SECONDARY TREATMENT / BIOLOGICAL TREATMENT

The effluent from primary treatment units are further treated, using microorganisms for removal of dissolved and colloidal organic matter. The basic principle behind the working is, biological decomposition of organic matter under aerobic / anaerobic conditions.

Types

- Aerobic & Anaerobic process
  (Also aerobic - anaerobic process)
- Attached & suspended growth process

Aerobic processes

Aerobic processes are those which occur in the presence of DO. The aerobic processes include the following:

→ Trickling filter
→ Activated sludge process.
→ Aerobic stabilization ponds.
→ Aerobic lagoons.

Anaerobic processes

Anaerobic waste treatment involves the decomposition of organic & inorganic matter in absence of molecular O_2. Anaerobic process consists of the following:-

→ Anaerobic sludge digestion
→ Anaerobic contact process.
→ Anaerobic filters.
→ Anaerobic lagoons & ponds.

Aerobic - Anaerobic process

Aerobic - anaerobic processes are those in which stabilization of waste is brought about by a combination of aerobic anaerobic

& facultative bacteria. Most of the biological treatment processes are preferred to work on aerobic bacterial decomposition bcz such decomposition doesn't produce bad smells & gases as produced by anaerobic decomposition & also bcz aerobic bacteria are about 3 times more active than the anaerobic bacteria @ 30°c.

Biological Treatment Techniques

The biological treatment techniques used may be classified under the following 3 heads.

- → Attached Growth process.
- → Suspended growth process.
- → Combined process.

Attached growth process

These are the biological process in which the microorganisms responsible for the conversion of organic matter / other constituents in the waste water to gases & cell tissues are attached with some inert medium such as ~~broth~~ rock, slag

or specially designed ceramic / plastic materials.
Such processes include the following:-

→ Intermittent sand filter.
→ ~~Continuous sand filter~~ Trickling filters.
→ Rotating biological contactors.
→ Packed bed reactors.
→ Anaerobic lagoons. / Ponds.
→ Fixed film denitrification.

Suspended growth process

These are the biological treatment process in which the microorganisms responsible for the conversion of the organic matter or other constituents in the waste water to gases and cell tissue are maintained in suspension within the liquid in the reactor by employing either natural or mechanical mixing. The suspended growth processes include the following :-

→ Activated sludge process
→ Aerated lagoons.
→ Sludge digestion system.

Combined Processes

These consists of both attached growth processes as well as suspended growth process. They ~~including~~ includes following:

→ Trickling filter, Activated sludge
→ Activated sludge, Trickling filter

Secondary Treatment methods

Mod. IV

1. Filtration
   - Contact beds
   - Intermittent sand filters
   - Trickling filters (TF)
     - Conventional
     - High rate TF
       - Bio filter
       - Accelo filter
       - Aerofilter
2. Activated Sludge Process (ASP)
3. Rotating Biological Contactors (RBC)

Mod. V

4. Oxidation ditch
5. Oxidation pond / Stabilisation pond.
6. Aerated lagoons
7. Upflow Anaerobic Sludge Blanket Reactor (UASB)
8. Septic tank
9. Imhoff tank

The filters which are mostly used in sewage treatment can be classified as follows:

1. The Intermittent Sand Filters

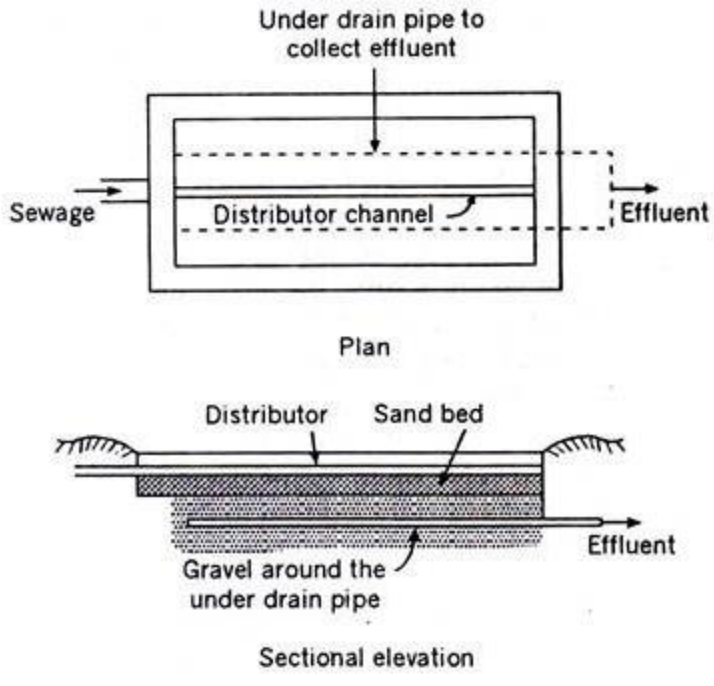
2. The Contact Beds

3. The Trickling Filters.

1. Intermittent Sand Filters:

These are the early development of sewage treatment units. These are similar in Construction to the slow-sand filter of water treatment. These require larger area, due to which these are not commonly employed in modern sewage treatment works.

Fig. shows an intermittent sand filter. It consists of layers of sands with an effective size of 0.2 to 0.5 mm and of uniformity coefficient 2-5. If the soil itself is sandy, there is no need of providing extra sand. But if the soil is of other variety, sands of the above specifications are laid in a depth of about 100-120 cm.



To carry off the effluent the open joint drainage pipes are laid in the bottom of the sand bed in 90 to 120 cm depth. Their drainage pipes are surrounded with layers of coarse stone and gravel graded from coarse to fine, to keep and the sand out. In some cases when the soil itself-sandy, the percolating effluent may reach the ground water table, and no effluent may reach the drainage pipes.

The sewage is applied evenly on the surface of the sand bed by influent waste water troughs as shown in Fig. The distribution trough has side openings to distribute the sewage uniformly. To prevent the scouring and displacement of sand the distribution trough is kept on concrete apron or protective stone. While applying the sewage the flooding is done from 3 to 10 cm depth after an interval of 24 hours. The capacity of these filters is 0.8 to 1.1 million litres/hectare per day.

The effluent from the intermittent sand filters is very clear and contains suspended solids less than 10 ppm which is well nitrified and stable. The effluent also has B.O.D. less than 5ppm and is free from odours. Therefore, the plant works without creating any nuisance at the site.

If the quantity of sewage is more 3 to 4 such beds can be constructed in parallel. For cleaning these filters, the sand from the top is scraped from time to time and are refilled with fresh clean sand.

The following are the advantages of intermittent sand filters:

(i) Operation is simple, only mechanical equipment is required for dosing.

(ii) The effluent is very clean and can be directly disposed of in natural watercourses without any further treatment.

(iii) There is no trouble of odour and insects.

(iv) Smaller head is required.

(v) There is no secondary sludge, which is to be disposed of except the occasional sand scraping.

The following are the disadvantages of intermittent sand filters:

(i) Their rate of loading is very small.

(ii) They require large area and much quantity of sand in their construction which makes them uneconomical.

(iii) They cannot treat large quantity of sewage, therefore cannot be employed at big plants.

The intermittent sand filters are most suitable for hospitals, institutions, small towns and factories, where it is not possible to dispose of the effluent of septic tanks on the ground surface.

2. Contact Beds:

In ancient time contact beds were very popular in the treatment of sewage, but now a days these are similar in construction to the intermittent sand filters, the only difference being in the filtering media. The filtering media consists of 2 to 2.5 cm size broken stone ballast or brick ballast.

The depth of the filtering media is between 90-150 cm. The sewage is uniformly applied over the whole surface of the filtering media, by means of distribution troughs and is collected at the bottom by means of a system of under drain pipes.

The operation of the contact beds includes the following:

(i) Filling:

In this operation the sewage is applied on the surface of contact beds as quickly as possible by means of dosing siphonThe filling may take one hour or so.

(ii) Contact:

In this operation, the dosing is stopped and the applied sewage is allowed to come in contact for about an hour with the bacterial film covering the filter medium. Within this time the soluble contents of sewage are absorbed by the organic film and are stabilized.

(iii) Emptying:

The contact beds are then slowly emptied and drained so that the absorbed soluble contents of the sewage are not washed out with the sewage, which is being drained.

(iv) Resting:

After emptying, the contact beds are allowed to remain at rest for 5-6 hours. Within this period the atmospheric air enters in the voids of the contact media and makes it ready for taking another sewage load. By supplying oxygen to the aerobic bacteria, which oxidize the organic matter present in the sewage which is transferred by sewage on the surface of the filtering media.

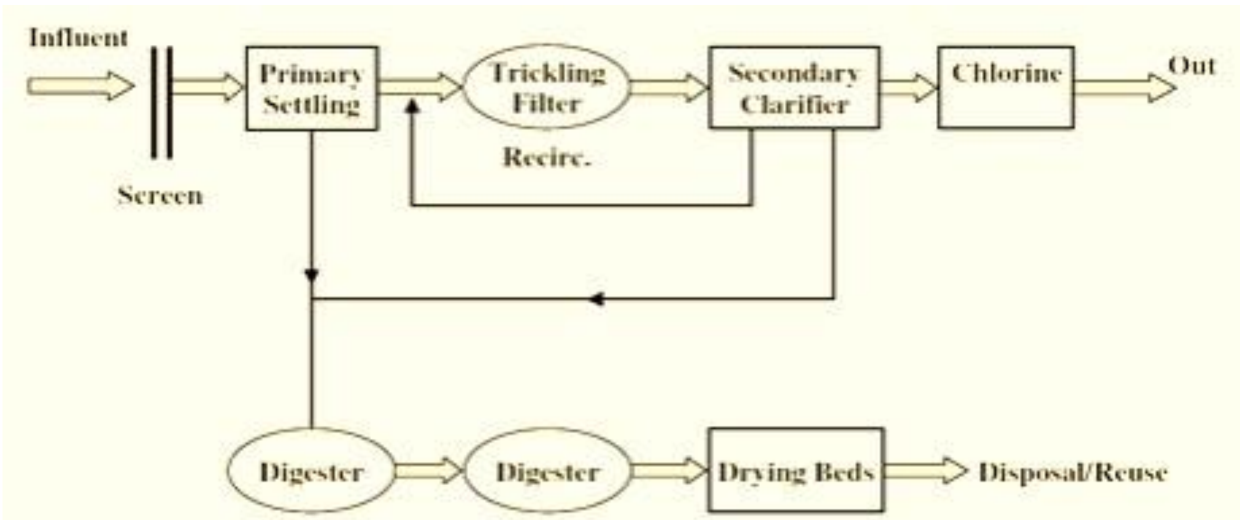
The complete cycle of operation takes 8-12 hours. As these contact beds are intermittent in action, therefore more numbers of units are constructed in parallel and the sewage is applied in turn to each unit. For this purpose continuous supervision is required.

The effluent obtained from these beds is also clear and odourless. These beds remove 80 to 90% suspended solids and 60 to 75% B.O.D. The rate of loading is very low 4500 to 6500 m3/hectare/day. The voids inside the filtering media continuously go on reducing due to accumulation of the solids in them.

After 4-5 years the filtering media is taken out, washed, dried and filled again. Similarly the under drain pipes are also washed and cleaned after 3-4 years. These are also not common these days.

Generally, the contact beds are also intermittent in their operation. The continuous operation of contact beds is possible by blowing air into the waste water flowing through them in sufficient quantity to keep the water and slime surface aerobic and in sufficient intensity to tear away ageing slime accumulation of solids on the surface.

3. TRICKING FILTER (TF)



TF Flow chart

Trickling filter is a type of waste water treatment process, which is an attached growth process. In this process the microorganisms responsible for digestion are attached to an inert filter material. This Packing material can be rock, gravel, sand and a wide range of plastic and other synthetic materials. In other words the removal of pollutants from waste-water involves both absorption & adsorption of organic materials by the layer of microbial bio-film. The packing media is typically chosen to provide a very high surface area to volume. It is also known as trickling bio-filter, trickle filter, bio-filter, biological filter.

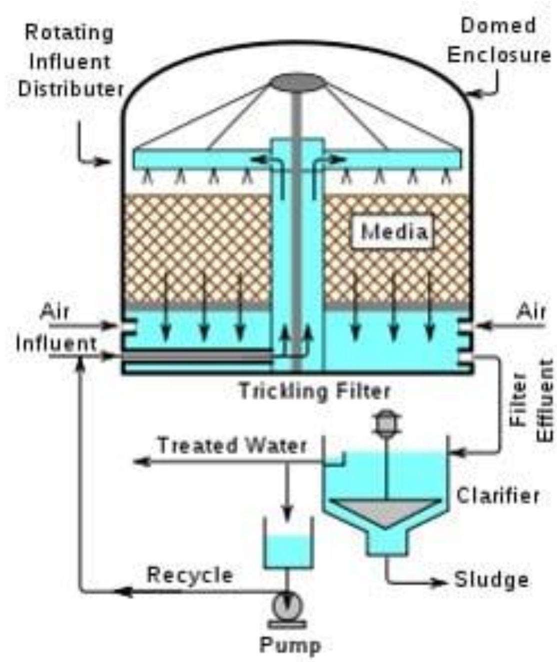
These systems have also called as roughing filters, intermittent filters, packed media bed filters, alternative septic systems, percolating filters, attached growth processes, and fixed film processes.

Functions

1. Remove Nutrient
2. Remove dissolved organic solids

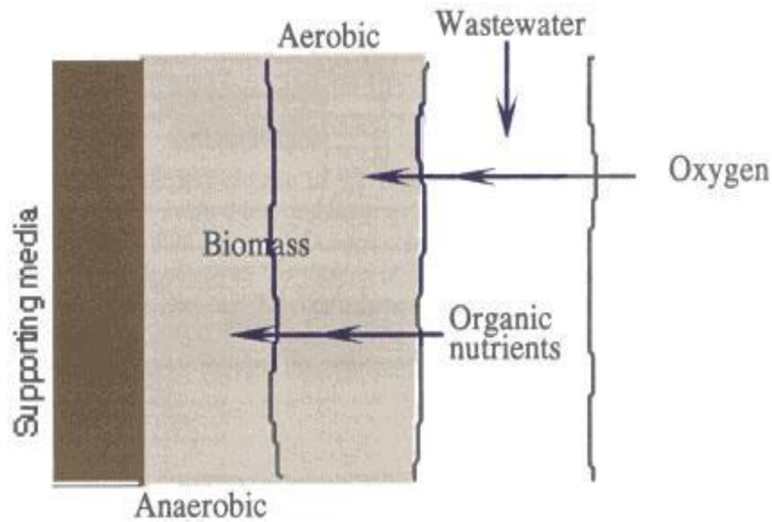
3. Remove suspended organic solids
4. Remove suspended solids

PROCESS



Trickling filter Diagram

Tank is filled with solid media like Rocks or Plastic, bacteria grows on surface of media. Wastewater is trickled over media, at top of tank. As water trickles through media, bacteria degrade BOD, The thickness of the aerobic layer is limited by the depth of penetration of oxygen into the microbial layer. Bacteria eventually die, fall off of media surface, known as **sloughing**. The sloughed off film and treated wastewater are collected by an underdrainage which also allows circulation of air through filter. Finally liquid is collected and passed to a settling tank used for separation of solid- liquid. Part of water is recycled back to the filter in order to maintain moist condition.



Biofilm formation

Design consideration

1. In-fluent waste water characteristics
2. Degree of treatment anticipated (BOD & TSS removal)
3. Temperature range of applied waste-water
4. Pretreatment processes
5. Type of filter media
6. Recirculation rate
7. Hydraulic & organic loadings on the filter
8. Under drainage and ventilation systems

TRICKLING FILTER- TROUBLES & REMEDIES

1. Filter Ponding

If the voids in the media get plugged, flow can collect on the surface in ponds. Excessive sloughing, excessive organic loading, non-uniformity in size of media and improper functioning of primary treatment units are its chief causes.

Remedies

- Wash the filter surface with a stream of water under high pressure.
- Dose the filter with heavy applications of chlorine.
- Take the filter out of service for a period of one day or longer to allow it to dry out.

2. Filter Flies

Slow rate TF often becomes infested by small moth like flies called Psychoda. Filter flies develop most frequently in an alternately wet and dry environment.

Remedies:

- Dose filter continuously, not intermittently.
- keep orifice openings clear
- apply insecticides to filter walls
- dose filter with chlorine
- keep weeds and tall grass cut around filter

3. Odour nuisance

The presence of "rotten egg" odour is an indication of anaerobic condition.

Remedies

- Maintain aerobic conditions in all units, including settling tanks and waste water system.
- Recirculate to filters.

4. Icing for Filter Surface

Cold weather not only reduces the efficiency of trickling filters by decreasing the activity of the microorganisms, but in severe cases actually can cause the wastewater to freeze on the medium surface.

Remedies

- decrease recirculation to the filter (influent is usually warmer than recycled flows)
- construct wind screens
- operate two-stage filters in parallel rather than in series

DESIGN OF TRICKLING FILTER

1. Design a circular TF unit for treating 4 MLD sewage, having a 5 day BOD of 160 mg/L. Also design underdrainage system as well as rotary system for the filter. Assume suitable data wherever required.

I. Design of filter

1 MLD = million litres per day = 10^6 l/day

Quantity of sewage generated per day = 4×10^6 l/day = 4×10^3 m^3/day

BOD of sewage = 160 mg/l

Total BOD = $4 \times 10^6 \times 160$ mg/day = 640 kg/day

Assume organic loading as 150 g/d/m^3 ⟷ *(80–320 g/d/m^3)

Volume of filter = $\frac{\text{Total BOD}}{\text{Organic loading}}$ = $\frac{640 \times 10^3 \text{ g/day}}{150 \text{ g/d/m}^3}$

= 4266.7 m^3

Check for hydraulic loading

Assume depth = 2.2 m

area = $\frac{\text{volume}}{\text{depth}} = \frac{4266.7}{2.2} = 1939.4 \text{ m}^2$

Dia. of filter ⇒ $\pi/4 \, d^2 = 1939.4$

$d = 49.7 \simeq 50$ m   *Dia (30–60m) Satisfactory

Actual surface area = $\pi/4 \times 50^2 = 1963.5 \text{ m}^2$

Surface area = $\frac{\text{Total sewage quantity}}{\text{Hydraulic loading rate}}$

∴ Hydraulic loading ($m^3/d/m^2$) = $\frac{4000 \text{ m}^3\text{/day}}{1963.5 \text{ m}^2}$ = 2.04 $m^3/d/m^2$ *(1–4 $m^3/d/m^2$)

Hence satisfactory.

Hence provide a filter of 50 m dia & 2.2 m depth.

II. Design of Rotary distributors

The pipe of rotary distributor is designed for peak velocity less than 2 m/s & avg. velocity ≮ 1 m/s.

Let's take peak flow factor as 2.25.

$\therefore$ Peak flow $= 2.25\,Q = 2.25 \times 4 \times 10^3$ m³/day

$Q_p = 0.1042$ m³/sec

a) Design of central column

Assume velocity as 2 m/s. (v_1)

Q_p = area x velocity

$0.1042 = \pi/4\, d_1^2 \times v_1$

$d_1 = 0.2575\text{ m} \simeq 25\text{ cm}$

Hence provide 25 cm ϕ central column.

$Q_p = 2.25\,Q_{avg}$

*Check for avg. velocity

avg. flow = avg. velocity x area.

$\frac{0.1042}{2.25} = v_{avg} \times \pi/4 \times d_1^2$

peak factor → 2.25

$v_{avg} = 0.97$ m/s (should be greater than 1 m/s)

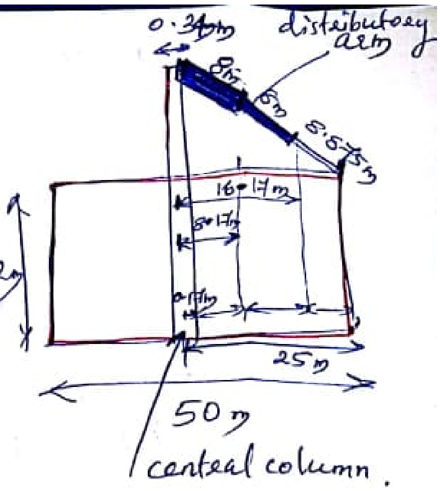
* $\therefore$ Pipe dia needs to be reduced for increasing velocity. (Remark)

b) Design of arms

Assume rotary distributor with 4 arms is provided.

Peak discharge per arm $Q_{p\,arm} = \frac{0.1042}{4} = 0.02605$ m³/sec

Length of arm $= \frac{50 - 0.25}{2} = 24.875$ m



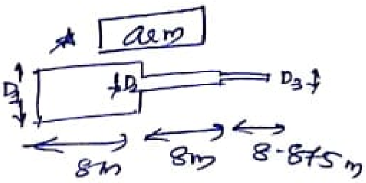
Let's provide 24.87m long arms with its size reducing from centre to the end.

∴ Provide 3 sections of arm,

1st two sections with 8 m &

3rd section with 8.875 m.

Let A_1, A_2, A_3 be the circular filter area covered by each length of arm.



Let's also provide 0.34 m φ in the centre.

$$A_1 = \pi(8.17^2 - 0.17^2) = 209.61 \ m^2$$

$$A_2 = \pi(16.17^2 - 8.17^2) = 611.73 \ m^2$$

$$A_3 = \pi(25^2 - 16.17^2) = 1142.07 \ m^2$$

$$1963.41 \ m^2.$$

Proportionate areas served by each section of arm;

$$P_{a1} = A_1/A \times 100 = \frac{209.61}{1963.41} \times 100 = 10.67\%.$$

$$P_{a2} = A_2/A \times 100 = \frac{611.73}{1963.41} \times 100 = 31.16\%.$$

$$P_{a3} = A_3/A \times 100 = \frac{1142.07}{1963.41} \times 100 = 58.17\%.$$

$$100\%.$$

i) Design of First Section of Arm

Assume velocity $(V_2) = 1.2$ m/s.

$Q_{p\,arm} = 0.02605$ m³/sec

$$\text{Area required} = Q/V_2 = \frac{0.02605}{1.2} = 0.02171 \ m^2$$

$$\pi/4 \ D^2 = 0.02171 \ m^2$$

$$D_1 = 0.166 \ m \simeq 170 \ mm$$

Hence provide 170 mm φ

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ii) Design of second section of arm

$$Q = 0.02605 \times \left(\frac{100 - 10.67}{100}\right)$$ → deduct the discharge served by 1st section

$$= 0.02327 \ m^3/sec$$

$$\pi/4 \, D_2^2 = Q/v_2 \Rightarrow D_2 = 160 \ mm \ \phi$$

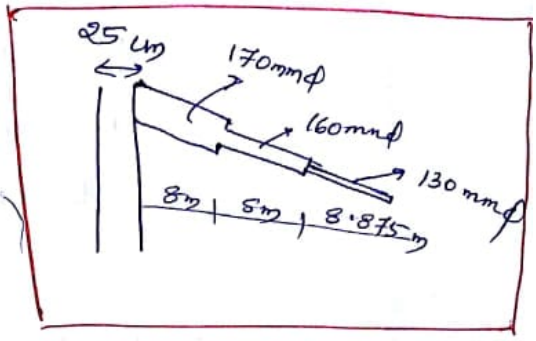
Similarly

iii) Design of third section of arm

$$Q = 0.02605 \times \left(\frac{100 - (10.67 + 31.16)}{100}\right)$$

$$= 0.01515 \ m^3/sec$$

$$\pi/4 \, D_3^2 = Q/v_2 \Rightarrow D_3 = 130 \ mm \ \phi$$



c) Design of Orifice

★ Let's provide 12 mm φ orifices with $C_d = 0.6$ & head causing flow = 1.5 m

$$Q_{orifice} = C_d \, a \sqrt{2gh}$$

$$= 0.6 \ \pi/4 \ (0.012)^2 \sqrt{2 \times 9.81 \times 1.5}$$

$$= 3.6813 \times 10^{-4} \ m^3/sec$$

No. of orifices required in each arm $= \frac{Q \text{ per arm}}{Q_{orifice}} = \frac{0.02605}{3.6813 \times 10^{-4}}$

$= 71$ nos

No. of orifices & spacing of orifices in each section

1st section, $n_1 = \frac{10.67}{100} \times 71 = 7.58 \simeq 8$ $\quad S_1 = \frac{\text{length}}{\text{no. of orifice}} = \frac{800}{8} = 100$ cm c/c

2nd section, $n_2 = \frac{31.16}{100} \times 71 \simeq 22$ $\quad S_2 = \frac{800}{22} = 36.36$ cm c/c

3rd section, $n_3 = \frac{58.17}{100} \times 71 \simeq 41$ $\quad S_3 = \frac{887.5}{41} = 21.65$ cm c/c

71 ✓

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III. Design of underdrainage system

Let's provide central channel of rectangular section, fed by radial laterals of semi-circular section discharging into the central channel.

a) Design of rectangular effluent channel

> ⇒ Velocity ≮ 0.75 m/s at peak instantaneous hydraulic loading
> ⇒ velocity ≮ 0.6 m/s at avg. instantaneous hydraulic loading

Let's provide a flow velocity of 1 m/s at peak flow.

$$Area = \frac{Q_{peak}}{v}$$

$$= \frac{0.1042}{1} = 0.1042 \; m^2$$

Assume width = 0.25 m.

$$Depth = \frac{Area}{width} = 0.4168 \; m \simeq 0.4 \; m$$

$B = 0.25 \; m$
$D = 0.4 \; m$
$A = BD$
$P = B + 2D$
$R = A/P$
$= 0.0952$

$$Q_p = A V$$

$$Q_p = A \frac{1}{N} R^{2/3} S^{1/2}$$

Let $N = 0.018$

On solving $S = \frac{1}{123.6} \simeq 1 \text{ in } 120$

∴ Provide a central effluent channel of width 25 cm, & depth 40 cm below the level of laterals, and lay the channel at a slope of 1 in 120.

b) Design of Radial Laterals

Assume radial underdrains are laid at a slope of 1 in 40. (designed to run half full)

$a/A = 0.25$ (semicircular)

$d/D = 0.298$

$q = 0.194\ Q$

Assume 10 cm dia semicircular underdrain blocks . .

~~Provide~~

$N = 0.013$

$R = A/P = D/4$

* $Q = \frac{1}{N} R^{2/3} S^{1/2} A$

$= \frac{1}{0.013} \left(\frac{0.1}{4}\right)^{2/3} (1/40)^{1/2} \frac{\pi}{4} \times 0.1^2$

$= 0.0082\ m^3/sec$

$q = 0.194\ Q$

$= 0.0016\ m^3/s$

$\text{No. of laterals} = \frac{Q_p \text{ filter}}{Q \text{ laterals}} = \frac{0.1042}{0.0016}$

$= 66$ nos.

$\text{Spacing} = \frac{\text{Dia. of filter}}{\text{No. of laterals}} = \frac{50}{66}$

$= 0.75\,m$

~~Provide 10 cm ϕ A.~~

Provide 66 nos. 10 cm ϕ laterals at 75 cm c/c .

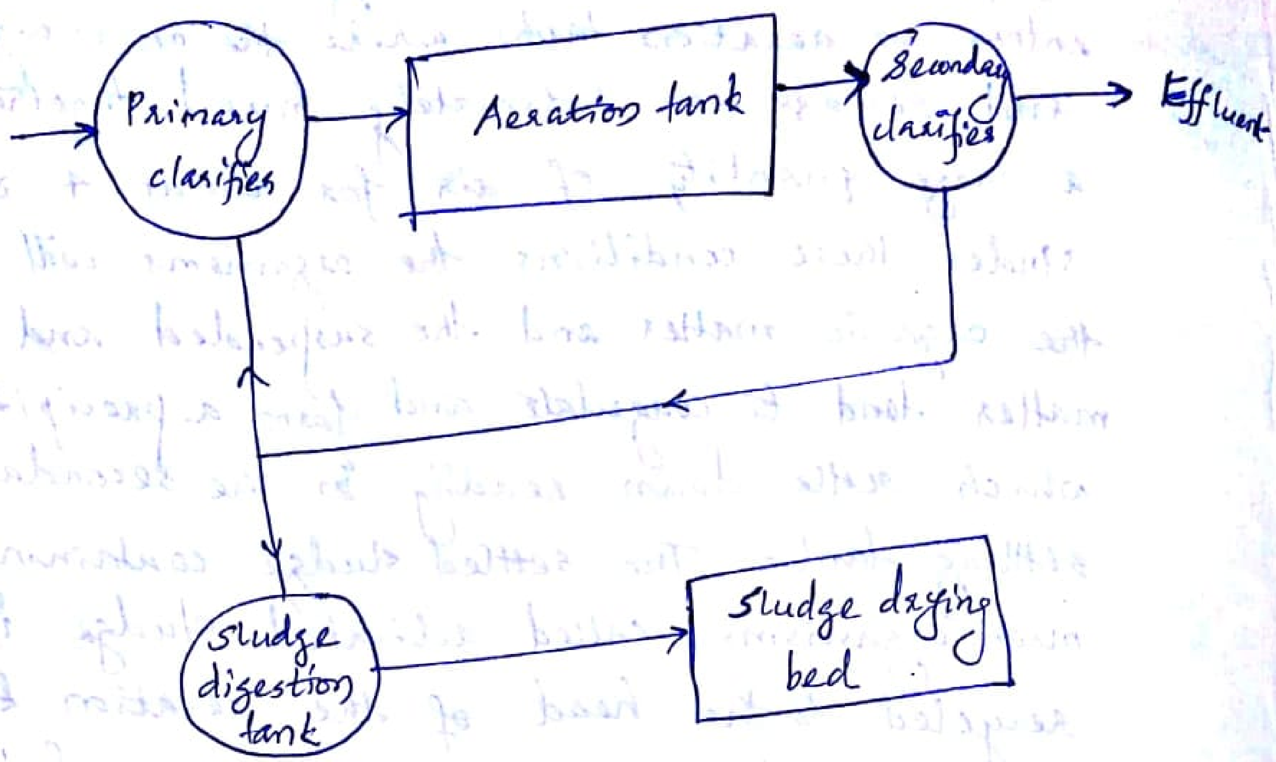
v. imp.

Activated Sludge Process

The activated sludge process provides an excellent method of treating raw sewage. The sewage effluent from primary sedimentation tank, which is normally utilized in this process is mixed with 20-30% of whole volume of activated sludge which contains a large concentration of highly active aerobic microorganisms. The mixture enters an aeration tank where the microorganisms and sewage are intimately mixed together with a large quantity of air for about 4-8 hrs. Under these conditions, the organisms will oxidise the organic matter and the suspended and colloidal matter tend to coagulate and form a precipitate which settle down readily in the secondary settling tank. The settled sludge containing microorganisms called activated sludge is then recycled to the head of the aeration tank to be mixed again with the sewage being treated. New activated sludge is continuously being produced by this process and a portion of it being utilised and sent back to the aeration tank. Whereas the excess portion is disposed off properly along with the sludge collected during the 1° sedimentation after digestion.

Fig. shows the flow diagram of Activated sludge process. Following are the 3 basic operations involved in the activated sludge process.

Flow diagram



I. Mixing of activated sludge.

The activated sludge is mixed properly with raw or settled sewage. The activated sludge is added to the effluent of primary clarifier.

II. Aeration

The mixed liquor containing activated sludge and effluent is agitated or aerated in the aeration tank. This is the chief operation of activated sludge process.

II. Settling in 2° clarifier

The mixed liquor after agitation is taken to the 2° clarifier. Sludge is allowed to settle in this tank. Settled sludge is the activated sludge and a portion of it is sent for recirculation. The extra activated sludge is taken to sludge digestion tank & sludge drying beds for further treatment.

Methods of aeration.

Following are the 3 methods which are employed for the purpose of aeration in activated sludge processes.

1. Diffused air aeration.
2. Mechanical aeration.
3. Combination of diffused air aeration & mechanical aeration.

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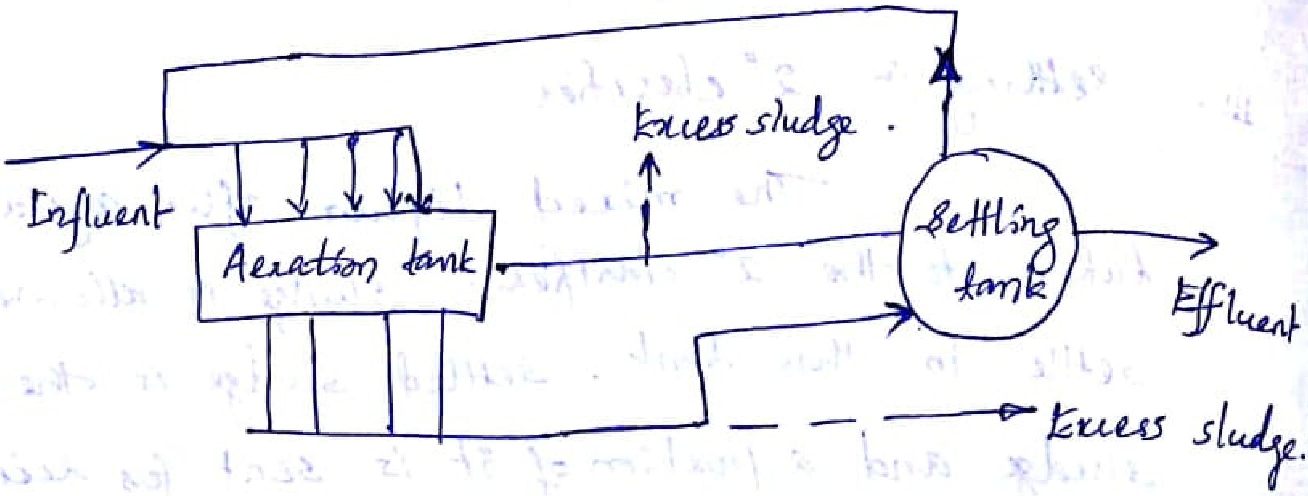
Extended aeration process



1° sedimentation is avoided in this process but grit chamber / communitor is often provided for screenings. As its name suggests the aeration period is quite large and extended to ≃ 20-30 hrs. The BOD removal η is also quite high, to say about 90-98% as compared to 85-95 % of a conventional plant.

The air requirement is ofcourse quite high which ↑ the running cost of the plant considerably. No separate sludge digestor is req. here; because the solids undergo considerable decomposition over the long detention period adopted in the aeration tank. The sludge production can be directly taken to sludge drying bed. Sludge production is also min. in this method. Oxidation ditch is working on a this principle.

Activated sludge process vs Trickling filter

In activated sludge process the bacterial film is contained in the fine suspended matter of sewage and this film is kept moving by constant (continuous) agitation - suspended growth.

In trickling filters, the bacterial film is formed around the particles of contact material and it is stationary.

The activated sludge process & trickling filter help in achieving more or less the same standards of purification.

13/1/19

MODULE - 4

BIOLOGICAL TREATMENTS [PPT]

objective

- To oxidize dissolved + particulate biodegradable pollutants to non polluting end products
- To remove nutrients like N_2+P
- To capture non settlable & suspended solids into a biofilm.
- To remove specifi trace organic compounds.

Types of biological treatment

1) Aerobic and anerobic
2) Attached & suspended

Aerobic	Anerobic
50% Carbon is converted to CO_2 40-50% of carbon is converted to bio mass High energy input Nutrient addition requirement is substantial Requires large area	94% of Carbon is converted into biogas. 5% Carbon converted into bio-mass. No external energy input Low nutrient requiremet Low area.

Attached Growth Process

Microorganisms that are used for the conversion of nutrients or organic material are attached to inert packing material

Suspended Growth Process

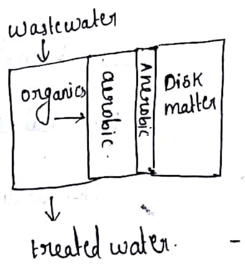
Secondary treatment

The effluent from primary treatment is treated further for removal of dissolved & colloidal organic matter.

ROTATING BIOLOGICAL CONTACTOR (RBC)

- Aerobic
- Attached growth treatment process.
- Consists of series of closely spaced circular plastic disks that are attached to rotating hydraulic shaft
- Disc 3m dia
  10mm thick
  30-40 mm spacing
- 40% of bottom of each plate is dipped in wastewater & the film which grows on the disk moves in & out of wastewater.
- Rotated 1-2 RPM.
- Biofilm formed attached to the surface of disk.

Film absorbs organic pollutant when submerged period of rotation.



- oxygen transfer occur during exposed to atmosphere.
- sloughing occurs when the thickness of biofilm increases & attachment decreases
- It is removed in the clarifier stage.

CONTACT BED FILTER

- A contact bed consists of a watertight tank filled with filtering media.
- The tank is usually constructed below the ground surface by excavating the earth and it is provided with a lining of cement concrete or watertight cement plaster on masonry, on sides as well as on bottom.
- Filtering media. Gravell, ballast or broken stones.
  20-40mm.

⇒ operation

1) Filling : depth of sewage 50-100mm above the top of the bed
   Filling take 1-2hrs.
2) Contact : contact time - 2 hours.
3) Emptying : The outlet ~~drain~~ valve of the underdrain is opened.
4) Oxidation : allow to stand for about 4-6 hrs, atmospheric air enters.

⇒ Removes 80% suspended solids.
⇒ Removes 60-75% BOD
⇒ Performance reduces with time
⇒ After 4-5 years filter have to be change completely
⇒ Rate of loading should not exceed 110 L/day/m².

INTERMITTENT SAND FILTER

Filter media is sand with effective size of 0.2-0.5 mm & uniformity coefficient of 2-5

once in 24 hrs 5-10 ~~mm~~ cm depth of sewage.

Secondary Sedimentation Tank.

→ Provided after the biological reactor to facilitate the sedimentation of the cells produced during biological oxidation of organic matter.

→ Represent about 40-60% of organic matter present in untreated wastewater.

Trickling Filter (Refer ppt)

8/4/19

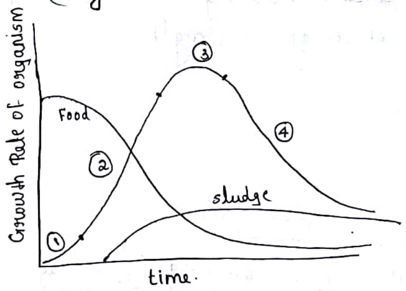
ACTIVATED SLUDGE PROCESS (ASP)

- organic matter - substrate (Food of microorganisms)
- organism mass is called Mixed Liquor Volatile Suspended Solids (MLSS)
- organisms need a balance of Food (BOD) and oxygen.
- The balance of Food to microorganism mass is known as F/M ratio.
- Suspended aerobic treatment process
- 4-8 hrs
- Microorganism will oxidise organic matter & suspended colloidal matter.
- Then settling in SST
- Organic matter + O_2 + Nutrients $\xrightarrow{\text{Micro-organisms}}$ $CO_2 + H_2O$ + Energy + microorganisms

Aeration Tank

- Rectangular, 3-4.5 m deep & 2-4 m wide
- length 20 - 200m
- Methods of aeration
  1) Diffused
  2) Mechanical
  3) Combined

(Diagram should draw)



① lag growth
② log growth / Exponential growth phase
③ Declining growth / stationary phase
④ Endogenous growth

Y → BOD Y_0 - initial Y_E - Effluent
X → conc. of MLSS
$X = X_T$ - inside the tank
X_R - Return sludge
Q → Quantity
Q_R → Return sludge
Q_w → wasted sludge

(Process Flow diagram)

Design Considerations.

Aeration tank loading

→ Aeration Period (HRT)

$$t = \frac{V}{Q} = \frac{\text{Vol. of tank}}{}$$

→ organic loading rate = $\frac{Q Y_0}{V}$

→ $F/M = \frac{Q \times Y_0}{V \cdot X_T}$

→ Sludge Age, $\theta_c = \frac{V \cdot X_T}{Q_w X_R}$

9/4/19 NB

Design suitable dimensions of circular trickling Filter For treating 5MLD of sewage per day. BOD of sewage 120 mg/l.

Ans) Quantity of sewage = 5MLD

BOD of sewage = 150 mg/L

Total BOD of sewage = $\frac{5\times10^6 \times 150}{10^{-6}}$ = 750 Kg/day

Assume organic loading rate (900-2200 Kg/he-m/day)

[For high rate trickling Filter (6000-18000 Kg/he-m/day)]

Here assuming organic loading rate = 1500 Kg/hec-m/day

Volume of Filter = $\frac{750 \text{ Kg/day}}{1500 \text{ Kg/hec-m/day}}$

= 0.5 hec-m

= 5000 m^3

check For hyrolic loading rate

Assuming depth = 2m

area of Filter = $\frac{5000}{2}$ = 2500 m^2

[Hydraulic loading rate : 22-44 ML/hec/day
110-330 ML/hec/day (high rate)]

surface area = $\frac{\text{Total sewage rate}}{\text{Hydraulic loading rate}}$

$\frac{2500\times10}{10^4}$ = $\frac{5}{\text{hydraulic loading rate}}$

Hydraulic loading rate = 20 ML/hec/day

HRT should ~~slightly~~ be increased to get a value ~~to~~ b/w 22-44 ML/hec/day.

Dia. of the Filter

$\frac{\pi}{4} d^2 = 2500 m^2$

d = 56.41 m

Since dia is greater than 40m so provide as 2 units.

Area of one unit = 1250 m^2

$\frac{\pi}{4} d^2 = 1250$

d = 39.89 ≈ 40m

(2+0.5 ↑ Freeboard)

∴ Provide 2 units of 40m diameter and a depth of 2.5m

Design of rotary arm

Flow = 5MLD

Peak Flow = 2.25×5 = 11.25 MLD

= $\frac{11.25\times10^6}{24\times60\times60\times10^3}$ = 0.13 m^3/sec

Flow in one unit = $\frac{0.13}{2}$ = 0.065 m^3/sec

(i) Design of Central column

Assume velocity = 2m/s   Flow = 0.065 m^3/sec.

Diameter, $\frac{\pi}{4} d^2 = \frac{0.065}{2}$

d = 0.2m

Check for avg Flow

Avg Flow $= \frac{0.065}{2.25} = 0.029\ m^3/sec.$

Avg. velocity $= \frac{0.0289}{\frac{\pi}{4} \times 0.2^2}$ $\left(V = \frac{Q}{A}\right)$

$= 0.92\ m/sec$ it should be greater than 1

Avg. velocity should be greater than 1m/sec. so we have to reduce the dia slightly. But as pipes are not available in the size we will provide dia of 0.2m.

(ii) Design of Arms

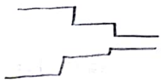
Assume rotary distributor with 4 arms

Discharge per arm $= \frac{0.065}{4} = 0.016\ m^3/sec.$

Assume velocity through arms $= 1.2\ m/sec$

Dia of arms $\frac{\pi}{4} d^2 = \frac{0.016}{1.2}$

$d = 0.13\ m = 130\ mm$



(iii) Design of orifice

Assume dia. of orifice $= 10\ mm$

Heading causing flow $= 1.5\ m$

Coe. of discharge $C_d = 0.65$

Discharge through orifice $= C_d A \sqrt{2gh}$

$= 0.65 \times \frac{\pi}{4} \times 0.01^2 \times \sqrt{2 \times 9.81 \times 1.5}$

$= 2.769 \times 10^{-4}\ m^3/sec$

No. of area $= \frac{0.016}{2.769 \times 10^{-4}}$

$= 57.7 \approx 58$

Spacing :

Length of arm $= \frac{40 - 2}{2} = 19\ m$ (2 → For central column)

Spacing $= \frac{19\ m}{58} = 0.3\ m$

Provide 58 orifice center to centre distance of 0.3m.

Design of Underdrains.

Design underdrinage system with central rectangular channel fed by radial laterals discharging into the channel

Total discharge $= 0.065\ m^3/s$

Assume velocity $= 1\ m/s$

Area $= \frac{0.065}{1} = 0.065\ m^2$

Assume width $= 0.25\ m$

$\therefore$ depth $= \frac{0.065}{0.25} = 0.26\ m$

$P = 2\ \text{depth} + 1\ \text{width}$

$R = \frac{A}{P} = \frac{0.065}{(2 \times 0.26) + 0.25}$

$= 0.084\ m$

$Q = \frac{1}{N} A R^{2/3} S^{1/2}$

$N = 0.018$

$$0.065 = \frac{1}{0.018} \times 0.065 \times (0.084)^{2/3} \times S^{1/2}$$

$$S = \frac{1}{115}$$

Provide central effluent channel of width 0.25 m & depth 0.26 m below the level of laterals at a slope of 1 in 115

Design of laterals

Provide a slope of 1 in 40 and assume 10 cm dia., semi-circular underdrain blocks.

Assume $\frac{d}{D}$ ratio = 0.2

$q = 0.196\,Q$	← Found from equations after finding α.
$a = 0.253\,A$	

$$R = \frac{A}{P} = \frac{\pi/4\, d^2}{\pi d} = \frac{D}{4}$$

$$Q = \frac{1}{N} A R^{2/3} S^{1/2} \qquad N = 0.013$$

$$= \frac{1}{0.013} \times \frac{\pi}{4} \times 0.1^2 \times \left(\frac{0.1}{4}\right)^{2/3} \times \left(\frac{1}{40}\right)^{1/2}$$

$$= 0.00816\ m^3/sec$$

$$q = 0.196 \times 0.00816$$

$$= 0.0016\ m^3/sec$$

Discharge through Filter = $0.065\ m^3/s$.

Discharge through laterals = $0.0016\ m^3/sec$

$$\text{No. of laterals} = \frac{0.065}{0.0016} = 40.6 \approx 40$$

$$\text{Spacing} = \frac{40\,m}{40} = 1\,m$$

Provide 40 lateral at a spacing of 1m centre to centre

10/4/19

OXIDATI
stabiliz
longdet
Aerobic
Anaero
Facultat

Aerobi
- Microbi
- Bost A
- shallo
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Facul
- Most
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