanogenic bacteria is strongly influenced by temperature, which approximately doubles for every 10°C rise in temperature in the range of 18°C to 38°C. However, high micro-organism concentration in high rate anaerobic reactors like a UASB, compensates the decreased activity of the anaerobic organisms at the lower temperatures.

(*viii*) The methanogenic bacteria do require iron, cobalt, nickel and sulphide, in addition to nitrogen and phosphorous. These elements are generally present in municipal wastewaters, but may have to be added to anaerobically treat some specific industrial wastewaters, which may have deficiency of these elements.

1. Design a UASB reactor treating an industrial wastewater with following data. Also calculate the detension time.

Flowrate = $1000\ m^3/d$

sCOD = $2000\ g/m^3$

L_{org} = $10\ kg\ sCOD/m^3.d$

Soln:- Nominal liquid volume, $V_n = \frac{QS_o}{L_{org}}$

$= \frac{(1000\ m^3/d)(2\ kg\ sCOD/m^3)}{(10\ kg\ sCOD/m^3.d)}$

$= \underline{200\ m^3}$

Total reactor liquid volume, $V_L = \frac{V_n}{E}$

$= \frac{200}{0.85}$ [Assume E = 0.85]

$= \underline{235\ m^3}$

Area of the reactor, $A = \frac{Q}{v}$ [$v = 1.5\ m/h$]

$= \frac{1000\ m^3/d}{(1.5\ m/h)\ 24} = \underline{27.8\ m^2}$

$\frac{\pi}{4} D^2 = 27.8$

$D = 6\ m$

Height of the reactor, $H_L = \frac{V_L}{A}$

$= \frac{235}{27.8} = 8.4\ m$

Total height of the reactor

$H_T = H_L + H_G$ [Assume $H_G = 2.5\ m$]

$= 8.4 + 2.5$

$= \underline{10.9\ m}$

Reactor dimensions :- Diameter = 6 m

Height = 10.9 m

Detention time of reactor $= \frac{V_L}{Q} = \frac{235 \times 24}{1000}$

$= \underline{5.64\ hrs}$

OXIDATION POND

Design Considerations

- Organic loading
  - → 150 - 300 kg/ha/day for hot tropical countries like India
  - → 60 - 90 kg/ha/day for colder countries situated at higher latitude.
- Area of one unit = 0.5 - 1 ha
- $L/B = 2$
- Depth = 1 - 1.5 m
- Freeboard = 1 m.
- Detention time = 7 - 42 days
- $DT = \frac{1}{K_D} \log\left[\frac{L}{L-Y}\right]$

L = BOD of sewage entering the pond.
Y = BOD removed
K_D = Deoxygenation constant.

Problem.

1. Design an oxidation pond for treating domestic sewage of 15,000 persons supplied with 200 l/c/day. The BOD and suspended solids are each of 400 mg/L. Permissible organic loading for the pond is 600 kg/ha/day. Detention period not exceeds 6 days. Assume L/B = 2 and operational depth as 1.8 m. Assume any other suitable data.

Ans. Quantity of sewage to be treated per day $= 15000 \times 200$
$= 30 \times 10^5$ l
$= 3000$ m³

Total BOD of sewage/day $= 400 \frac{mg}{l} \times 3000$ m³
$= 400 \times 10^{-6} \times 3000 \times 10^3$ kg/day
$= 1200$ kg/day

∴ Surface area required = $\frac{\text{Total BOD}}{\text{organic loading}}$

$= \frac{1200 \text{ kg/day}}{600 \text{ kg/ha/day}}$ → Given

$= 2$ ha

$= 2 \times 10^4$ m²

L/B = 2 (Given)

Area = LB

$= 2B^2 = 2 \times 10^4$

B = 100 m

L = 200 m

Using a tank of eff. depth as 1.8 m we have the provided capacity $= 200 \times 100 \times 1.8$

$= 36000$ m³

Capacity = sewage flow/day × Detention time (DP)

∴ DP $= \frac{36000}{3000} = 12$ days

But given that DP ≯ 6 days.

∴ Use two oxidation ponds with L = 200 m, B = 100 m, D = 1.8 m

Assuming a freeboard of 1 m

Overall depth $= 1.8 + 1 = 2.8$ m.

Hence provide two oxidation ponds each of size

L = 200 m

B = 100 m

D = 2.8 m with

Detention time = 6 days.

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Design of Inlet Pipe

Assuming an average velocity of sewage as 0.9 m/s and daily flow for 8 hrs. only.

Discharge for one pond = $\frac{3000/2}{8 \times 60 \times 60}$ (m^3/day for one pond; 8 hrs)

$= 0.052\ m^3/s$

Area of inlet pipe $A = Q/V$

$= \frac{0.052}{0.9} = 0.0579\ m^2$

$\pi/4\ d^2 = 597\ cm^2$

$\therefore\ d = 27.6 \simeq 28\ cm$.

Design of Outlet Pipe

Dia. of outlet pipe = 1.5 × Dia. of inlet pipe

= 1.5 × 28

= 42 cm.

DESIGN OF UPFLOW ANAEROBIC SLUDGE BLANKET REACTORS (UASB)

Design Considerations

- **Upflow velocity** (superficial velocity)

$$v = Q/A = \frac{\text{Influent flow rate} \quad m^3/h}{\text{Reactor c/s area} \quad m^2}$$

Temporary peaks: 2 m/h to 6 m/h

- **Reactor volume & Dimensions**

→ Nominal liquid volume of the reactor $V_n = \frac{Q S_o}{L_{org}}$

Q ⇒ Influent flowrate, m^3/h

S_o ⇒ Influent COD, $kg\ COD/m^3$

L_{org} ⇒ Organic loading rate $kg\ COD/m^3.day$.

→ Total liquid volume of the reactor exclusive of gas storage area $V_L = \frac{V_n}{E}$

E ⇒ Effectiveness factor (0.8–0.9)

→ Area of the reactor $A = \frac{Q}{v}$

→ Height of the reactor $H_L = V_L/A$ (H_L based on liquid volume)

→ Total height of the reactor = $H_T = H_L + H_G$

(H_G based on gas collection & storage 2.5 – 3m)

- **Physical features**

→ Feed inlet

→ Gas separation

→ Gas collection

→ Effluent withdrawal

1. Design an UASB reactor treating an industrial wastewater with following data. Also calculate the detention time.

Flowrate = 1000 m^3/d

COD = 2000 g/m^3

Load = 10 kg COD/m^3.d.

Ans. Nominal liquid volume $V_n = \frac{Q S_0}{Load}$

$= \frac{1000 \frac{m^3}{day} \times \frac{2000}{1000} kg/m^3}{10 \ kg/m^3.day}$

$= 200 \ m^3$

Total liquid volume $V_L = V_n / E$ (Let $E = 0.85$)

$= 200/0.85$

$= 235 \ m^3$

Area of the reactor $A = Q/v$ (Let $v = 1.5$ m/hr)

$= \frac{1000 \ m^3/day}{1.5 \ m/hr \times 24}$

$= 27.8 \ m^2$

$\pi/4 \ D^2 = 27.8$

$D \approx 6$ m

Height of the reactor $= H_L = V_L/A$

$= \frac{235}{27.8} = 8.4$ m

Total height of reactor $= H_T = H_L + H_G$ (Let $H_G = 2.5$ m)

$= 8.4 + 2.5$

$= 10.9$ m

∴ Reactor Dimensions are,

Diameter = 6 m

Height = 10.9 m

Detention time of reactor $= \frac{V_L}{Q} = \frac{235 \ m^3 \times 24 \ hr}{1000 \ m^3/day} = 5.64$ hr

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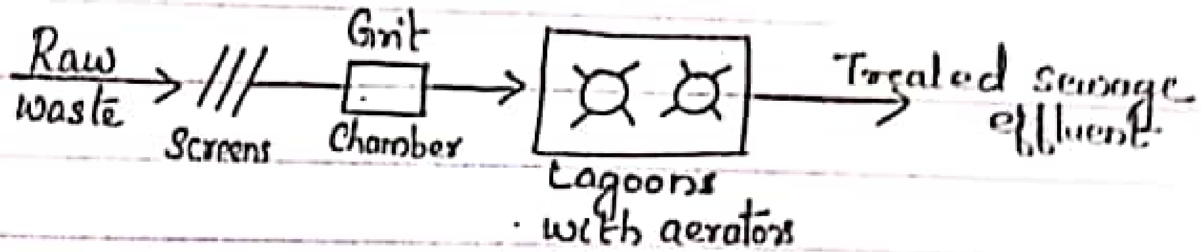
Aerated Lagoons.

Aerated lagoon is a deeper oxidation pond, with oxygen introduced by mechanical aerators rather than relying on the photosynthetic oxygen production alone. As these ponds are deeper than the oxidation ponds and as they are artificially aerated, less detention time and areas are required.

→ The depth of basin ranges between 2.4 to 3.6m.
→ Detention time - 1 to 10 hours.
→ The land area required is about 5 to 10% of that required for an equivalent oxidation pond.
→ Efficiency obtained ranges b/w 65 to 90%.
→ The aerated lagoons are frequently used for treating industrial waste waters.



→ Depending upon the extent of mixing, the lagoons may be classified as:

i) Complete mix lagoons (aerobic aerated lagoons)

→ In this type, greater amount of aeration is provided to keep all the solids in suspension due to which the entire pond is aerobic.
It consists of two units.
In the first, the mechanical surface aerators are

so designed that solids do not settle to the bottom of the tank, while the second unit is used as settling tank for the removal of suspended solids.

→ BOD removal – 75 to 85%

(ii) Partially mixed lagoons (facultative aerated lagoons)

→ The lagoons are operated at a low rate of aeration not adequate to keep all the solids in suspension but enough to keep top layers aerobic.

→ The sewage solids tend to settle down and anaerobic bottom is established.

→ A large portion of incoming solids and the biological solids produced within the lagoon settle to the bottom of the tank where anerobic decomposition takes place.

→ The effluent from this type of tanks is more stable.

→ BOD removal – 75 to 90%.

Septic Tank

Septic tank may be defined as a primary sedimentation tank with a longer detention time of 12 to 36 hrs, and with extra provision for digestion of settled sludge. Digestion of sludge is carried out by anaerobic decomposition process which results in release of foul gases.

The settling tank directly admits raw sewage & removes about 60-70% of organic matter.

The effluent will be foul in nature and hence it should be disposed off by subsurface irrigation, soak pits or by trickling filter before disposing it into water bodies.

The settled sludge and oils & greasy matter floating on the surface as scum is retained for several months for the process of sludge digestion.

The digested sludge from the tank is periodically removed at intervals of 6 to 12 months.

Septic tanks are provided in areas ~~about~~ where sewers have not been laid.

Parts of the septic tank

- T-shaped inlets & outlets are provided to prevent direct currents between the tank inlet and outlet
- Baffles walls can also be provided to serve the above purpose.
- The tank is covered at top with a R.C.C slab.

→ Manhole covers are provided in the top slab, so as to permit inspection and maintenance.
→ A vent pipe is provided to remove foul gases.

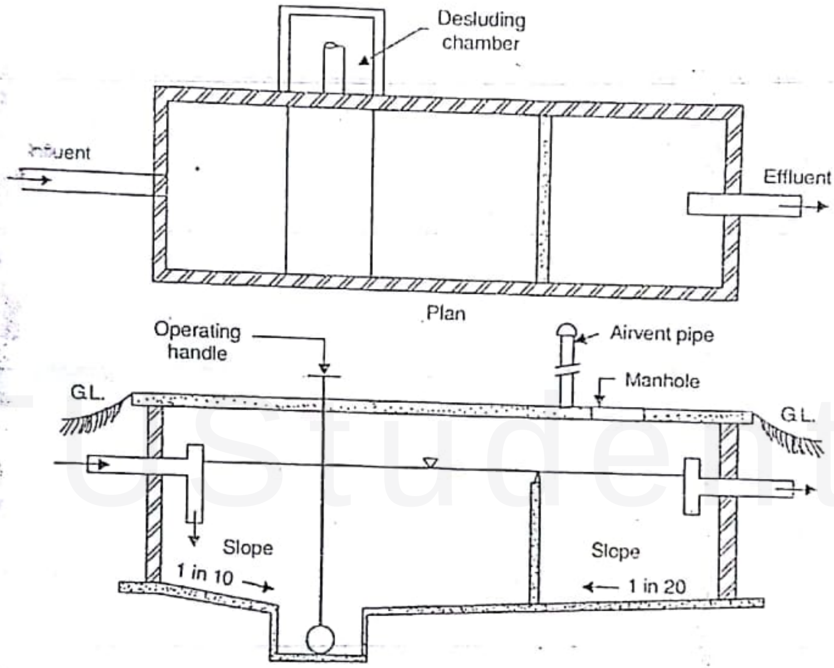


Fig. 7.22 Septic tank

Design considerations

(1) Sewage flow = 40 to 70 L/capita/day [only water closets are connected to septic tank].

= 90 to 150 L/capita/day [water closets and sullage pipes are connected]

(2) Rate of accumulation of sludge = 30 L/person/year.

(3) Minimum capacity for about 8 to 10 persons
→ 2250 L [when all liquid waste are discharged into the tank]
→ 1400 L [when only water closet wastes are discharged]

(4) Freeboard – 0.3 m

(5) Detention Period — 12 to 36 hours
(Usually 24 hrs is adopted)

(6) Length to width ratio – Length is 2 to 3 times width
– Width should not be less than 90 cm.

(7) Depth of the tank – 1.2 to 1.8 m.

Disposal of effluent from the septic tanks.

The effluent coming out from septic tank contains large amount of putrescible organic matter and its BOD is quite high. So it should be disposed of carefully, so as to cause minimum nuisance or risk to the health of the people. The following three methods of disposal of septic tank effluent are usually adopted:

(i) Soil absorption system
(ii) Biological filters
(iii) Upflow anaerobic filters.

Soil absorption system

- → It involves the disposal of effluent on land.
- → It can be adopted only when sufficient land is available and soil is sufficiently porous
- → Percolation rate should not exceed 60 minutes.
- → The percolation rate of a soil is defined as the time in minutes required for seepage of water through that ground by 1 cm.
- → The soil absorption system may be of following types:
  - (a) Seepage pit or soak pit
  - (b) dispersion pit.

Disposal in soak pits

→ A soak pit is a circular covered pit, through which the effluent is allowed to be soaked or absorbed into the surrounding soil.

- → The soak pit may either be filled with stone aggregates or may be kept empty.
- → When the soak pit is empty, the pit is lined with brick, stone or concrete block with dry open joints. The brick lining is supported below the inlet level by atleast 75 mm thick backing of the coarse aggregate.
- → If the soakpit is filled with stone aggregates, no lining is required except for the top masonry ring which is constructed to prevent damage by flooding of the pit by surface runoff.

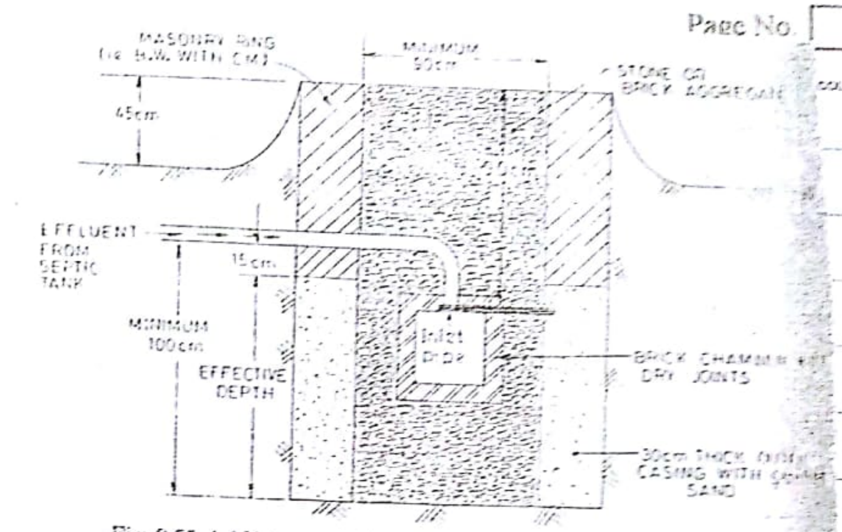


Fig. 9.55. (a) Unlined soak pit filled with stone or brick aggregate.

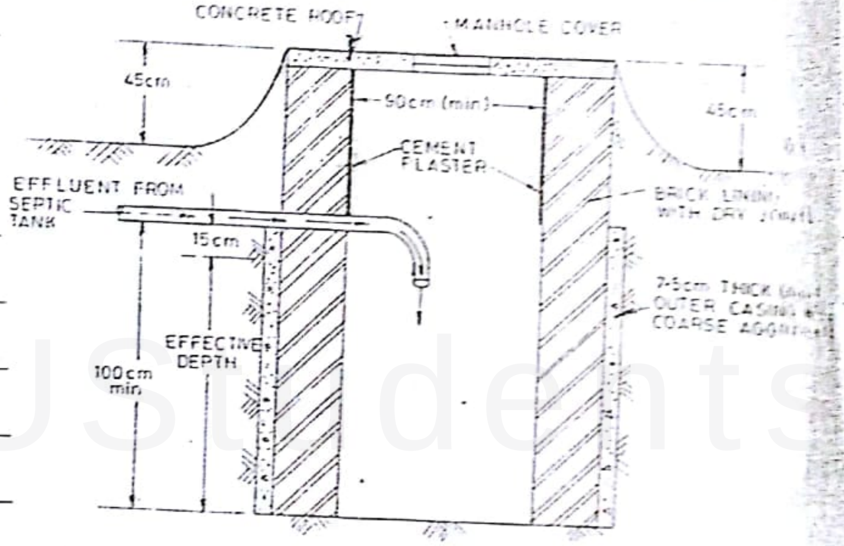


Fig. 9.55. (b) Lined soak pit.

Disposal in absorption trenches.

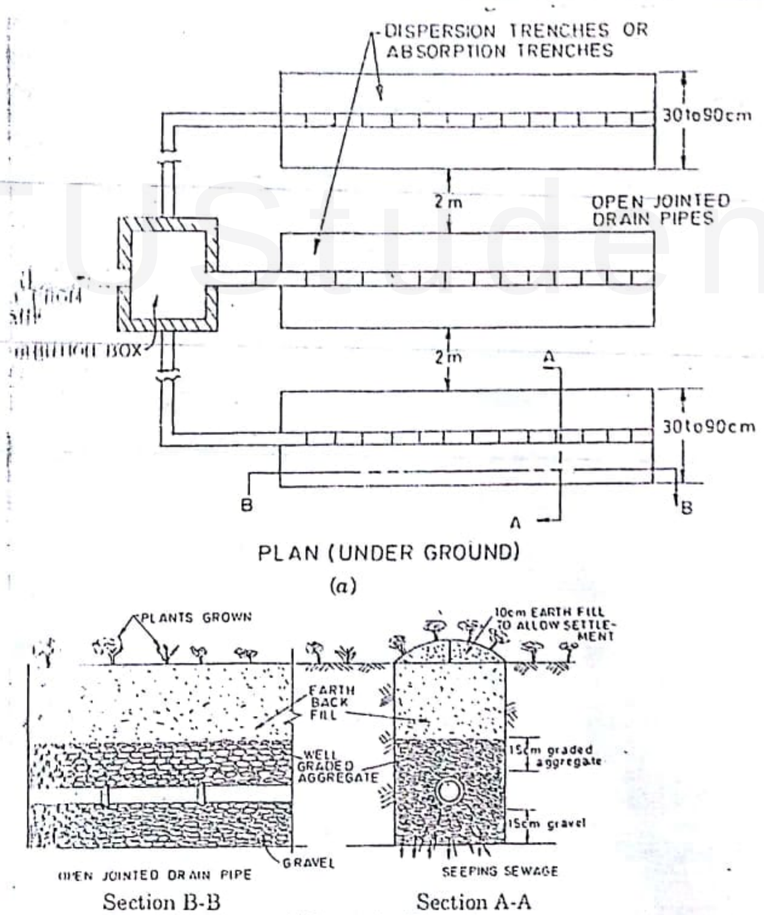
In this method, the septic tank effluent is allowed to enter into a masonry chamber (Distribution box), from where it is uniformly distributed through an underground network of open jointed pipes into absorption trenches called the dispersion trenches.

The suspended organic matter present in the effluent will be absorbed in the absorption trenches which are filled with gravel and well graded aggregate. The clearer water will seep down to the water table

Plants are usually grown on the top of the absorption trenches, which get their irrigation water requirement fulfilled by capillarity from the seeping water in the absorption trenches.

The value of maximum allowable rate of effluent application is given by:

$$Q = \frac{204}{\sqrt{t}}$$ where t = standard percolation rate in minutes.



(b) Sections of Dispersion Trench.

Fig. Absorption Trench Method for disposal of septic tank effluent.

Biological filters:

→ They are suitable for treating septic tank effluent where the soil is relatively impervious or in waterlogged areas or where limited area is available.

→ In this filter, the effluent from the septic tank is brought into contact with a suitable medium, the surfaces of which become coated with an organic film.

→ The film assimilates and oxidises much of the polluting matter through the agency of microorganisms.

Upflow anaerobic filters:

→ They are operating under submerged condition.

→ They are used for giving secondary treatment to the effluent of septic tank before its disposal.

→ In such filters, the septic tank effluent is introduced from the bottom and the microbial growth is retained on the stone media. Making possible higher rates and efficient digestion.

→ BOD removals of 70% can be expected and the effluent is clear and free from odour and nuisance.

Advantages of septic tank.

1. It can be easily constructed and donot require a skilled supervision during construction.
2. The cost is reasonable as compared to the advantages it offers.
3. The performance of a properly constructed septic tank is very good. It can remove about 90% of BOD and about 80% of suspended solids.
4. The sludge volume to be disposed of is quite less.
5. The effluent from the septic tank can be disposed of on land in a soak pit without much trouble.

Disadvantages of septic tank.

1. If the tank is not properly functioning, the effluent is dark and foul-smelling. It is even worse than the influent.
2. They require too large sizes for serving many people.
3. The leakage of gases through the top of septic tanks leads to air pollution.
4. Periodical cleaning, removal and disposal of sludge remains a tedious problem.
5. The working of a septic tank is unpredictable and non-uniform.

Q. (1) Design a septic tank for the following data.
No: of people = 100
Sewage/capita/day = 120 litres
De-sludging period = 1 year.
Length : Width = 4:1
What would be the size of its soak well if the effluent from this septic tank is to be discharged in it. Assume percolation rate through the soak well to be 1250 l/m³/d.

Ans:- Quantity of sewage produced = 120 × 100
= 12 m³/day.

Assume detention period = 24 hrs.
Capacity of the tank = Discharge × detention period

$= \frac{12 \, m^3}{24} \times 24$

$= 12 \, m^3$

Desludging period = 1 year
Assume rate of deposition of sludge as 30 L/capita/year
Quantity of sludge produced = 30 × 100 × 1
= 3 m³

Total capacity of tank = 12 + 3 = 15 m³

Assume depth of tank = 1.5 m
∴ area of the tank = $\frac{15}{1.5} = 10 \, m^2$

$L:B = 4:1$

$L = 4B$

$B \times 4B = 10$

$B = 1.58 \approx 1.6\,m$

$L = 6.4\,m$

Freeboard = 0.3 m

Overall depth = 1.5 + 0.3 = 1.8 m

Hence use a tank of size 6.4 m × 1.6 × 1.8 m

Design of soak well

Percolation rate = 1250 litres/m^3/day

Sewage outflow = 12 m^3/day. = 12000 L/day

$$\text{Volume required for soak well} = \frac{\text{Sewage flow}}{\text{Percolation rate}}$$

$$= \frac{12000\ L/day}{1250\ L/m^3/day}$$

$$= 9.6\,m^3$$

Let the depth of soak well = 2 m

$$\text{Area of soak well} = \frac{9.6}{2} = 4.8\,m^2$$

$$\frac{\pi}{4} D^2 = 4.8$$

Diameter of soak well, D = 2.47 m ≈ 2.5 m

(2) Design the absorption field system for the disposal of septic tank effluent for a population of 100 persons with sewage flow rate of 135 lpcd. The percolation rate for the percolation test carried out at the site of the absorption field may be taken as 3 minutes.

Soln: Total sewage flow = 135×100

$= 13500\ l/day$

For dispersion trench, the maximum rate of effluent application is given by

$$Q = \frac{204}{\sqrt{t}}$$

$$= \frac{204}{\sqrt{3}} = 117.78\ l/d/m^2$$

Area of trench required $= \frac{13500}{117.78}$

$= 114.62\ m^2$

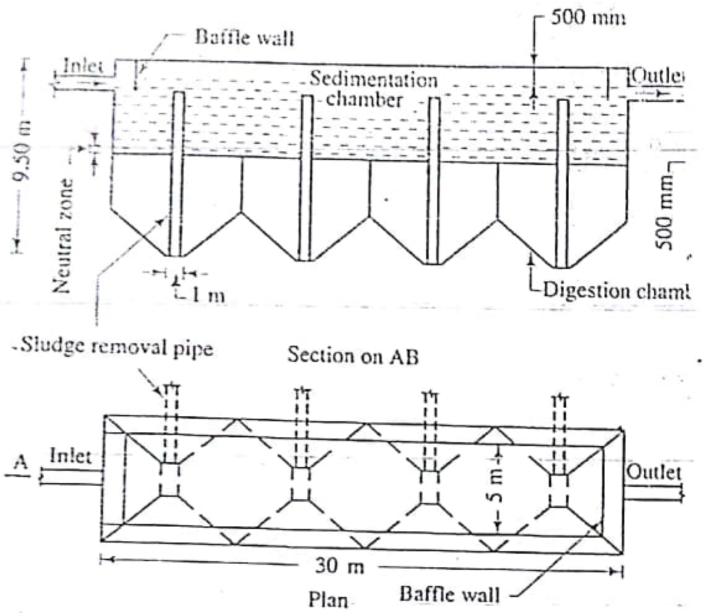
Using 0.75 m bottom width of the trench,

Length of the trench $= \frac{114.62}{0.75} = 153\ m$

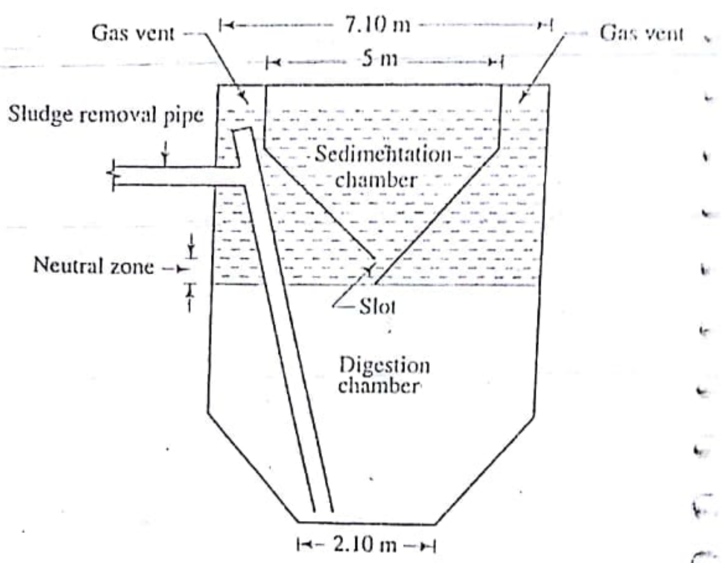
Depending on the availability of ~~us~~ space, we may use 3 no. of trenches, each of length

$\frac{153}{3} = 51\ m$

Imhoff tank.



Imhoff tank
FIG. 31-5



Imhoff tank

→ Imhoff tank is an improvement over septic tank in which the incoming sewage is not allowed to get mixed up with the sludge produced and the outgoing effluent is not allowed to carry with it large amount of organic load.

→ They are two-storey digestion tanks

→ It consists of a rectangular tank with two chambers.

→ The upper chamber is called the sedimentation chamber. Sewage flows through this chamber at a very low velocity.

The lower chamber is the digestion chamber, in which the sludge gets digested due to anaerobic decomposition.

→ An entrance slot is provided at the bottom of sedimentation chamber to pass solids into digestion chamber.

→ The gas vent (scum chamber) is also provided above the digestion chamber and along side the sedimentation chamber to take care of the gases escaping to the surface.

→ The space between bottom of sedimentation chamber and top of digestion chamber is called neutral zone. It prevents the entry of sludge or scum from digestion chamber into the sedimentation chamber.

→ The digestion chamber is divided into a number of interconnected compartments

→ The bottom of each digestion compartment is made up in the form of an inverted cone or hoppers with sides sloping 1:1

→ The digested sludge from the bottom of the hoppers is removed periodically through desludging pipes.

Design considerations:

A) Sedimentation chamber:

→ Rectangular in shape

→ Detention period = 2 to 4 hours (usually 2 hrs)

→ Flowing through velocity should not be more than 0.3 m/minute.

→ Surface loading
- should not exceed 30000 litres/m² of plan area/day
- may be increased to about 45000 L/m²/day for effluent coming from activated sludge plant or where recirculation is adopted.

→ Length of tank
- Should not exceed 30m.

→ Length to width ratio
- 3 to 5.

→ Depth of the chamber ; Total depth of
- 3 to 3.5m ; Imhoff tank = 9 to 11 m

→ Free board
- 45 to 50cm.

(B) Digestion chamber.

Minimum capacity - 57 Litres per capita.

For warmer climates capacity is reduced to about 35 to 45 L/capita.

(C) Gas vent / Scum chamber

The surface area of the scum chamber should be about 25 to 30% of the area of the horizontal projection of the top of the digestion chamber.

(D) Neutral zone

Depth varies from 45 to 50cm.

Q. Design an imhoff tank to treat the sewage from a small town with 30,000 population. The rate of sewage may be assumed as 150 L/capita/day.

Design of sedimentation chamber.

Total quantity of sewage = 30,000 × 150
= 4500 m^3/day.

Assume detention period as 2 hrs

Volume of sewage in 2 hrs = $\frac{4500}{24} \times 2$ = 375 m^3

Assuming effective depth of 2.2m and width of 4.3m

Length of the sedimentation chamber

= $\frac{375}{2.2 \times 4.3}$ = 39.64 m ≈ 40 m

Since this length is too large for a single tank, adopt 2 tank units each of length 20 m & width 4.3 m.

Check : $\frac{L}{B} = \frac{20}{4.3}$ = 4.65 b/w 3 & 5 ∴ OK

Discharge through each unit = $\frac{1}{2}$ × 4500 = 2250 m^3/day
= 2.25 ML/day

Check for velocity

Velocity = $\frac{\text{Length}}{\text{Detention time}}$ = $\frac{20}{2 \times 60}$ = 0.167 m/min
< 0.2 m/min
∴ OK

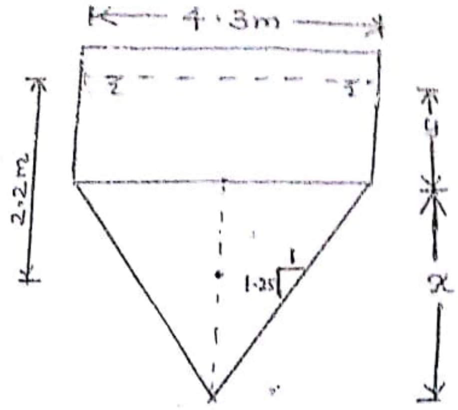
Check for surface loading

= $\frac{Q}{L \times B}$ = $\frac{2250 \times 10^3}{20 \times 4.3}$ = 26162.79 L/m^2/day
< 30,000 L/m^2/day ∴ OK

With 4.3 m width & bottom sides
Sloping 1 H : 1.25 V
Height of sloping bottom,

$$\frac{x}{1.25} = 2.15$$

$$x = 2.69\,m$$



With effective depth of 2.2m, the height of the vertical portion below the liquid surface (y) is given by

$$y = 2.2 - \frac{2.69^{*}}{2} = 0.855\,m \simeq 0.86\,m$$

[*Effective depth of triangular portion will be half, to make it equivalent to a rectangular section]

Assuming 0.45m for the free board
Total depth of sedimentation chamber upto bottom at the entrance of the slot

$$= 0.45 + 2.69 + 0.86$$
$$= \underline{4m}$$

Design of Gas vent and neutral zone.

Provide a neutral zone of 0.45m below the depth of 4m.

The tank is of 20m length, but below this 4m depth, it shall be divided into no: of compartments, each of length $= \frac{20}{4} = 5m$

Using an overall width of 6.5m, total width of gas vent assuming 15cm thickness of chamber walls is obtained as

$$6.5 - 4.3 - (2 \times 0.15) = 1.9\,m$$

This width should be about 25 to 30% of the total width of the tank.

Check :- $\frac{1.9}{6.5} \times 100 = 29.2\%$ b/w 25 & 30% ∴ OK.

Hence $\frac{1.9}{2} = 0.95\,m$ width of gas vent will be provided on either side of sedimentation chamber.

Design of digestion chamber:-

Assuming the capacity of the digestion chamber @ 40 litres/capita.

Capacity of the digestion chamber

$= 30000 \times 40$

$= 1200\,m^3$

Considering four compartments or units per tank (8 units in both tanks with 6.5m width)

Capacity of each unit or compartment

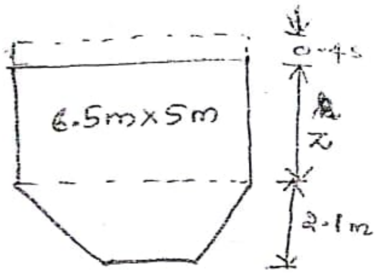
$= \frac{1200}{8} = 150\,m^3$

Assuming the depth of each hopper as 2.1 m, side slopes 1:1

Bottom dimension

$5 - 2.1 \times 2 = 0.8 m$

$6.5 - 2.1 \times 2 = 2.3 m$

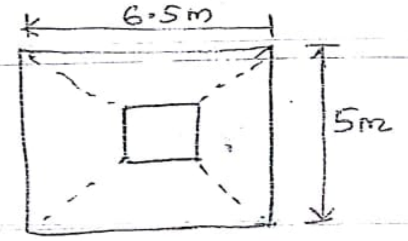


Capacity of each hopper

$= \frac{h}{3}\left[A_1 + A_2 + \sqrt{A_1 A_2}\right]$

$= \frac{2.1}{3}\left[32.5 + 1.84 + \sqrt{32.5 \times 1.84}\right]$

$= 29.45 m^3$



$h = 2.1$

$A_1 = 6.5 \times 5 = 32.5 m^2$

$A_2 = 2.3 \times 0.8 = 1.84 m^2$

Capacity of rectangular portion

$= 150 - 29.45$

$= 120.55 m^3$

Height of rectangular portion $(x) = \frac{120.55}{6.5 \times 5} = 3.71 m$

Total height of digestion chamber including neutral zone $= 0.45 + 3.71 + 2.1 = 6.26 m$

Total height of tank from top to bottom

= Ht. of sedimentation chamber + Ht. of digestion chamber

$= 4 + 6.26 = \underline{\underline{10.26 m}}$

This height is within the practical limits (9 to 11m)

∴ The design is ok.

Oxidation ponds.

→ Stabilization pond which is used to treat partially treated sewage is called oxidation ponds.

→ The purifying action in this pond is because of unique relation between bacteria and algae.

→ The bacteria oxidise organic matter producing CO_2, NH_3, PO_4

→ The algae use these compounds along with solar energy for synthesis releasing O_2 into the solution.

→ The O_2 released by algae is again used by bacteria and the cycle is repeated.

→ Oxidation pond being shallow depth (< 2m), both aerobic and anaerobic biochemical reactions take place simultaneously.

→ Detention period is usually 2 to 6 weeks

Design considerations

(1) Organic loading

- 300 to 150 kg/ha/day [hot tropical countries like India]
- 90 to 60 kg/ha/day [colder countries situated at higher latitude]

(2) Area of one unit – 0.5 to 1.0 ha.

(3) $\frac{L}{B}$ ratio = 2

(4) Depth of the tank = 1 to 1.5m

(5) Free board = 1m

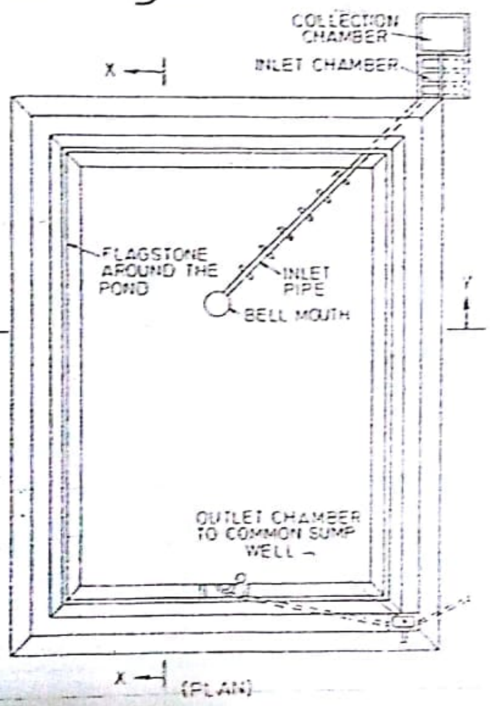
(6) Detention time – 70 to 42 days.

$$DT = \frac{1}{K_D} \log_{10}\left[\frac{L}{L-y}\right]$$

where L = BOD of effluent entering the pond.

y = BOD removed (90 to 95% of L)

K_D = Deoxygenation constant



Q. Design an oxidation pond for treating sewage from a hot climatic residential colony with 5000 persons, contributing sewage @ 120 litres/capita/day. The 5-day BOD of sewage is 300 mg/l

Soln:- Quantity of sewage to be treated per day

$= 5000 \times 120 = 600 m^3$

BOD content in sewage $= 300 \times 600 \times 10^{-3}$

$= 180 kg/day$.

Assuming organic loading in the pond as 300 kg/ha/day

Surface area required $= \frac{\text{total BOD}}{\text{Organic loading}}$

$= \frac{180}{300} = 0.6$ ha

$= 0.6 \times 10^4 m^2$

$= 6000 m^2$

Assuming $\frac{L}{B} = 2$

$2B \cdot B = 6000$

$B = 54.77 m \approx 55 m$

$L = 110 m$

Using a tank with effective depth as 1.2 m

Capacity $= L \times B \times H$

$= 55 \times 110 \times 1.2$

$= 7260 m^3$

Detention time = $\frac{\text{Capacity}}{\text{Sewage flow}}$

$= \frac{7260}{600} = 12.1 \approx 12$ days

Assuming a free board of 1m

Overall depth = 1.2+1 = 2.2 m

Hence provide an oxidation pond 110 X 55 X 2.2 m with DT = 12 days.

Design of inlet pipe.

Assuming an avg. velocity of sewage as 0.9 m/s and daily flow for 8 hours only

Discharge = 600 m^3/day

$= \frac{600}{8 \times 60 \times 60} = 0.021\ m^3/s$

Area of inlet pipe required = $\frac{\text{Discharge}}{\text{Velocity}}$

$= \frac{0.021}{0.9} = 232 cm^2$

$\frac{\pi}{4} D^2 = 232$

Diameter of inlet pipe, D = 17.2 cms ≈ 18 cm

Diameter of outlet pipe = 1.5 X Dia of inlet pipe

= 1.5 X 18 = 27 cm

Oxidation Ditch.

An oxidation ditch consists of an endless ditch for the aeration tank and a rotor for aeration mechanism. The ditch consists of a long continuous channel, usually oval in plan. The channel may be earthen with lined sloping sides and lined floor or it may be built in concrete or brick with vertical walls.

The sewage is aerated by a surface rotor placed across the channel. The rotor entrains the necessary oxygen into the liquid and keeps the contents of the ditch ~~mo~~ mixed and moving. They are designed to impart a velocity of 0.3 to 0.4 m/s to the mixed liquor. Cage rotors usually have a dia. of 70 cm and a speed of 75 rpm.

The standard oxygen transfer capacity of rotor is 2.8 kg O_2 per m length at 16 cm depth of immersion. Power requirements per metre length is about 1.35 kW at the rpm and immersion depth stipulated. The depth of ditch is kept as 1.0 to 1.2 m and the length of the ditch is designed to give the required aeration tank volume.

The raw sewage and return sludge are discharged into the ditch upstream of the rotors. The outlet of the ditch should be located geometrically opposite to the inlet.

Oxidation ditches are constructed in two types:

(i) Continuous flow type

ii) Intermittent flow type.

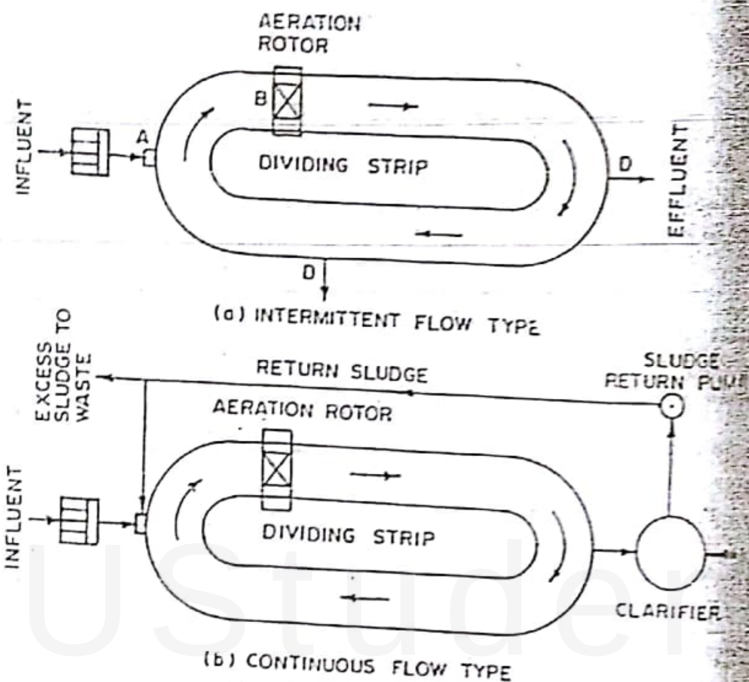


FIG. 15.1. OXIDATION DITCHES

Intermittent flow type.

In this type, no separate settling tank is used. The flow in the ditch remains suspended during a predetermined period, by stopping the rotor, and the ditch itself is used for settling. The supernatant is withdrawn through the outlet. The surplus sludge, settled in the ditch, is removed with the aid of a sludge trap. For intermittent

operation, the cycle consists of (i) closing the inlet valve and ~~eat~~ aerating the wastewater.
(ii) stopping the rotor and letting the contents settle,
(iii) Opening both inlet and outlet valves, thereby allowing the incoming wastewater to displace an equal volume of clarified effluent.

Continuous flow type:

In this type, the operation is kept continuous by allowing the mixed liquor to settle in a separate settling tank. Quiescent conditions in the clarified liquid passes over the effluent weir for final disposal. The settled sludge is removed from the bottom of the clarifier by an air lift or pump and is returned to the ditch.

Aerated Lagoons.

Aerated lagoon is a deeper oxidation pond, with oxygen introduced by mechanical aerators rather than relying on the photosynthetic oxygen production alone. As these ponds are deeper than the oxidation ponds and as they are artificially aerated, less detention time and areas are required.

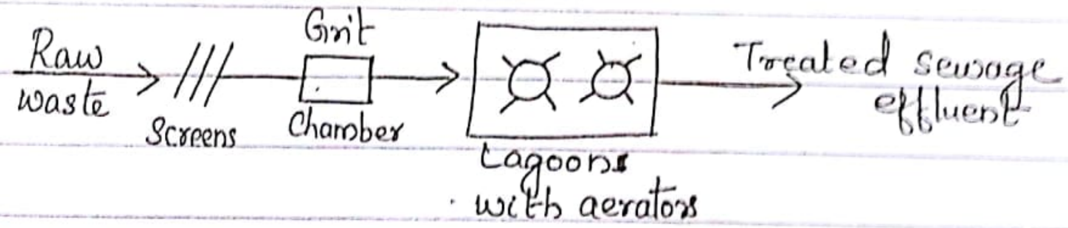
→ The depth of basin ranges between 2.4 to 3.6m.

→ Detention time – 4 to 10 hours.

→ The land area required is about 5 to 10% of that required for an equivalent oxidation pond.

→ Efficiency obtained ranges b/w 65 to 90%.

→ The aerated lagoons are frequently used for treating industrial waste waters.



→ Depending upon the extent of mixing, the lagoons may be classified as:

(i) Complete mix lagoons (aerobic aerated lagoons)

→ In this type, greater amount of aeration is provided to keep all the solids in suspension due to which the entire pond is aerobic.

→ It consists of two units.

→ In the first, the mechanical surface aerators are so designed that solids do not settle to the bottom of the tank, while the second unit is used as settling tank for the removal of suspended solids.

→ BOD removal – 75 to 85%

(ii) Partially mixed lagoons (facultative aerated lagoons)

→ The lagoons are operated at a low rate of aeration not adequate to keep all the solids in suspension but enough to keep top layers aerobic.

→ The sewage solids tend to settle down and anaerobic bottom is established.

→ A large portion of incoming solids and the biological solids produced within the lagoon settle to the bottom of the tank where anerobic decomposition takes place.

→ The effluent from this type of tanks is more stable.

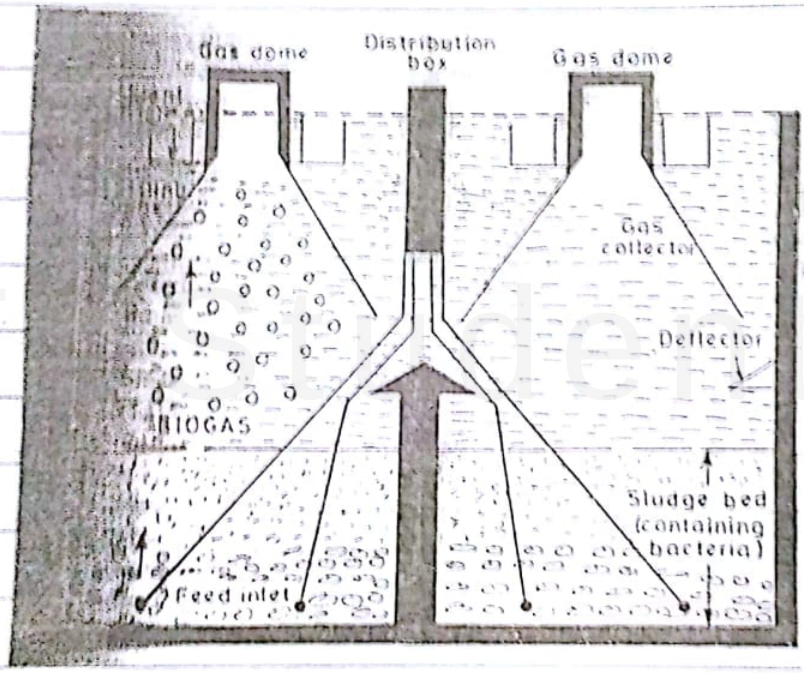
→ BOD removal – 75 to 90%.

Upflow Anaerobic Sludge blanket reactors (UASB reactor)

→ The wastewater flows upwards through a layer of very active sludge to cause anaerobic digestion of organics of the wastewater.

→ At the top of the reactor, three-phase separation between gas-liquid-solid takes place.

→ Any biomass leaving the reaction zone is directly recirculated from the settling zone.



→ This reactor consists of an upflowing treatment tank, provided with a feed inlet distribution system at the tank bottom.

→ A gas-solid-liquid separator device is provided at the top to help to provide a quiescent zone at the top of the reactor.

→ The wastewater enters the tank from the bottom and flows upwards through the sludge bed, which gets formed during the process itself.

→ The sludge bed develops micro-organisms capable of flourishing in an oxygen deficient environment.

→ The sludge bed traps the suspended organics of the ~~upcoming~~ upmoving wastewater.

→ The suspended solids trapped in the sludge bed are degraded by the anaerobic bacteria producing methane and carbondioxide.

→ The biogas produced during the anaerobic decomposition helps in providing gentle mixing and stirring of the biomass. This increase the efficiency of decomposition, reducing the BOD and suspended solids of the wastewater.

→ The methane or biogas is collected at the top of tank in a gas collector. It can be used as a gas for domestic or industrial use.

→ The treated effluent is collected in gutters and discharged out of the reactor. ~~The~~

→ The sludge is periodically shifted into the drying beds to be used as a soil enricher.

Design Considerations.

I) Upflow velocity (Superficial velocity)

→ It is based on the flow rate and reactor area

→ Temporary peak superficial velocities – 6m/h to 2m/h

→ The upflow velocity is equal to the feed rate divided by the reactor cross-section area:

$$v = \frac{Q}{A}$$

where v = design upflow superficial velocity, m/h
A = reactor cross section area, m^2
Q = Influent flow rate, m^3/h

II) Reactor volume and dimensions.

* The nominal liquid volume of the reactor,

$$V_n = \frac{Q S_o}{L_{org}}$$

where Q = Influent flow rate, m^3/h
S_o = Influent COD, kg COD/m^3
L_{org} = organic loading rate, kg COD/$m^3 \cdot d$

* Total liquid volume of the reactor exclusive of the gas storage area, $V_L = \frac{V_n}{E}$

Where, V_n = nominal liquid volume of reactor, m^3
E = Effectiveness factor [0.8 to 0.9]

* Area of the reactor, $A = \frac{Q}{V}$

* Height of the reactor, $H_L = \frac{V_L}{A}$

* Total height of the reactor

$$H_T = H_L + H_G$$

Where H_L = reactor height based on liquid volume.
H_G = reactor height to accomodate gas collection and storage, m
[2.5m to 3m]

III <u>Physical features</u>:
(i) Feed inlet
(ii) Gas separation
(iii) Gas Collection
(iv) Effluent withdrawal.

1. Design a UASB reactor treating an industrial wastewater with following data. Also calculate the detension time.

Flowrate $= 1000\ m^3/d$

$_sCOD = 2000\ g/m^3$

$L_{org} = 10\ kg_s COD/m^3.d$

Soln: Nominal liquid volume, $V_n = \frac{Q S_0}{L_{org}}$

$= \frac{(1000\ m^3/d)(2\ kg\ _sCOD/m^3)}{(10\ kg_s\ COD/m^3.d)}$

$= 200\ m^3$

Total reactor liquid volume, $V_L = \frac{V_n}{E}$

$= \frac{200}{0.85}$ [Assume E = 0.85]

$= 235\ m^3$

Area of the reactor, $A = \frac{Q}{v}$ [$v = 1.5\ m/h$]

$= \frac{1000\ m^3/d}{(1.5\ m/h)\ 24} = 27.8\ m^2$

$\frac{\pi}{4} D^2 = 27.8$

$D = 6m$

Height of the reactor, $H_L = \frac{V_L}{A}$

$= \frac{235}{27.8} = 8.4\,m$

Total height of the reactor

$H_T = H_L + H_G$ [Assume $H_G = 2.5m$]

$= 8.4 + 2.5$

$= 10.9\,m$

Reactor dimensions :- Diameter = 6 m

Height = 10.9 m

Detention time of reactor $= \frac{V_L}{Q} = \frac{235 \times 24}{1000}$

$= 5.64$ hrs