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est. 1914

1852

5) solutes - stabilizers

1891

such that observations y_i are correlated

Delivered as received by Special Collector

in the universe of discourse.

Novel

4) Solid benzene: Tr

10. *Chelidonium*

these solids are removed (1) according to the

from water depending on the floatation

Upd. of 2012

10. 11. 1901

5) Nutritional or Molecular

TABLE 1

5) Intellectual control

7) Miscellaneous operations - disinfectant

10 copper wire

⑤ oblique

iv) Thermal de-

Tuesday
12/12/2019

3) Solid concentration and stabilization

- i) thickening
- ii) centrifuging
- iii) chemical conditioning
- iv) electrolysis
- v) flocculation
- vi) vacuum filtration
- vii) air drying
- viii) Heat drying
- ix) sludge digestion
- x) incineration
- xi) wet combustion

also see in last page

also see in last page

Site selection for water treatment plant:- (7 modes)

The water treat. plant should be located as near as town as possible. If the source of water supply is tube well, then the treatment plant should be located central part of town so that purified water may be reach to public as early as possible and also the water head

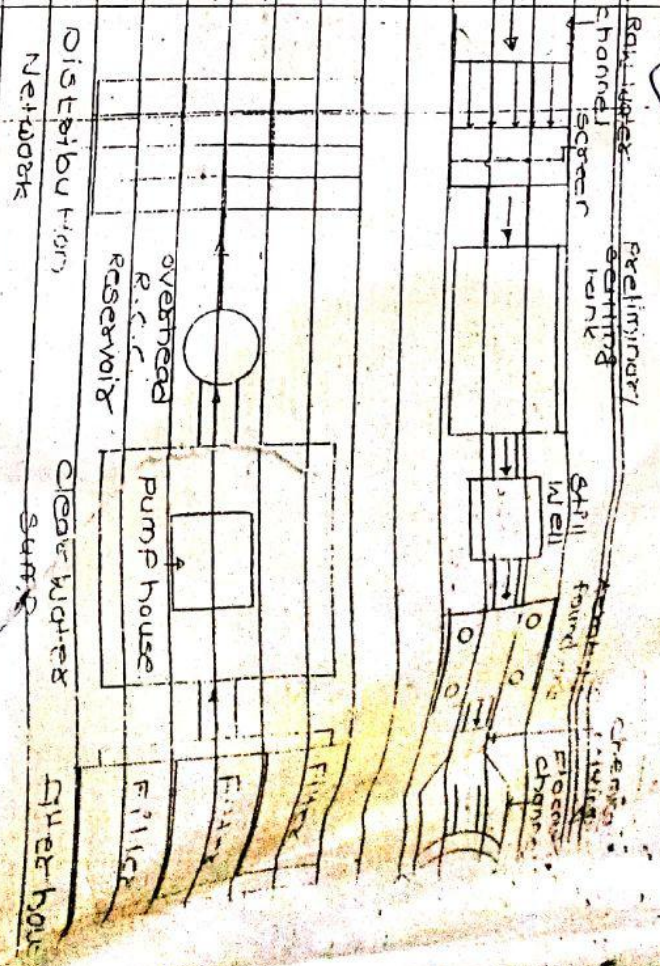
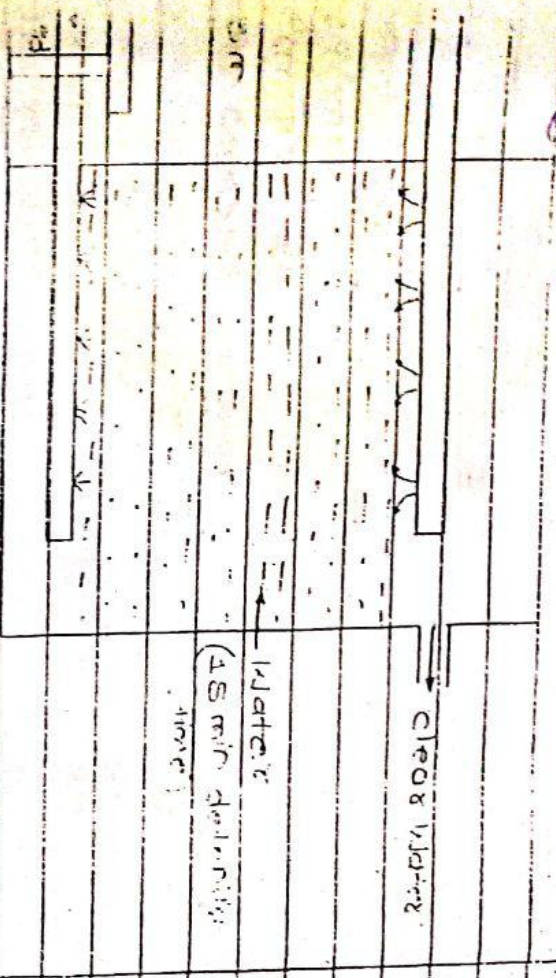


Fig: Schematic layout of water treatment plant

- 1) at extreme source point will be
- 2) If the city is very large then tube well with necessary treatment plant should be provided for
- 3) The source of water is river reservoir the treatment plant should be located as near the town in central place so that the water to consumer with more purity

- ② Spraying operation
- ③ Trickling operation
- ④ Trickling bed operation
- ⑤ Trickling bed operation
- ⑥ Trickling bed operation

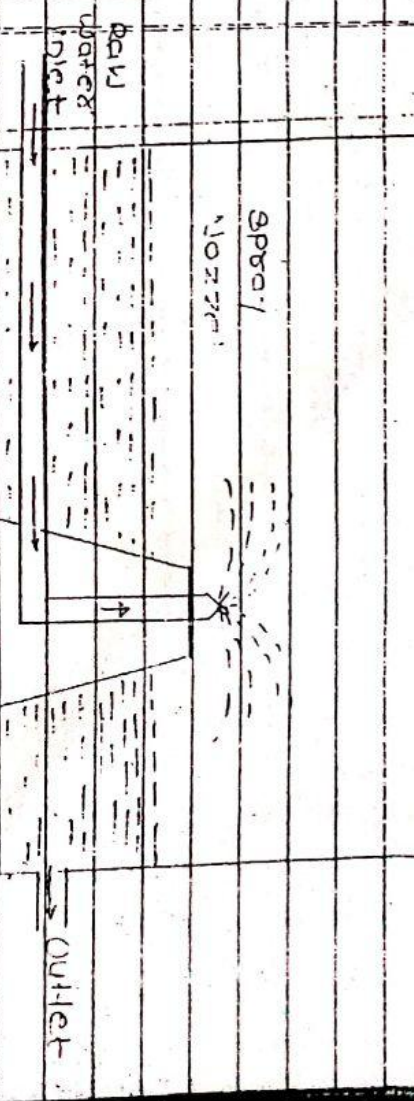


AIR DIFFUSER

By using spray nozzle :-

In this method water is thrown out in the air into fine spray to height of 2 to 2.5 m. under water pressure of 2 to 2.5 kg/cm² when a small particle of water comes in contact with scattered surface area of air they absorbed it and water is created the dissolved gases like CO₂, Hydrogen sulphide escape into atmosphere and oxidation of various substances to organic matter takes place

Trickling Jets



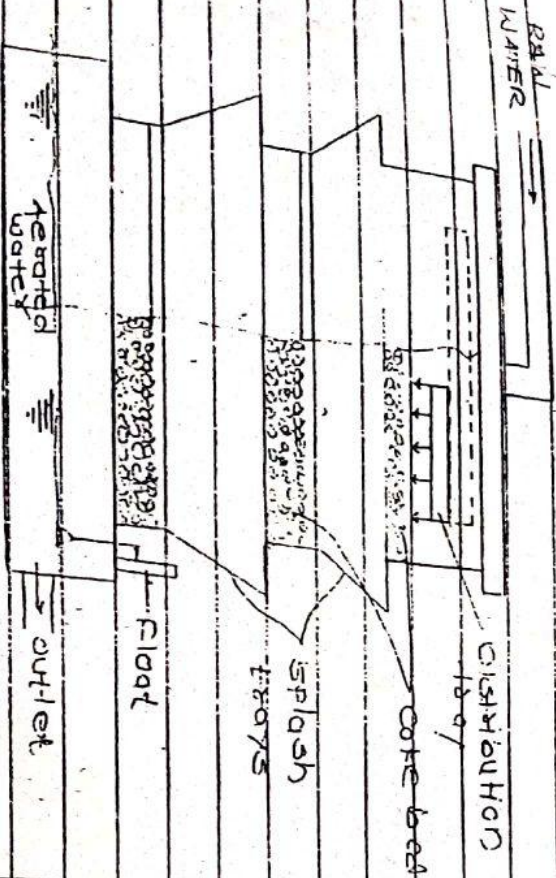
3) Trickling jets :-

In this type of method water is allowed to flow on the trickling bed on which are supported on perforated bottom of trays. Trickling bed filter having three bed trays the water is allowed to trickle from top to bottom under gravitation force under this downward movement the water get mixed with the air and oxidation takes place. The efficiency of this method is more than cascade but is less effective than method of spray nozzle.

Design criteria :-

- 1) No. of trays normally 4 to 5 in numbers
- 2) Depth of media in each tray normally 20 to 40 cm

- spacing between trays 30 to 70 cm,
 5) The space requirement is 0.015 to 0.042 $m^2/m^3/hr$.



* Air diffusion design criteria :-

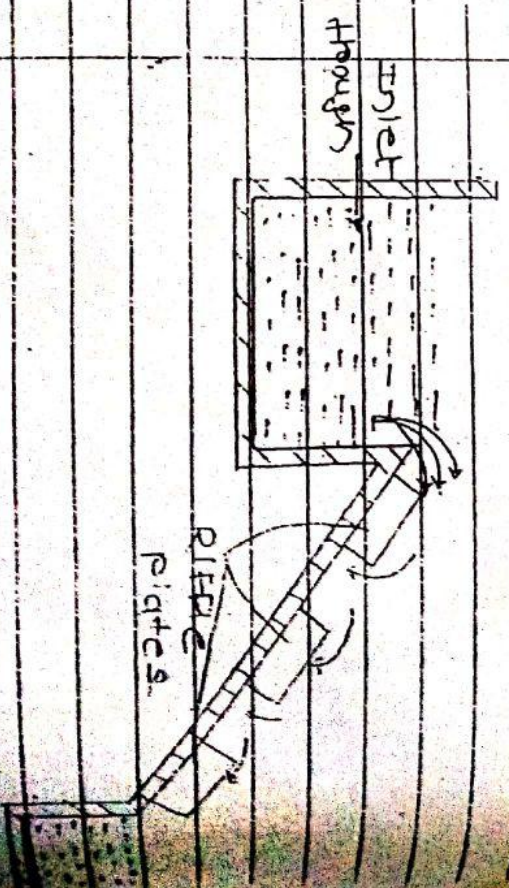
- 1) Depth of tank 3 to 4.5 m.
- 2) Width of tank 3 to 3 m.
- 3) Width to depth ratio 2
- 4) Detention period 10 to 30 min.
- 5) Amount of air 0.06 to 1 m^3/m^3 of water
- 6) Location of diffuser along one longitudinal side of tank.
- 7) Power requirements of air blowers 6 to 10 hp to 15 hp

* Spray type design criteria :-

- 1) Nozzle diameter 10 to 40 mm.
- 2) Spacing of nozzles 0.5 to 1 m
- 3) Pressure required at nozzle 10 to 20 kg/cm^2
- 4) Discharge per nozzle 18 to 26 m^3/hr
- 5) Area requirement 0.03 to 0.09 m^2/m^3
- 6) Efficiency of absorber 10 to 90%

* Advantages

- 4) Inclined absorber with plates :-



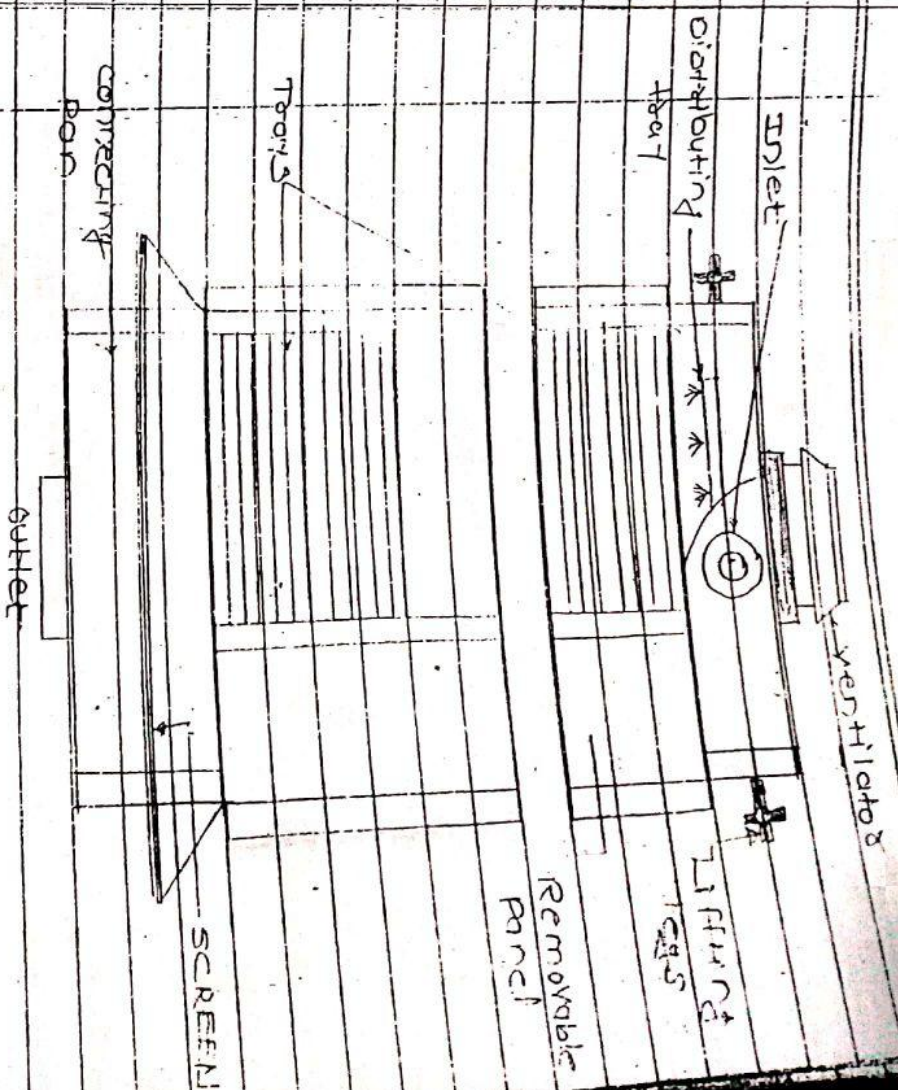
IN this type of absorber with inclined plane which is usually ripple plates in bearing form

The breaking out too sheet and consequent aeration of water

Slat tray aerator:-

This is the most commonly used aerator. It consists of closed bound square structure containing series of loosely set superimposed wood slat tray. Water enters top of the aerator and is evenly distributed over the top most tray. The salts in tray are staggered so that the films of water raining over the edges of slat in one tray fall on the center of the slat in the tray below.

Air is supplied in the bottom of water with help of blower which blows it upward a ventilator is provided at top which discharges air and gases to the atmosphere and water is collected in the collector at bottom from where it flows through sump basin or reservoir.



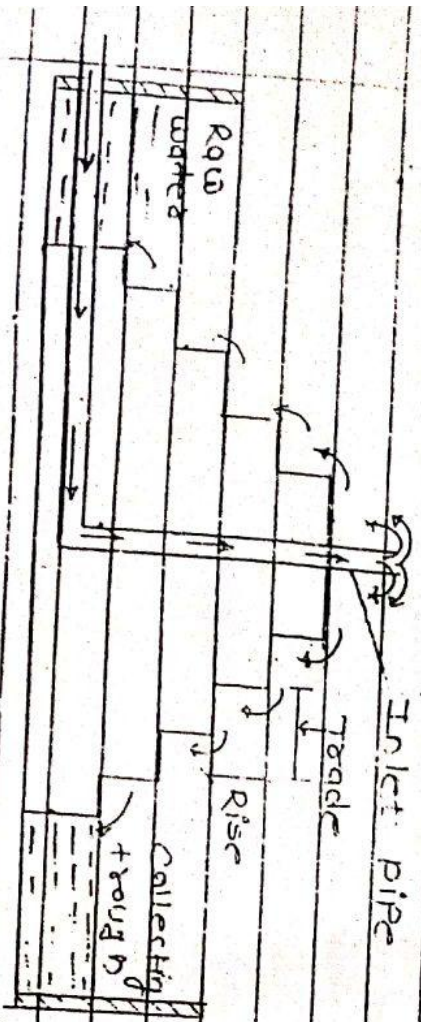
6) By using Cascade Aerator:-

In this type of aerator water is allowed to fall over a series of concrete steps or over a weir. A thin film during the fall of water gets aerated. This method can remove iron, magnesium, CO_2 , hydrogen sulphide

and other oxide and some amount of organic matter present in water.

Design criteria :-

- 1) total area required 0.015 to 0.045 $m^2/m^3/hr.$
- 2) Height of aerator 1 to 3 m (3 m)
- 3) No. of steps 3 to 9 (4)
- 4) Rise of each step 0.2 to 0.5 m
- 5) Trade of each step 0.2 to 0.4 m
- 6) Velocity of water in inlet pipe 0.6 to 0.9 m/s
- 7) CO_2 removal efficiency is 20 to 45%



Q.1

Design a cascade aerator having population 70000 and per capita demand of 140 lpcd

Given data :-

$P = 70,000$

Per capita demand = 140 lpcd

1) Design inlet pipe :-

$Q = \text{Population} \times \text{water demand}$

$= 70,000 \times 140$

$= 9.8 \times 10^6 \text{ lit/day}$

$= 9.8 \text{ MLD}$

$= 9.8 \times 10^6 \times 10^{-3}$

24

$Q = 408.33 \text{ m}^3/hr$

$= 408.33$

60×60

$Q = 0.113 \text{ m}^3/sec$

Assume the velocity of inlet in inlet pipe

$V = 0.6 \text{ m/sec}$

Area of inlet pipe $(A) = \frac{Q}{V} = \frac{0.113}{0.6}$

$= 0.188$

$$V \cdot d^2 = 4$$

$$V \cdot d^2 = 0.185$$

$$d = 0.5 \text{ m}$$

Actual Area of Inlet Pipe

$$= \frac{\pi}{4} \times 0.5^2$$

$$= 0.196 \text{ m}^2$$

Design of Aerator :-

Assume area required for cascade

$$0.03 \text{ m}^2/\text{m}^3/\text{hr}$$

$$Q = 408.33 \text{ m}^3/\text{hr}$$

Area req^d for aerator $\times Q$

$$\text{Total area of Aerotor} = 0.03 \times 408.33$$

$$= 12.25 \text{ m}^2$$

Total Area of cascade Aerotor =

Area of Actual Area of Inlet

Aerator + PIPC

$$= 12.25 + 0.196$$

$$= 12.446 \text{ m}^2$$

$$3) \text{ Area of Aerotor} = A = \frac{\pi}{4} \times D^2 = 12.44$$

$$12.446 = \frac{\pi}{4} \times D^2$$

$$D = 3.981 \text{ m}$$

Assume number of steps is five and height of aerotor 2 m

Rise of each step = $\frac{\text{height of aerotor}}{\text{No of step}}$

$$= \frac{2}{5} = 0.4 \text{ m}$$

$$= 40 \text{ cm}$$

Total of Dia of Aerotor - Dia of Inlet

each step = $\frac{\text{Dia of Aerotor}}{2}$ No. of steps

$$= \frac{(3.981 - 0.5)}{2} \times 5$$

$$= 0.35 \text{ m}$$

Q. ~~Q.1~~ W08, W12,

* GAS TRANSFER OR Factor Affecting Aeration process :-

It is the process of transfer of gas from one face to another face. This process gas is transferred from liquid phase to gaseous phase with desorption or release of gases from gaseous phase to liquid phase by absorption.

Liquid Phase $\xrightarrow{\text{Desorption}}$ Gaseous Phase

Gaseous phase $\xrightarrow{\text{Absorption}}$ Liquid phase

* Derivation of Rate of mass gas transfer :-

$$\frac{dm}{dt} \propto (C_s - c)$$

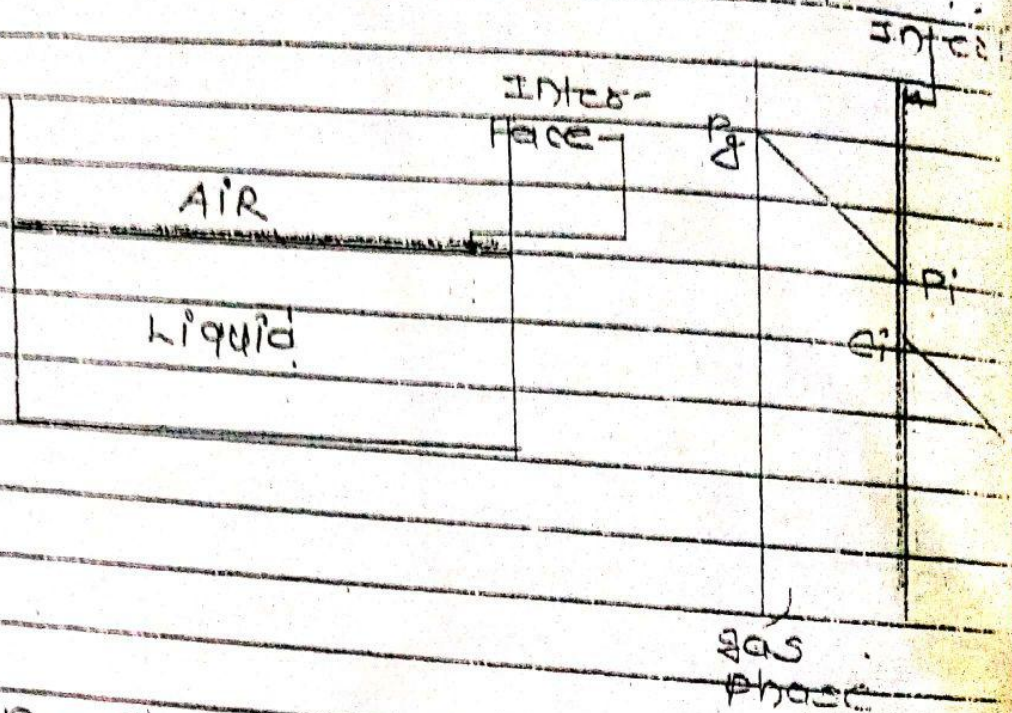
$$\frac{dm}{dt} \propto A$$

Where,

C_s = standard saturation concentration of liquid at temp t .

c = Actual saturation concentration of liquid at temp t .

TWO FILM Theory :-



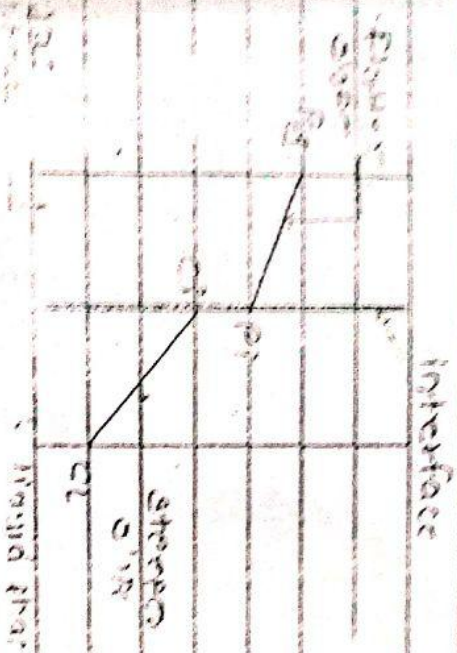
P_g = Partial pressure at gas film
 P_l = Partial pressure at interface
 C_i = Saturation conc. at interface
 C_l = Saturation conc. at liquid film.

The two film theory explains phenomenon of transfer of gas in water body from one phase to another.

Total theory of gas transfer proposed by Lewis and Whitman.

This theory of gas transfer based physical model in which two film exist liquid interface. The two film is so one is gas & another is liquid film. liquid film provides resistance to the of gas molecule between liquid & interface.

When gas is highly soluble in water the ammonia:-

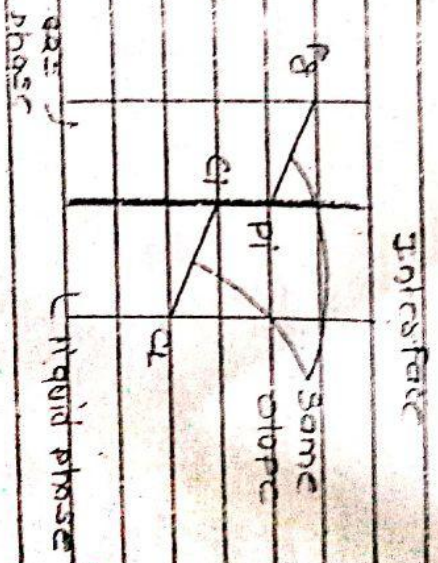


saturation conc. of intermediate interface is large and partial pressure of interface is small therefore gas can easily transfer through liquid phase but because of low pressure gradient in gas film it provides primary resistance to transfer of gas to liquid phase.



For such type of gases pressure gradient in gas film is sufficient and hence gas can be easily transferred from gas film but, concentration gradient in liquid film is very small and hence liquid film provides primary resistance for transfer of such gases.

5) Intermediate soluble gases:-



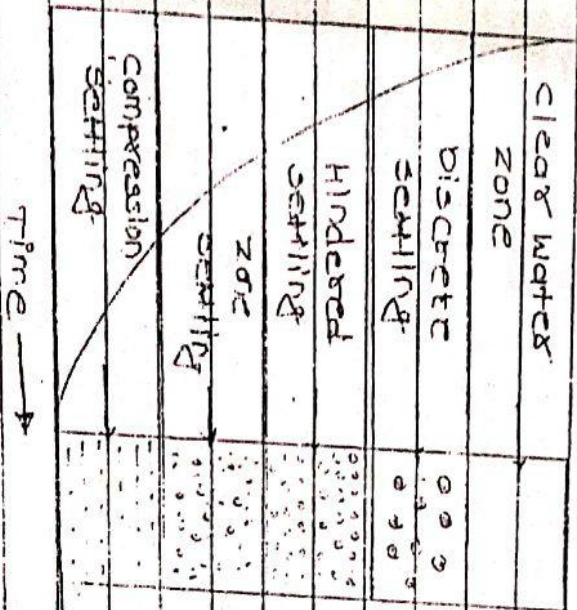
During transfer of gases intermediate solubility is present for example - the gas both films are provide primary resistance.

$$\frac{(C_g - C_l)}{(C_g - C_i)} = 0.811 \frac{k_m}{k_g + 0.5 k_l}$$

$$k_m = \pi^2 \quad \text{v.t.}$$

of- finally divided suspended and the colloidal matters.

SOB Types of settling:- S13, W12



Depending on concentration of suspended matter & characteristic of particles Fitch (1958) has classified silt sedimentation process.

- 1) Discrete settling
- 2) Hindered settling
- 3) Zone settling
- 4) Compression settling

1) Discrete settling:- This corresponds to sedimentation of discrete particles. Suspension of low solids concentration. Discrete particles are those which not undergo any change in size, shape and density during settling. Particles interact with neighbouring particles through also k/a free settling. Since the particles have little tendency to flocculate upon contact with each other.

eg:- settling of sand on inorganic material in a settling chamber.

2) Hindered settling:- This type of settling refers to dilute suspension of particles that flocculate during sedimentation. Flocculation particle increases in number and settle at faster rate.

eg:- settling in clariflocculator.

3) Zone settling:- In this type of settling, flocculent suspension of intermediate concentration. Interparticle forces keep the particles together and hence the whole mass of particles subsides as a whole.

SEDIMENTATION :-

Def. :- Separation of suspended particle by gravitation settling is k/as the sediment

Principle of sedi. :-

Most of the suspended impurities present in water do have sp. gravity more than water (sp. gravity of water = 1). In steady water this impurities will therefore tend to settle down under gravity in normal raw water supply they remain in suspension because of turbulence in water. Hence,

As soon as turbulence is retarded by offering storage to the water this impurities tend to settle down at the bottom of tank offering such storage.

Types of sedimentation :-

1) Plane sedimentation :- The impurities being separated from suspended fluid by gravitational and natural aggregation.

2) Sedimentation with coagulation :- The addition of chemical substances i.e. coagulants to increase the aggregation and settling.

where

$C_D = \text{drag coefficient}$

$$C_D = \frac{24}{Re} \quad \left(\because Re = \frac{\rho V d}{\mu} \right)$$

$$C_D = \frac{24 \mu}{\rho V d}$$

$\mu = \text{dynamic or absolute viscosity in } \text{Ns/m}^2$

$Re = \text{Reynold's number} = 0.5 \text{ to } 10^4$

$V = \text{velocity of sphere in } \text{m/s}$

$d = \text{dia. of particle in } \text{m}$

Now

Put the value of C_D in eq?

$$V_s^2 = \frac{2(\rho_s - \rho) g}{24 \times \mu \cdot g} \times \frac{2}{3} d \times \rho V d$$

$$V_s^2 = \frac{1}{18} \frac{(\rho_s - \rho) g \times d^2 \times V}{\mu}$$

$$V_s = \frac{1}{18 \mu} (\rho_s - \rho) g d^2$$

$$\frac{\rho_s}{\rho} = G$$

$$V_s = \frac{1}{18 \mu} \left(\frac{\rho_s}{\rho} - 1 \right) g d^2$$

$$V_s = \frac{1}{18 \mu} (G - 1) g d^2$$

$$\left(\because V = \frac{\mu}{\rho} \right)$$

$$V_s = \frac{1}{18 \mu} (G - 1) g d^2$$

where,

$G = \text{sp. gravity of particle}$

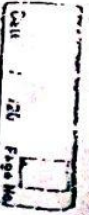
$g = \text{acceleration of gravity}$

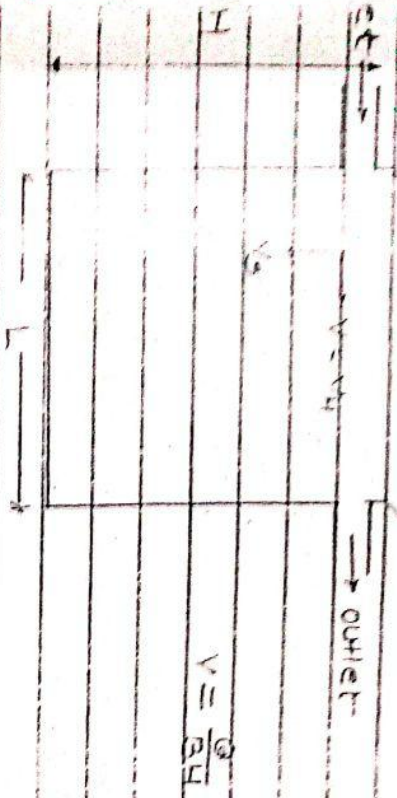
since,

viscosity is depend upon temperature
the formula of settling velocity

$$V_s = \frac{4.18 (G - 1) d^2 (3T + 70)}{100}$$

when $d < 0.1 \text{ mm}$





If a discharge entering in the basin the flow velocity is given by

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

where

B = width of the tank

H = depth of water in tank

If particle entering at top of the tank & so remove. All the particle with the same settling vel. will be remove even if they enter the tank at lesser height.

At extreme case an overflow

Figure

$$\frac{V}{V_s} = \frac{L}{H}$$

$$V_s = V \times \frac{H}{L}$$

$$V_s = \frac{Q}{BH \times \frac{H}{L}}$$

$$V_s = \frac{Q}{BL}$$

$$V_s = \frac{Q}{A_s}$$

A_s = surface area of tank

Q = discharge

A_s = surface overflow rate

1) ^{W/L} Surface overflow Rate (SOR):—

The discharge per unit area

is called surface overflow rate or

surface loading. It is assume that the

sediment is uniformly distributed &

water enter in the basin.

$$SOR = \frac{Q}{BL}$$

2) Weir loading:— The discharge per unit

parameter is called weir loading

$$\text{Weir loading} = \frac{Q}{P}$$

3) Detention time:- It is the theoretical time taken by batch of water to travel from inlet to outlet of sedimentation tank

$$D.T. = \frac{\text{Volume of tank}}{\text{Rate of flow}} = \frac{LBH}{Q}$$

Detention time for plain sedimentation tank is 2 to 4 hours flow through period

4) Flow through period:-

It is the actual time taken by batch of water from inlet to outlet on sedimentation tank. Flow through period is always less than D.T.

Design detail for sedimentation tank →
1) Detention period:-

For plain sedi. 2 to 4 hours & after coagulation of sedimentation 2 to 4 h

ii) Velocity of flow:-

Not greater than 30 cm/min.

iii) Tank dimension - L/B ratio is 3 to 5:1

If L/B ratio is more chance of short-circuiting is less. Generally L is 30 to 100 meter and width is 6 to 10 m

- Dia. of circular tank = 20 to 40 m
- Depth of tank - 2 to 5 meter
- For surface loading or surface rate for plain sedimentation for water range is 12000 to 18000 of tank area, ~~for~~ $\frac{m^3}{ha}$

+ For flocculated water 24000 to 114/day/m² of tank area.

- In horizontal flow of circular to 30000-40000 114/day/m² of tank area

iv) Slope - Rectangular tank 1% to inlet

Types of sedimentation tank
1) Circular tank -

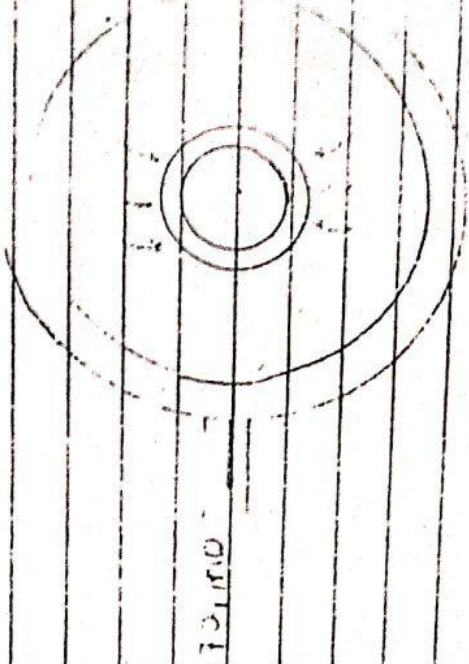
a) Radial flow with central inlet -

The water enters through centrally located shaft with multiple radial ports. The water then travels radially outwards in uniform radial flow. The velocity decreases as water travels to periphery. The clarified water is collected in peripheral annular tank.

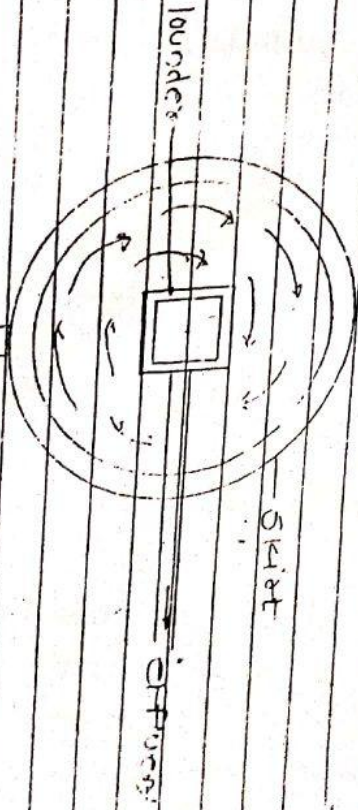
located sledge pit at the bottom with mechanical scrapers.

2) Radial flow with peripheral inlet—

In this case water enters the tank from periphery and sludge settles at removed to the centrally located outlet arrangement.



Explain with neat sketch the working of 'clariflocculator' with

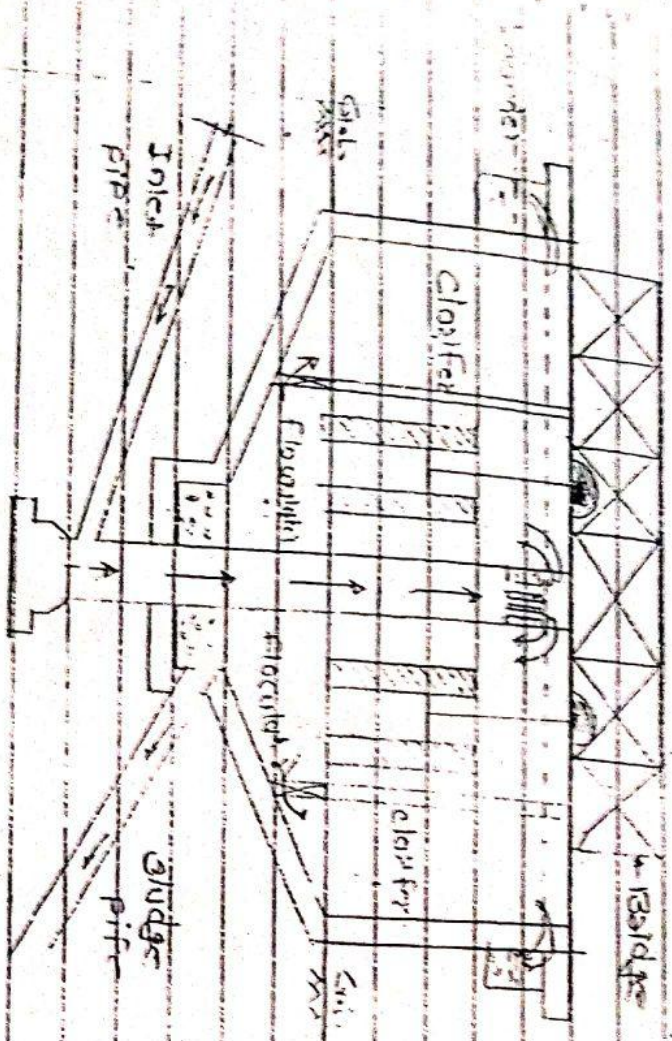


3) clariflocculator— It is generally used in the country for water treatment.

In this unit both flocculation & clarification takes place. The water mixed with chemical is feed in the flocculation compartment through the central shaft the flocculation process takes place in the centrally located tank with the help of 2 or 4 other set of paddles supported by scrapers bridge.

The clarification of water takes place in outer tank surrounding the flocculation tank the sludge settled at the bottom goes off steady condition in clarifier. The flocules get collected in centrally located sledge pit with the help of slow moving rotating scrapers where they are remove sludge (sludge pit) while the clarified water get collected in peripheral launder.

2) Conventional clariflocculator & -

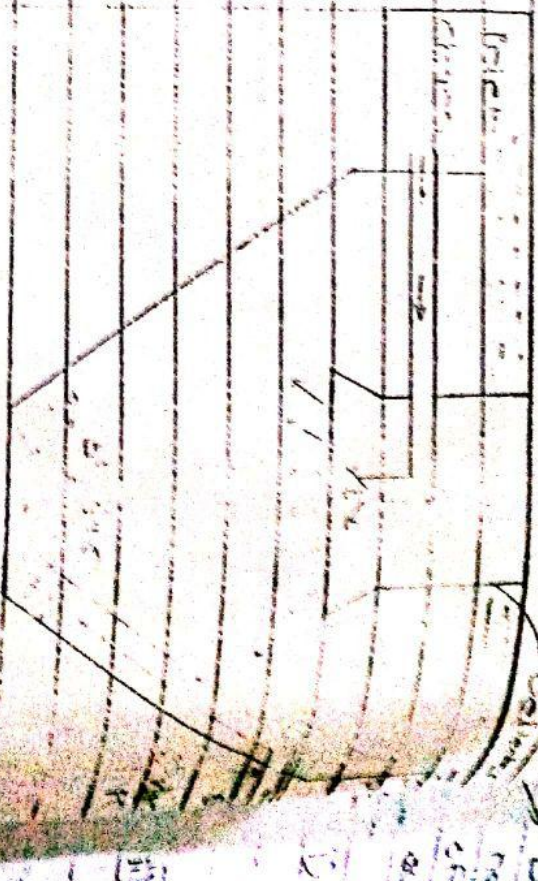


Rectangular tanks are normally suitable for small areas i.e. subequal areas the flow is longitudinal in this case the bottom of the tank the sludge is settled down & this sludge is removed by mechanical scrapers. The sludge is removed continuously manner.

Hopper bottom is provided for

collection of sludge in the mechanical scrapers.

3) Hopper bottom tank with vertical



* Design Criteria for plain settling

Tank :-

- i) Detention time - 4 to 8 hr.
- ii) Velocity of flow - 0.15 to 0.4 m/min
- iii) Depth of tank 3 to 4.5 m
- iv) width of the tank 12 m or less
- v) extra depth of sludge 25% of the
- vi) free board tank is 0.5 m

i) Design for max. water demand
 $= 2 \times \text{Average demand}$

Design sedimentation tank for a city having population 4 lacs and rate of eff. water supply 200 Lpcd assume suitable data.

i) Max demand $= 2 \times \text{Avg. dem}$
 $= 2 \times 200$
 $= 400 \text{ Lpcd}$

ii) Discharge $= Q = 400 \times 1 \times 10^5$
 $= 40 \times 10^6 \text{ lit/day}$
 $= 40 \times 10^6$
 24×1000

$Q = 1666.67 \text{ m}^3/\text{hr}$

iii) Volume of the tank = Discharge $\times$ Detention Time

assume detention time 10 hr

$V = 1666.67 \times 10$

$V = 10,000 \text{ m}^3$

iv) vel of flow 0.3 m/min assume

$= 0.3 \times 60 = 18 \text{ m/hr}$

v) Length of tank =

$DT = \frac{\text{Length}}{V}$

$6 = \frac{L}{18}$

$L = 108 \text{ m}$

vi) Width of tank - $\frac{\text{Vol of tank}}{\text{Depth} \times L}$
 (B) is given by $= \frac{10000}{\text{Depth} \times L}$
 (depth = 4 assume) $B = 2500$

$B = \frac{10000}{L \times D} = \frac{10000}{108 \times 4} = 23.15 \text{ m}$

$\therefore$ Provide two tanks i.e. $108 \times 11.5 \times 4$

vii) Provide extra depth for sludge = 25%

total dept

Free Board + sludge depth

$4 + 0.5 + 1 = 5.5 \text{ m}$

$\rightarrow$ Provide two units $= 108 \times 11.5 \times 5.5$

2)

750 primary settling basins of 25 m diameter with 2.5 m size of water depth influent flows are located on the periphery of tank for water flow 30000 m³/day. find out

- i) surface area & volume
- ii) overflow rate in m³/m²/day
- iii) detention time in years hr.
- iv) weir loading m³/m/day

Solution:-

i) Vol^m of tank = $V = D^2 (0.0110 + 0.785H)$
 $= 25^2 (0.011 \times 2.5 + 0.785 \times 2.5)$
 $= 1398.44 \text{ m}^3$

ii) $Q = 20000 \text{ m}^3/\text{day}$
 $Q = 15000 \text{ m}^3/\text{day}$ (conc tank)
 $Q = 625 \text{ m}^3/\text{hr}$

iii) OT = Vol^m of tank / Discharge
 $= 1398.44 / 625 = 2.237 \text{ hr}$

iv) $Q = 625 \text{ m}^3/\text{hr}$
 $A_s = \text{circular tank} = 0.785 D^2$
 $= 490.625 \text{ m}^2$

25 m dia
 2.5 m depth

$Q = 490.625$
 $Q = 1.274 \text{ m}^3/\text{hr}/\text{m}^2$
 $Q = 1.274 \times 24$
 $Q = 30.57 \text{ m}^3/\text{day}/\text{m}^2$

iv) Weir loading = Q/P
 $P = \text{Perimeter of tank} = \pi \times D$
 $= 78.54$
 $\text{Weir loading} = 15000 / 78.54$
 $= 190.98 \text{ m}^3/\text{day}$

3) A settling basin is designed to treat 1200 lit/hr/m². What will be the following diameter will be tank

i) 0.01 mm ii) 0.03 mm
 Assume kinematic viscosity of water 1.01 mm²/sec and specific gravity of 2.65.

Solution:-
 $Q = \text{Discharge} = 1200 \text{ lit/hr}$
 $A_s = 1 \text{ m}^2$

$V_s = \frac{Q}{A_s} = \frac{1200}{1} = 1200 \text{ lit/hr}$

Case 5/1/12/12/12/12

2) For $d = 0.01 \text{ mm}$

$$V_s = 418 (G - 1) d^2 \left(\frac{31 + 70}{100} \right)$$

$$= 418 (2.65 - 1) \times 0.01^2 \left(\frac{31 + 70}{100} \right)$$

$$= 0.66897 \text{ mm/sec}$$

% Removed Particle = $\frac{V_s}{V_t} \times 100$

$$= \frac{0.66897}{0.33} \times 100$$

$$= 20.9 \%$$

For $d = 0.03 \text{ mm}$

$$V_s = 418 (2.65 - 1) \times 0.03^2 \left(\frac{31 + 70}{100} \right)$$

$$= 0.62 \text{ mm/sec}$$

Removed Particle = 0.62

$$\frac{0.53}{0.53} \times 100$$

$$= 187.88\% \sim 7100$$

The particle of dia 0.03 mm are settled removed because the 187.88% > 100.

Tuesday
05/11/2012

4)

A rectangular sedimentation tank is treating $2.5 \times 10^6 \text{ lit/day}$ the size of tank is $17.5 \times 5.5 \times 3.5$ if 80 ppm suspended solids are present in water assuming the % removal in basin and the average specific gravity as two determine following

i) Detention time

ii) Deposition of solids in tank per day

sol $\rightarrow$

Volume = $17.5 \times 5.5 \times 3.5$

$$= 336.875 \text{ m}^3$$

$Q = 2.5 \times 10^6 \text{ lit/day}$

$$= 104.17 \text{ m}^3/\text{hr}$$

i) $D.T = \frac{\text{Volume}}{\text{Flow rate}} = \frac{336.875}{104.17} = 3.23 \text{ hr}$

Quantity of water treat in water tank will be $2.5 \times 10^6 \text{ lit/day}$. concentration of suspended particle is 80 ppm

Quantity of suspended solid entering per day

$$= 2.5 \times 10^6 \times \frac{80}{10^6}$$

$$= 200 \text{ kg}$$

$$= 0.2 \text{ m}^3$$

Sedimentation with coagulation :-

- Feeding of coagulant

Mixing

Hdraulic (Baffle mixing)

Mechanical (Flag mixer)

- Formation of floc

- Sedimentation

Coagulation :-

When there are less turbidity in water plain sedimentation is not suitable hence sedim. with coagulation is used in this condition)

In this process chemical is added in water which react with water and form gelatinous precipitate which called as floc. This floc is remain in suspension form in water. The suspended particulate matter present in raw water comes in contact with floc and get stick with floc as the floc get removed in sedimentation the suspended particle also get remove with floc. Such process of removal of suspended particle & colloidal particle by addition of chemical is called coagulation.

Coagulant :-

The chemical which is added the process of coagulation is called as coagulant.

Flocculation :-

The process of maintaining in suspension form is called flocculation. The unit is called flocculator.

ASS. TYPES OF COAGULATION :-

CO-AGULATION AIDS :-

Some times the addition of only coagulant does not helping to of suspended or colloidal particles in case certain chemicals added to water along with main coagulant coagulant is called as coagulant

50% Explain various types of mixing devices :-

100%

The mixing process first mix the coagulant with water but to get effect upon the formation of

It is usually to introduced the chemical some point of high turbulence in

after the dose of chemical added

water. so mix.

Compressed Air agitation :-

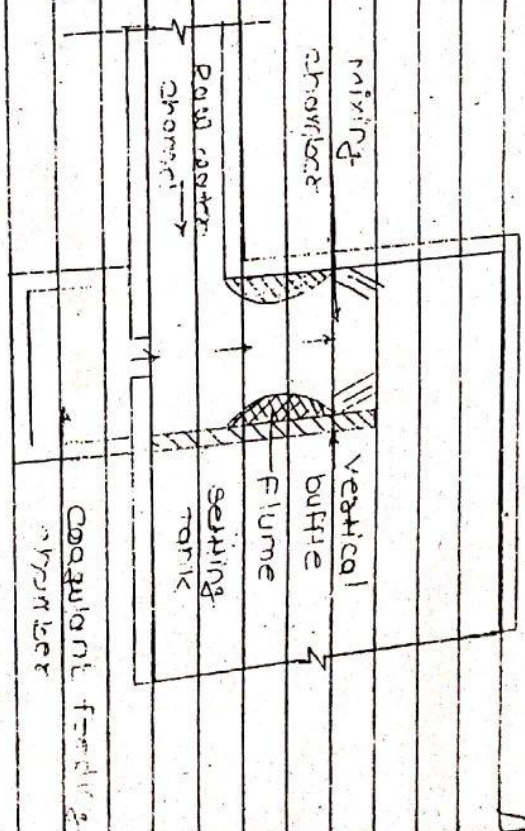
In this method the raw water feed with coagulant is agitated or mixing vigorously by diffusing compressed air at the bottom of mixing basin.

Centrifugal Pump :-

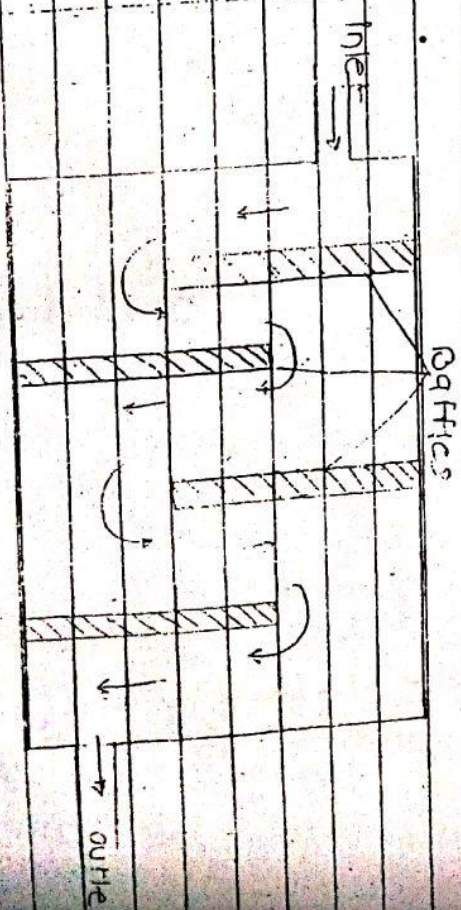
Centrifugal Pump is used to raise the raw water to settling tank the required dose of chemical can be added to the suction line of the pump when water feed with the coagulant passes through the impeller of the pump mixing is accomplished by agitation.

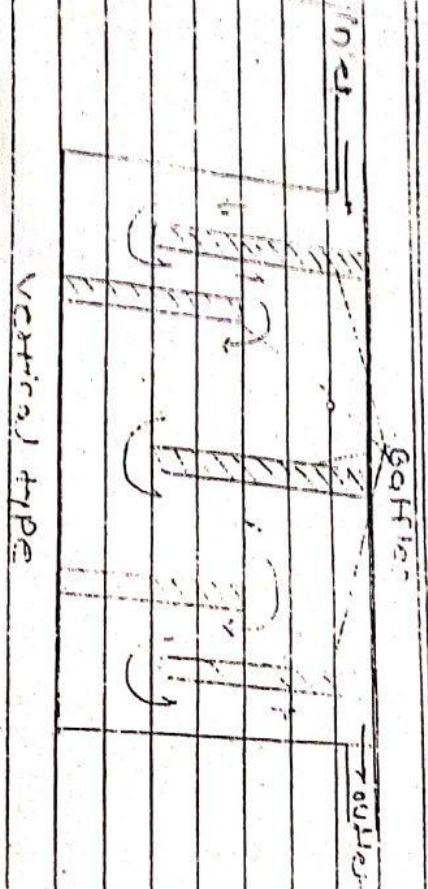
Narrow mixing channel with flume :-

Coagulant is feed from the feeding tank the turbulence is caused by vertical baffle mixes thoroughly a flume is some time provided to develop hydraulic jump for causing turbulence & also to measure the flow.



4) Mixing basins with baffle walls :-





Vertical type

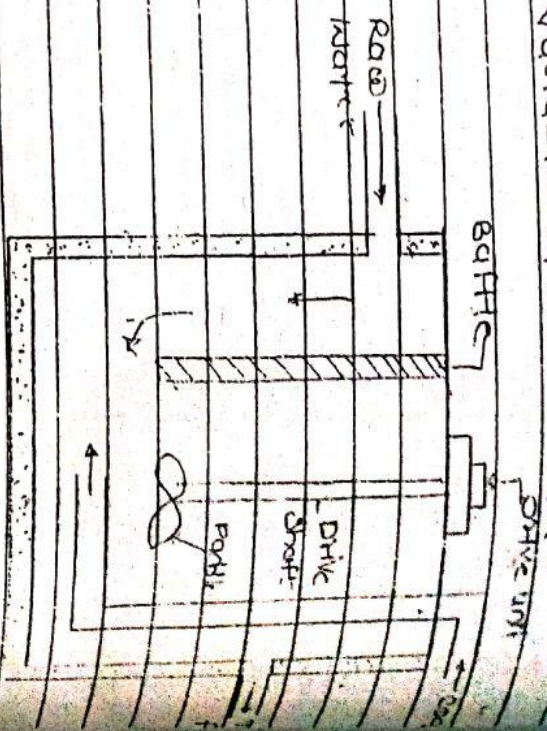
The water flows horizontally for a short distance mix a complete tank and continues back and forth around the end of baffles. This causes turbulence and hence the mixing is done properly this type of basin is called as horizontal type while another type i.e. vertical use in smaller plants has vertical hanging baffle due to which water flows up and down.

Mixing basins with baffle wall are generally not use because of high head losses & variation in velocity.

5) Flash Mixture :-

Most of the modern water treatment plant use flash mixture as a mixing device in which the raw water and coagulant are injected vigorously by paddle operated by

Variable speed motor



* Design Criteria for Flash mixture

- 1) Detention time 30 to 60 sec
- 2) Speed of paddles 80 to 100 rpm
- 3) Power requirement 1 to 3 W/m³
- 4) Depth of tank 1 to 3.5 m
- 5) Velocity at the tip of blades 3 m/sec
- 6) Height to dia. ratio 1:1 to 3:1
- 7) Ratio of impeller dia to tank dia 0.8
- 8) Free board 0.3 m
- 9) The value of 'G' = 300 per sec or min
- 10) Absolute viscosity of water 1.008 x 10⁻³

Mean velocity gradients (G) :-

It is define as the rate of change of vel. per unit distance normal to the section. its unit is per sec⁻¹

$$G = \sqrt{\frac{P}{\mu V}}$$

where,

P = Power dissipated in water

μ = Absolute viscosity N sec/m²

V = Volume to which P is applied in m³

G = Mean vel. gradient sec⁻¹

If vel. gradient G is combined with detention time

$$G \cdot D.T = \frac{V}{Q}$$

$$G \times D.T = \left[\frac{P}{\mu V} \times \frac{V}{Q} \right] = \frac{\sqrt{P V / \mu}}{Q}$$

The above exp. $\sqrt{P V / \mu}$ is term as power induces flow while Q is k/a displacement.

hence, G.D.T is ratio of power induce rate of flow to displacement induce rate of flow.

The total no. of particle collision is proportional to Gt hence it is useful parameter to design a flash mixture in flocculator.

For other type of flocculator

G = 20 to 75 sec⁻¹ and Gt is 1×10^4 to 1.6×10^5 Gt

1) Design suitable flash mixture for design flow 17 MLD draw neat sketch,

Sol:-

i) Q = 17 MLD

$$Q = \frac{17 \times 10^6 \times 10^{-3}}{24}$$

$$Q = 708.33 \text{ m}^3/\text{hr}$$

$$Q = 0.1967 \text{ m}^3/\text{sec}$$

ii) Design tank for flash mixture - Assume inlet velocity at inlet pipe 0.9 m/sec so

$$\text{Area of pipe} = \frac{Q}{V} = \frac{0.1967}{0.9} = 0.22 \text{ m}^2$$

$$A_p = \pi \times \frac{D^2}{4}$$

$$0.22 = \pi \times \frac{D^2}{4}$$

$$D_p = 0.53 \text{ m}$$

iii) Vol. of tank - DT = V/G Assume DT. 40 sec

$$40 = \frac{V}{0.1967}$$

$$V = 7.87 \text{ m}^3$$

$$\text{Area of tank} = \frac{\text{Volume}}{\text{Depth}}$$

$$= \frac{9.87}{0.87}$$

Assume dia of flash mixture 2.5 m

$$= 2.5$$

$$\text{Area of tank} = 8.15 \text{ m}^2$$

$$8.15 = \frac{\pi}{4} \times D^2$$

$$D = 2 \text{ m}$$

IV) Total depth = dia of inlet pipe + depth of water tank + F.B.

$$= 0.8 + 2.5 + 0.3$$

$$\text{Assume F.B.} = 0.3 \text{ m}$$

$$\text{Total depth} = 4.18 \text{ m}$$

Provide flash mixture tank of dia 2 m and depth is 3.8 m

3) Design of Impeller — dia of impeller D_i

Assume D_i

$$D_i = 0.3$$

$$\text{Dia of impeller} = \frac{D_i}{2} = 0.3$$

$$F.N. = 0.1 \text{ m}$$

To find velocity at tip of blade

$$V_p = \frac{2\pi r n}{60}$$

Assume $n = 100 \text{ rpm}$ ($n = \text{rev}/100$)

$r = \text{Radius of impeller} = 0.6 \text{ m}$

$$V_p = \frac{2\pi \times 0.6 \times 100}{60}$$

$$60$$

$$V_p = 3.14 \text{ m/s}$$

IV) Find out power required for impeller

$$G_i = \frac{P}{\rho V}$$

$$G_i = 0.2 \text{ m}^3/\text{s}$$

$$\frac{11.86}{0.1} = \frac{4.29}{0.1} - 1.86 \text{ m/s}$$

$$11.86 = \pi \times 11.86 \times 10^2$$

$$D = 11.86 \text{ m}$$

(iv) total depth = $0.91 + 0.4 + 0.3$
 $= 1.61 \text{ m}$

Design of Impeller:-

$$\frac{D_1}{D_2} = \frac{8.13}{1.44}$$

$$\frac{D_1}{D_2} = 5.64$$

$$D_1 = 0.462 \text{ m} = 46.2 \text{ cm}$$

To find velocity at HP of blade-

$$V_p = \frac{2\pi r n}{60} = \frac{2\pi \times 0.23 \times 1000}{60}$$

$$V_p = 2411 \text{ m/s}$$

$$V_p = 8.225 \text{ m/s}$$

Find out power required for impeller

$$P = \frac{F \cdot V}{1000}$$

$$P = 2411 \text{ W}$$

$$P = (300) \times 1.005 \times 10^3 \times 0.24$$

$$P = 424.94$$

$$416.47$$

$$P = 1.01 \text{ watt (m/s)}$$

Design of blade

$$P = 1/2 \rho A_b \cdot f (N_r)^3$$

$$V_p = 0.75 \times V_p = 0.75 \times 8.225 = 2.91$$

$$\therefore 424.94 = \frac{1}{2} \times 1.8 \times A_b \times 1000 \times (2.91)^3$$

$$A_b = 0.033 \text{ m}^2$$

$$\text{Area of one blade} = 0.033/2 = 0.0165$$

$$A = L \times B$$

$$0.0165 = L \times 0.05$$

$$L = 0.33 \text{ m}$$

Provide blade 0.33 x 0.05 m

Classification:-

Types of Classification

1) Orthokinetic Floculation - 808,

When the coagulant is added in water, during the mixing process it reacts with alkalinity present in water. A ferm micro floc, this process is kinetic floculation. This process is take place in rapid unit i.e. Flash mixer efficiency of entire floculation process is completely depend on phenokinetic floculation.

2) Orthokinetic Floculation - 808

Formation of bigger and

compact floc by combination of micro

floc is called as orthokinetic floculation. This process takes place in floculator unit.

Orthokinetic floculation is fully depend upon chemical acceleration. orthokinetic floculation is responsible for removal suspended particulate matter.

Floculation factor affecting floculator

3) Coagulation process

type of coagulant

Quantity or dose of coagulant

5) Characteristics of water such as

- a) Type and quantity of suspended matter
- b) Temperature of water
- c) pH of water

4) Time, method of mixing

The rate of formation of floc is influence more on the temp. than on the amount of coagulant through mixing of coagulating chemical with water accomplished by short period of violent agitation this is then followed by a longer period of conditioning involving gentle stirring during which the desired floc is developed

Design criteria for clarifloculator-

- 1) Detention time is to 40 min, normally 30 min
- 2) Depth of floculator is to 4.5 m
- 3) Total area of paddles = 10 to 25% of c/s area of tank
- 4) Velocity of paddle (V_p) = 0.2 to 0.9 m/s
- 5) Peripheral velocity of water 30 to 40% vel. of paddle
- 6) Vel. gradient (G) = 10 to 80/sec
- 7) Radial dist of paddle = (radius of floculator - 0.5 to 4 m)

8) Multiplication Factor (Gt) = 10^4 to 10^5

Design criteria for sedimentation tank or clarifier :-

- 1) Detention time 2 to 4 hr
 - 2) Depth of water in tank 3 to 4.5 m
 - 3) Surface overflow rate for circular tank 1 to 1.5 $m^3/hr/m^2$ for rectangular
- For Rectangular 1 to 1.25 $m/hr/m^2$
- 4) Clear loading 100 to 200 $m^3/day/m$
 - 5) Depth of 90° V-notch collecting 5 cm
 - 6) C/c distance of 90° V-notches is 20 to 30 cm
 - 7) Velocity of flow in collecting channel 1's 0.2 to 0.4 m/s

- 8) Width of collecting channel 0.5 to 1 m
- 9) Length of paddle = Depth of flocculator = 0.5 m

1) Design clariflocculator of design flow 350 m^3/hr

2) Design flocculator -

$$Q_1 = 350 \text{ } m^3/hr$$

$$Q = 0.097 \text{ } m^3/sec$$

3) Design of inlet pipe -

Assume vel. of flow = 0.9 m/s

$$A_{pipe} = \frac{Q}{V} = \frac{0.097}{0.9} = 0.108 \text{ } m^2$$

$$\text{Diameter of inlet pipe} = \frac{\pi}{4} \times d^2 =$$

$$d = 0.37 \text{ } m$$

3) Design flocculator unit -

$$Vol^n \text{ of tank} = \text{Area} \times \text{Depth}$$

$$D.T. = \frac{V}{Q} \quad \text{Assume D.T.} = 30 \text{ min.}$$

$$Vol^n \text{ of tank} = 20 \times 60 \times 0.097$$

$$Vol^n = 114.6 \text{ } m^3$$

Assume depth of tank 4 m

$$174.6 = 4 \times 4$$

$$\text{Area of tank} = 45.65 \text{ } m^2$$

$$\pi \times D^2 = 45.65$$

$$D \text{ of flocculator} = 7.45 \text{ } m$$

4) Design of paddle -

Total area of paddle $\leq 15\%$ of c/s area of tank

$$= \frac{15}{100} \times 45.65$$

$$= 6.85 \text{ } m^2$$

Vertical c/o area = Dia x L

$$= 29.8$$

Area of Paddle = 15% vertical c/o area

$$= \frac{15}{100} \times 29.8$$

$$= 4.47 \text{ m}^2$$

Assume 4 no. of Paddles

$$\text{Area of each Paddle} = \frac{4.47}{4} = 1.12$$

$$\text{Assume length of Paddle} = 4 - 0.7 (\text{end board}) = 3.3 \text{ m}$$

width of Paddle

$$\text{Width of Paddle} = \frac{\text{Area}}{\text{Length of Paddle}} = \frac{1.12}{3.3} = 0.32 \text{ m}$$

Assume V_p (Velocity) = 0.3 m/s

$$V_{01} (\text{Vel. at periphery}) = 80 \text{ to } 90\% = 80\% \text{ of } V_p$$

$$= \frac{80}{100} \times 0.3$$

$$= 0.09$$

Assume provided dist. of Paddle = Radius of tank

$$= 0.77$$

$$= 7.4 - 0.77$$

$$V_p = \frac{2\pi r n}{60}$$

$$0.3 = \frac{2 \times \pi \times r \times n}{60 \times 2.95}$$

$$n = 0.97 \text{ rpm} \approx 1 \text{ rpm}$$

Provide 4 no. of Paddle of size 2.97×0.32 with speed of 1 rpm.

Check for design parameter:-

$$\text{Power dissipated (P)} = \frac{1}{2} C_D \times A \times \rho \times (V_s)^3$$

$$V = V_p - V_0 = 0.3 - 0.09 = 0.21 \text{ m/s}$$

$$= \frac{1}{2} \times 1.8 \times 4.47 \times 1000 \times (0.21)^3 = 37.26 \text{ watt}$$

$$\text{Velocity gradient (G)} = \sqrt{\frac{P}{\mu \times V}}$$

$$\text{Assume } \mu = 1.008 \times 10^{-3} \text{ N s/m}^2$$

$$P = G^2 \times \mu \times V$$

$$37.26 = 1.008 \times 10^{-3} \times 174.6 \times G^2$$

$$G = 14.55 \text{ sec}^{-1} \left\{ 16 - 80 \text{ sec}^{-1} \right\}$$

$$G_1 = 14.55 \times 80 \times 60$$

$$= 26190.36 \times 10^4 = 10^5$$

Assume total depth of flocculator = depth of clarifier + depth of sludge

$$\text{depth of sludge} = \frac{25}{100} \times \text{total depth}$$

$$= \frac{25}{100} \times 4$$

$$= 1$$

$$\text{Total} = 4 + 1 + 0.3$$

$$= 5.3 \text{ m}$$

2) Design of clarifier:- (settling tank)

Detention time = 3 hr

$$Q = 550 \text{ m}^3/\text{hr}$$

$$\text{Vol}^n \text{ of tank} = Q \times DT$$

$$= 350 \times 3$$

$$\text{Vol}^n \text{ of clarifier tank} = 1050 \text{ m}^3$$

$$\text{Vol. flocculator} = 174.6 \text{ m}^3 \checkmark$$

$$\text{Total Vol}^n \text{ of clarifier} = \text{Vol}^n \text{ of flocculator} + \text{Vol}^n \text{ of clarifier}$$

$$= 1224.6 \text{ m}^3$$

$$\text{Depth of water} = 4 \text{ m}$$

$$\text{Area of clarifier} = \frac{\text{Vol}^n \text{ of tank}}{\text{Depth of water}}$$

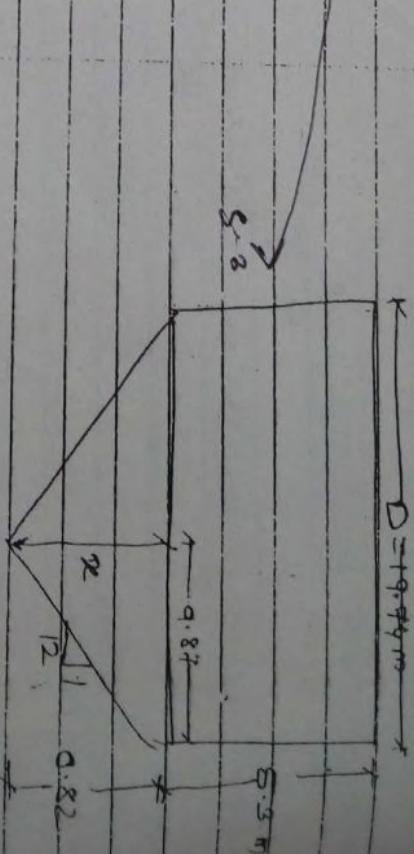
$$= \frac{1224.6}{4}$$

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

$$306.15 = \pi \frac{D^2}{4}$$

$$D = 19.74 \text{ m}$$

To find total depth of clarifier



$$\frac{1}{12} = \frac{x}{9.87}$$

$$x = 0.82 \text{ m}$$

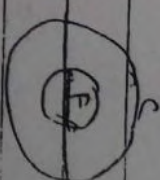
$$\text{Total ht} = 0.82 + 5.3 = 6.12 \text{ m}$$

3) Check for surface overflow rate

$$A_c = C \cdot F$$

$$= 19.74 \cdot 7.95$$

$$= 12 \text{ m}$$



$$= 1.3$$

$$= \frac{\pi}{4} (0.2^2 - 0.1^2)$$

$$= \frac{\pi}{4} (0.2^2 - 0.1^2) = 7.46 \text{ m}^2$$

$$= 262.45 \text{ m}^2$$

Check for weir loading:-

$$\text{Weir loading} = \frac{Q}{\text{Perimeter}} = \frac{350}{\pi D}$$

$$= 340$$

$$\pi \times 18.28$$

$$= 5.64 \text{ m}^3/\text{hr/m}$$

$$= 6.09 \text{ m}^3/\text{hr/m}$$

$$= 146.2 \text{ m}^3/\text{hr/m}$$

ok

Design of Weir:-

Assume 90° V-notch all over the Perimeter of tank.

Spacing of V-notch & distance 2420 to 30

cm. Assume C/C dist = 20 cm

$$\text{Perimeter} = \pi D = \pi \times 18.28$$

$$= 57.201 \text{ m}$$

$$\text{No. of V-notches} = \frac{\text{Perimeter}}{\text{Spacing}}$$

$$= \frac{57.201 \times 10^2}{20} = 286 \text{ Nos.}$$

Discharge per V-notch = 350

$$\frac{350}{310}$$

$$= 1.13 \text{ m}^3/\text{hr/notch}$$

$$= 3.189 \times 10^4 \text{ m}^3/\text{hr}$$



$$Q = 1.417 (h)^{5/2}$$

$$3.189 \times 10^4 = 1.417 (h)^{5/2}$$

$$h = 0.034 \text{ m}$$

$$h = 3.4 \text{ cm}$$

Provide 90° V-notch of height 3.4 cm

D) Design of collecting channel:-

$$Q = \frac{350}{2} = 175 \text{ m}^3/\text{hr}$$

Assume width of collecting channel = 0.75

vel. of flow in collecting channel 0.3 m/s

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

$$A = \frac{175 \text{ m}^3/\text{hr}}{0.3 \text{ m/s}} = 0.0486 \text{ m}^2/\text{s}$$

$$= 0.162 \text{ m}^2$$

(1) Difference between Rapid sand filter and slow sand filter (3 marks)

| | SDF | RDF |
|-------------------|--|---------------------------------------|
| Rate of Filter | 0.1 to 0.2 m ³ /hr/m ² | 3-6 m ³ /hr/m ² |
| Capacity required | | |

Types of sand

| Size of sand | |
|-------------------------|--------------------------|
| $D_{10} = 0.2$ to 0.4 | $D_{10} = 0.35$ to 0.6 |
| $C_u = 1.8$ to 3 | $C_u = 1.8$ to 1.7 |

Method of cleaning

Cleaning is done by back washing removing top layer is done of sand (15 to 20 cm) and replace with new sand

Period of cleaning

Rate of removal

| | |
|---------------|-------------|
| 1 to 3 months | 2 to 3 days |
| 95 to 99 % | 80 to 90 % |

8) Quality of water obtain

Good

Moderate

9) Cost of economy

Less equipment cost acquired & maintenance cost because of large area required hence it is uneconomical.

10) Skilled supervision

Not Required

Required

~~Design~~ Design criteria of Rapid sand filter

- 1) Rate of filter 3 to 6 m³/hr/m²
- 2) Surface area 10 to 180 m²
- 3) Depth of filter 2.5 to 3.5 m
- 4) L/B = 1.1 : 1.66
- 5) No. of filter unit = $\frac{Q}{q}$ $\frac{1.1}{4.69}$ $\sqrt{0.4}$

where, $q = m^3/hr$

6) Quantity of backwash water 2 to 6% of total quantity of water

Design criteria for underdrainage system -

- 1) Total area for perforation = 0.15 to 0.5 % of total filter area
- 2) Total ext area of lateral = 2 to 4 times area of perforations
- 3) Total Area of manifold = 1.5 to 3 times of area of lateral.
- 4) Dia. of perforations = 6 to 18 mm
- 5) Spacing of lateral = 15 to 30 cm
- 6) Length of lateral on each side of manifold = > 60 times of dia. of orifice or perforations.
- 7) Dia of lateral = 6 to 12 cm

Design criteria for backwashing or wash water tube :-

- Rate of back wash = 18 to 54 m/hr/m²
 width of tube = (b) = 0.2 to 0.4 m
 spacing of tube = < 1.8 m
 $Q = 1.878 \times U \times h^{3/2}$

- 1) Number and size of rapid sand filter for design flow 18 MLD 400 des under drainage system.

Solⁿ:- A) Design of filter :-

Assume that 3% of filter area is used for backwashing of filter per day.

$$\therefore \text{Filter water required} = \frac{100}{100} = 1\% = 0.03 \times 18 = 1.03 \text{ MLD}$$

$$\rightarrow 1.03 \times 18 = 18.54 \text{ m}^3/\text{D}$$

Assume 30 min required for backwash

$$\therefore \text{The quantity required for filter water per hr.} = \frac{18.54}{(24 - 0.5)} \quad (\text{as } 30 \text{ min})$$

$$= 0.79 \text{ million lit/hr} = 788 \text{ m}^3/\text{hr}$$

$$\text{Ass. avg. rate of filtration } 5 \text{ m/hr} = 5000 \text{ lit/hr/m}^2$$

$$\text{Area of filter} = \frac{\text{Dis. of filter}}{\text{Rate of Filtration}}$$

$$= \frac{788}{5} = 157.6 \text{ m}^2$$

$$= 157.6 \text{ m}^2$$

$$\text{No of filter unit} = \frac{Q}{q} = \frac{1788}{4.69} = 380.8 \approx 381$$

$$= 5.98$$

$$= 26 \text{ m}$$

$$\text{Assume } L/B \text{ ratio} = 1.23$$

$$\text{Area req. per filter unit} = (A_0) = \frac{157}{6}$$

$$= 26.17 \text{ m}^2$$

$$\text{Area of each unit filter} = \frac{L}{B} = 1.23$$

$$L = 1.33 B$$

$$26.66 = L \times B$$

$$26.66 = 1.33 B \times B$$

$$B = 4.48 \text{ m}$$

$$L = 5.9 \text{ m}$$

Provide 6 no. of filter unit having length

5.9 m and width 4.48 m

3) Design under-drainage system:-

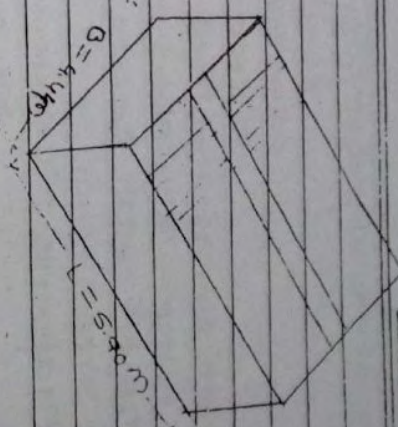
$$B = 4.48 \text{ m manifold}$$

$$L = 1.85 \text{ m}$$

$$\text{dia of lat} = 6 \text{ cm}$$

$$\text{dia of lat} = 6 \text{ cm}$$

$$15 \text{ cm}$$



The total area of perforation in under drainage system to be taken as 0.3% of entire area of filter.

$$\text{Total Area of perforation} = 0.3\% \times 4.48 \times 5.9$$

$$= 0.0786 \text{ m}^2$$

Assume dia. of perforation 12 mm

Total Area of lateral = 2 times the total Area of perforation

$$= 2 \times 0.0786$$

$$= 0.157176 \text{ m}^2$$

The total area of manifold = 2 x Area of lateral

$$= 2 \times 0.157176$$

$$= 0.314 \text{ m}^2$$

$$\text{Dia. of manifold} = \sqrt{\frac{0.314 \times 1}{\pi}}$$

$$= 0.682 \text{ m}$$

$$\approx 68 \text{ cm}$$

Provide dia of manifold 0.65 m laid along the filter unit. Place the laterals at spacing of 15 cm centre to centre

$$\text{No. of laterals} = \frac{\text{Length of filter}}{\text{spacing of lateral}}$$

$$= \frac{5.9}{0.15}$$

$$= 39.33$$

$$\approx 39 \text{ nos. } 40 \text{ no.}$$

Provide 40 no. of lateral on both side by central manifold.

$$\text{Length of each lateral} = \frac{\text{width of dia of manifold}}{2}$$

$$= \frac{4.44 - 0.65}{2}$$

$$= 1.895 \text{ m}$$

$$\text{Total area of perforation} = 0.0785 \text{ m}^2$$

$$\text{dia. per } \leq 12 \text{ mm}$$

$$N \times \frac{\pi}{4} \times d^2 = 0.0785$$

$$N \times \frac{\pi}{4} \times (12)^2 = 0.0785 \times 1000^2$$

$$N = 694.91$$

$$N \approx 695 \text{ nos.}$$

$$\text{No. of perforation in each lateral} = \frac{80 (\text{no. of lateral})}{695}$$

$$= 8.6875$$

$$\approx 9 \text{ no.}$$

$$\text{Area of lateral perforation} = 9 \times \frac{\pi}{4}$$

$$= 9 \times \frac{\pi}{4}$$

$$= 19171$$

$$\text{Area of each lateral} = 2 \times \text{Area of lateral}$$

$$= 2 \times 1017.87$$

$$= 2035.75$$

$$\text{dia of lateral} = 50.90 \text{ mm}$$

$$= 6 \text{ cm}$$

Design wash water trough :-

Assume a no. of trough having width of 4.4 m

$$\text{Spacing of trough} = \frac{\text{width of filter}}{\text{No. of trough}} = \frac{4.4}{3} = 1.48 \text{ m}$$

3a. discharge per trough = catch water discharge from one filter
No. of filters

$$\text{Assume wash water rate} = 36 \text{ m}^3/\text{hr}/\text{m}^2$$

$$\begin{aligned} \text{Wash water discharge in one filter} &= 36 \times 4.4 \times 5.90 \\ &= 943.056 \text{ m}^3/\text{hr} \times \\ &= 0.262 \text{ m}^3/\text{sec} \end{aligned}$$

$$\begin{aligned} \text{Discharge per trough} &= 0.262 \\ &= 0.08732 \text{ m}^3/\text{sec} \end{aligned}$$

The water depth at upper end is given by

$$\text{Discharge (a)} = 1.376 \times \text{box}(y)^{3/2}$$

$$\begin{aligned} \text{Assume width of trough (b)} &= 0.4 \text{ m} \\ \text{Box} &= 1.376 \times 0.4 \times (y)^{3/2} \\ y &= 0.29 \text{ m} \end{aligned}$$

$$\begin{aligned} \text{Free board} &= 0.1 \text{ m or } 10 \text{ cm} \\ \therefore \text{total depth (Y)} &= 0.29 + 0.1 \\ &= 0.39 \text{ m} \end{aligned}$$

Provide a no. of wash water trough of size 40 cm x 89 cm

D) Determination of depth of sand in
 309
 Assume 80 cm of depth of sand having effective size 0.5 mm

Size of gradation of gravel 2mm at the top, 5mm, 10mm, 20mm, at bottom 40mm the necessary length or depth of each gravel layer is calculated as

$$1 = \text{depth of gravel} = 2.54 \text{ K} (\log d)$$

$$\begin{aligned} \text{media} &= 0.5 \text{ mm (s.s.)} \\ \text{depth of water} &= 1.2 \text{ m} \\ \text{depth of gravel} &= 0.60 \text{ m} \end{aligned}$$

| | |
|--------|-------|
| Gravel | 5 mm |
| media | 10 mm |
| | 20 mm |
| | 40 mm |

| size of
sieve (mm) | 2 | 5 | 10 | 20 | 40 |
|---------------------------------|-------|--|-------------------------|-------------------------|-------|
| Depth of
sieve media
(cm) | 9.175 | 21.13 | 30.48 | 39.66 | 48.83 |
| Increment
of sieve
media | 9.175 | 21.13 - 9.175 =
21.8 = 30.48 -
21.8 = 9.18 | 39.66 -
30.48 = 9.18 | 48.83 -
39.66 = 9.18 | |

$$= 2.54 \times 10 \times 1.49(2)$$

$$= 9.175$$

Provide the sieve media of 50 cm (Foot H)

Determination of total depth of filter

$$= \text{Depth of under drain} + \text{depth of sand} \\ + \text{depth of sand} + \text{depth of water} \\ + \text{Free board}$$

$$= 0.65 + 1.50 + 0.6 + 1.2 + 0.8$$

$$= 3.25 \text{ m}$$

Provide filter of size 600. of size 4-44 m
 $5.9 \text{ m} \times 3.25 \text{ m}$

STOCKS AND ANALYSIS:-

of 21

1) prepare size 0.6 mm and uniformity coefficient 1.5 from a stock sand the sieve size for stock sand given as follows

| sand size
(mm) | cumulative weight
passing in % |
|-------------------|-----------------------------------|
| 0.21 | 3.5 |
| 0.3 | 11 |
| 0.42 | 22 |
| 0.84 | 42 |
| 1.12 | 64 |
| 1.68 | 83 |
| 2.38 | 90 |

Given: — $D_{10} = 0.5 \text{ mm}$

$$C_u = 1.5$$

$$C_u = \frac{D_{60}}{D_{10}}$$

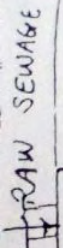
$$D_{10}$$

$$1.5 = \frac{D_{60}}{0.5}$$

$$D_{60} = 0.75$$

511,573

EFFLUENT FOR
DISPOSAL



- Preliminary unit
- Primary unit.
- Secondary unit

2 set

I) PRELIMINARY UNIT:-

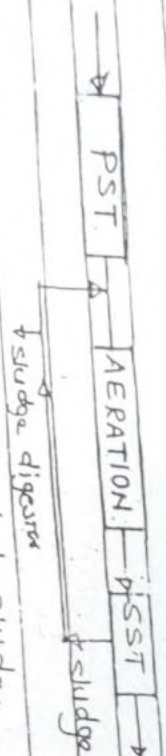
larger size floating particles present in sewage

II) PRIMARY TREATMENT UNIT:-

Explanation
in (Notes-1)

1) ACTIVATED SLUDGE PROCESS:-

It includes following parts
in figure-

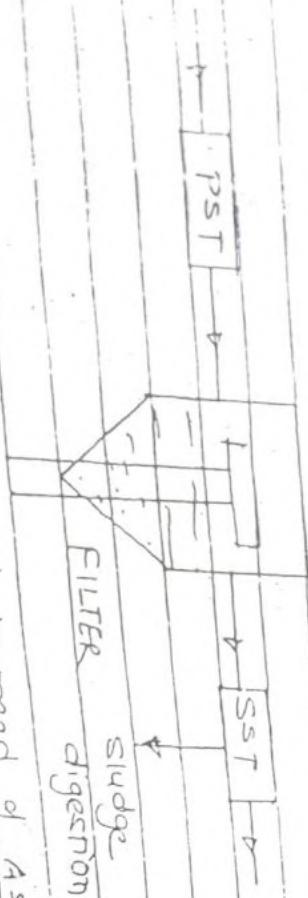


Hence, ASP i.e., activated sludge process consists of aeration and SST unit. Excess sludge goes to the sludge digester.

It is used to remove organic matter (BOD) present in sewage.

A2) TRICKLING FILTER:-

It also removes BOD.



It can be used instead of ASP.

B) SLUDGE DIGESTOR:- It digests the sludge by anaerobic decomposition.

C) SLUDGE DRYING BEDS:- To dry the sludge in presence of sunlight.

IDEAL REQUIREMENTS OF SEWAGE TREATMENT PLANT:-

Following are the ideal requirements of sewage treatment plant:-

1. It should be away from city or sufficient distance in order to avoid the chance of pollution.
2. It should be nearer to the disposal point as far as possible, so that transportation of sewage effluent be minimum.
3. As far as possible, the T.P. should be provided at lower elevation than of elevation of outfall sewers, so no pumping should be required.
4. Sufficient space should be provided at T.P. site for the future expansion if required.
5. Topographical conditions of the site should be such that it should be a water logging area.
6. T.P. should be situated on the downstream side of intake and (used for water supply scheme).
7. Topographical conditions of the ground should be such that it should be away from city or sufficient distance in order to avoid the chance of pollution.

should flow by the action of gravity from unit to unit.

Site should be easily approachable during worst period (season).

Site should be provided with the facilities like electric supply, etc.

SCREEN:- 0 SGT, 508, 0008, 509, 009, 010
A unit which is provided in

The sewage treatment plant in order to remove the big size floating particles is called as screen.

The material which is removed in screen is called as screening.

According to the size, screens are divided into three categories-

- Coarse screen ✓
- Medium screen ✓
- Fine screen ✓

COARSE SCREEN:-

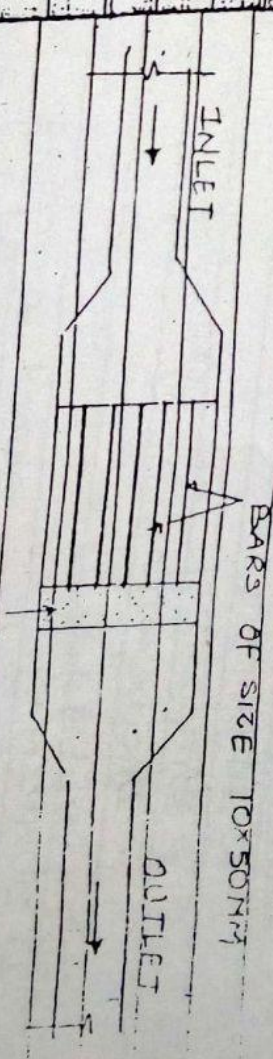
When the size of clear opening provided in the screen ranges from is greater than 50mm, the screen is called as coarse screen.

It is manufactured by using steel pipes or rods. Sometimes it is

called as loads. It is not common in waste water plant. They are provided along the road entrance, drain, etc.

2) MEDIUM SCREEN:-

When the size of clear opening provided in the screen ranges in between 20-50mm, the screen is called as medium screen.



It is manufactured by using steel bars. Sometimes it is also called as bar screen. It is most commonly used in waste water treatment plant.

3) FINE SCREEN:-

When the size of clear opening provided in the screen ranges from 20-30mm, screen is called as fine screen.

It is manufactured by using wires. Hence, this type of screen is called as wire mesh screen. It is not commonly used for domestic waste water treatment. But it is used in industrial waste screening.

DISPOSAL OF SCREENING:-

Following are the usual methods adopted for the disposal of screenings:-

- Dumping
- Composting
- Incineration.

1) DUMPING:-

In this method screenings are buried in the ground. If the screen contains large size of organic particles, it is dumped at lower depth. When it contains large quantity of inorganic particles & it is necessary to raise the ground level above former level, dumping is done.

2) COMPOSTING:-

If the composting material contains more organic matter, they

are allowed to bury into the ground for one year. For this a trench is made in which they are buried. In this span, these material gets under decomposition and converted into manure which can be used for various purposes.

3) INCINERATION:-

If the screening materials sufficient calorific value, then it is allowed to dry in sunlight. They are burnt in kiln under controlled conditions.

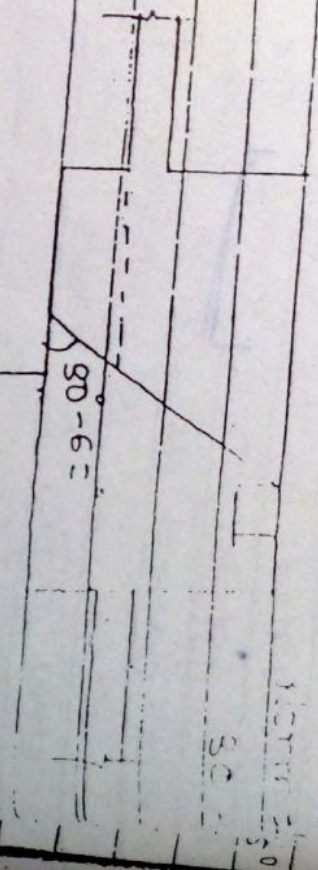
DESIGN CRITERIA FOR BAR SCREEN

1) Velocity of flow through vertical screen:
 $V_f = 0.6 - 1.2 \text{ m/s}$

2) Size of rectangular bars = $10 \times 50 \text{ mm}$ with 10mm side facing the flow.

3) Clear opening = $25 - 50 \text{ mm}$

4) Inclination of bar



width of screen = 1.2-1.5m

DESIGN STEPS FOR BAR-SCREEN:-

1) Calculation of design discharge (QD).

i.e.

$$A_{net} = QD$$

$\frac{1}{1}$

2) To find the total area required on the vertical plane (A_{total})

$$c/c \text{ distance of bar} = \text{clear opening} + \text{area of opening bar}$$

Assume clear opening of screen & size of bar provided. Also mentioned side of bar facing the flow.

$$A_{total} = A_{net} \times c/c \text{ distance of bar}$$

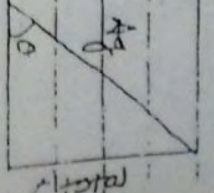
clear opening.



4) To find gross area on inclined plane.

Assume inclination of bar.

$$A_g = \frac{A_{total}}{\sin \theta}$$



5) To find total depth of screen:-

$$\text{water depth} = \frac{A_g}{\text{width of screen}}$$

Assume width of screen, $B \approx 1.2-1.5m$

$$\therefore \text{Total depth} = \text{water depth} + \text{free board}$$

Assume F.B. = 0.2-0.4m

6) NO. of bars (N) :-

$$N = \frac{\text{width of screen}}{c/c \text{ distance of bar}} - 1$$

7) Design sketch

NUMERICAL:-

Design a suitable bar screen for a town having population 2 lakhs. Water supply rate = 180 lpd. Assume suitable data.

→ Given:-

Population = 2 lakhs

Water supply rate, $Q_w = 180$ lpd.

1) Calculation of design discharge:-

$$Q_w = 2 \times 10^5 \times 180$$

$$= \frac{36 \times 10^6 \times 10^{-3}}{24 \times 3600}$$

$$= 0.4167 \text{ m}^3/\text{sec}$$

Assuming $Q_{seepage} = 0.75 \times Q_w$

$$= 0.75 \times 0.4167$$

$$= 0.3125 \text{ m}^3/\text{sec}$$

Assuming design discharge,

$$Q_D \approx 2 \times Q_s$$

$$= 2 \times 0.3125$$

$$= 0.625 \text{ m}^3/\text{sec}$$

2) To determine net opening area required on vertical plane:-

Assuming velocity of flow,

$$V = 0.9 \text{ m/s}$$

$$A_{net} = \frac{Q_D}{V} = \frac{0.625}{0.9}$$

$$= 0.694 \text{ m}^2$$

3) To find total area required on vertical plane:-

Assuming clear opening in screen = 30 mm

Size of bar provided is 10 mm with 10 mm side facing the flow

c/c distance of bars = clear opening + bar

bar

opening + bar

$$= 30 + 10$$

$$= 40 \text{ mm}$$

∴ $A_{total} = A_{net} \times \text{c/c distance}$

clear 30

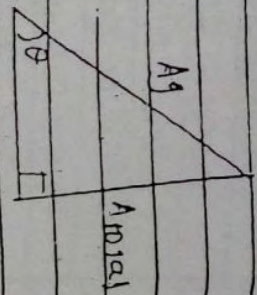
$$= 0.694 \times 40$$

30

$$= 0.925 \text{ m}^2$$

To find gross area on inclined plane:-

Assuming the direction of bar of 45°



$$A_g = \frac{A_{\text{total}}}{\sin \theta}$$

$$= \frac{0.925}{\sin 45}$$

$$= 1.308 \text{ m}^2$$

To find the total depth of screen:-
Assume width of screen = 1.3 m

$$\text{water depth} = A_g \times \text{width of screen}$$

$$= \frac{1.308}{1.3} \approx 1 \text{ m}$$

Assuming free board $\approx 0.3 \text{ m}$

$$\text{Total depth} = 0.3 + 1$$

$$= 1.3 \text{ m}$$

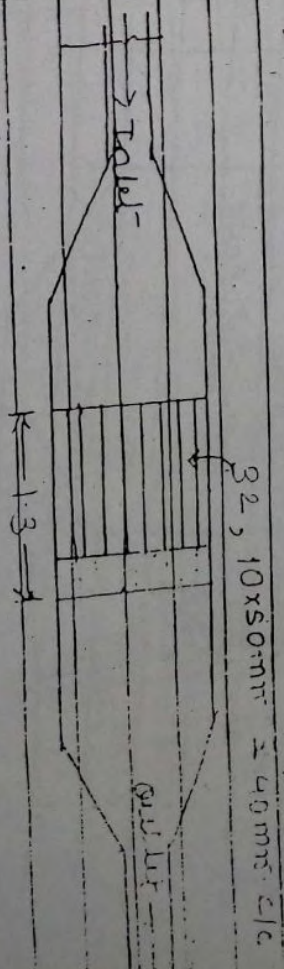
6) No. of bars (N):-

$$N = \frac{\text{width of screen}}{\text{c/c distance of bar}}$$

$$= \frac{1300}{40} - 1$$

$$= 31.5 \approx 32$$

7) Design sketch:-

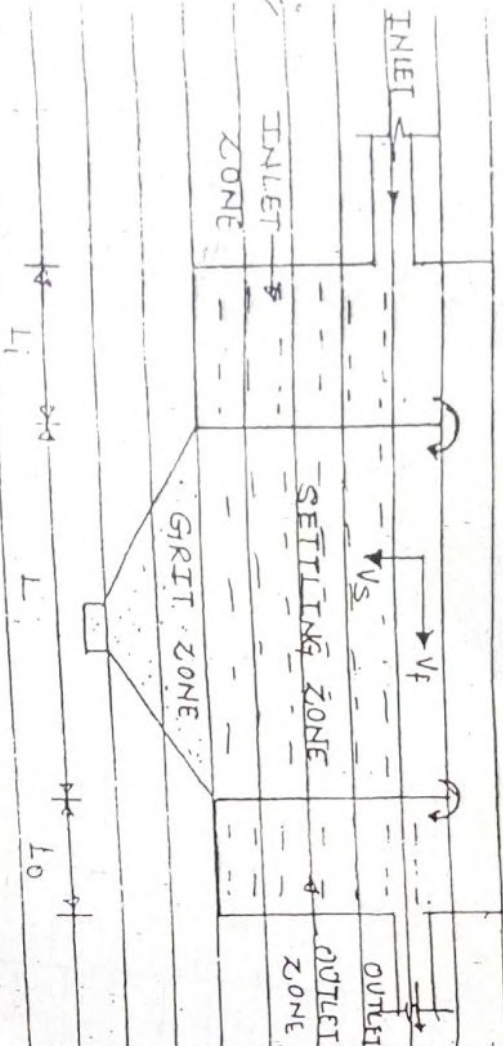


GRIT CHAMBER :-

GRIT :-

The ^{inorganic} particles of size greater than 0.2 mm and specific gravity ≈ 2.65 are called as grit.

The unit provided for removal of grit is called as grit chamber.



The length of grit zone is greater than the width.

Based on the principle of sedimentation, the grit chamber is divided into-

- Inlet zone
- Outlet zone

- Settling zone
- Grit zone

Waste water containing grit is entered through the inlet in the inlet zone. It is then passed into the settling zone where it is allowed to detained for a period of 40-60 sec. During this period all inorganic particles having size greater than 0.2 mm are settled down by the action of gravity.

Grit chamber is provided in order to reduce the load on PST. Hence, it is a preliminary unit and is not necessary to provide.

The effluent free from grit then collected through outlet. Grit is deposited at the bottom and is removed frequently. It can be used in construction material for road etc.

DESIGN CRITERIA FOR GRIT CHAMBER:-

- 1) Detention time (T) = 40-60 sec
- 2) Velocity of flow (V_f) = 0.15 - 0.3 m/sec
 ≈ 0.23 m/sec
- 3) settling velocity (V_s) = $d(8T + 70)$
 d - dia. of grit
 T - Temp. in $^{\circ}\text{C}$
 $V_s \approx 0.02 - 0.03$ m/sec.
- 4) Sewage depth in chamber $\approx 1.2 - 1.8$ m
- 5) Depth of grit accumulation = 25% of sewage depth
- 6) F.B. $\approx 0.2 - 0.4$ m
- 7) Length of inlet zone = Length of the outlet zone $\approx 10 - 20$ % of settling zone length.

NUMERICAL:-

Design a suitable grit chamber for a sewage T.D. having capacity 18 MLD.

$$Q_D = 18 \text{ MLD}$$

$$= \frac{18 \times 10^6 \times 10^{-3}}{24 \times 3600}$$

$$Q_D = 0.21 \text{ m}^3/\text{s}$$

- 1) Assuming detention time, $T = 50$ sec.
 $\therefore$ Vol. of sewage in chamber (V) = $Q \times T$

$$V = 10.5 \text{ m}^3$$

- 2) Assuming velocity of flow,

$$V_f \approx 0.23 \text{ m/s}$$

$$\text{But } V_f = \frac{L}{T}$$

$$\therefore L = V_f \times T = 0.23 \times 50$$

$$L = 11.5 \text{ m}$$

- 3) Assuming size of grit

$$d \approx 0.2 \text{ mm}$$

and sewage temp. = 20°C

settling velocity,

$$V_s = d(3T + 70)$$

$$= 0.2 \times 10^3 (3 \times 20 + 70)$$

$$V_s = 0.026 \text{ m/s}$$

$$\text{Buf } V_s = d$$

$$\therefore d = V_s \times T$$

$$= 0.026 \times 50$$

$$d = 1.3 \text{ m}$$

(1.2-1.5 m) etc

4) width of chamber (B),

$$B = \sqrt[4]{Q}$$

$$L \times d$$

$$= 10.5$$

$$11.5 \times 1.3$$

$$B = 0.7 \text{ m}$$

Overall depth (H):-

$$H = \text{Sewage depth} + 25 \text{ to } 50 + F.B.$$

$$= 1.3 + 0.25 \times 1.3 + 0.25$$

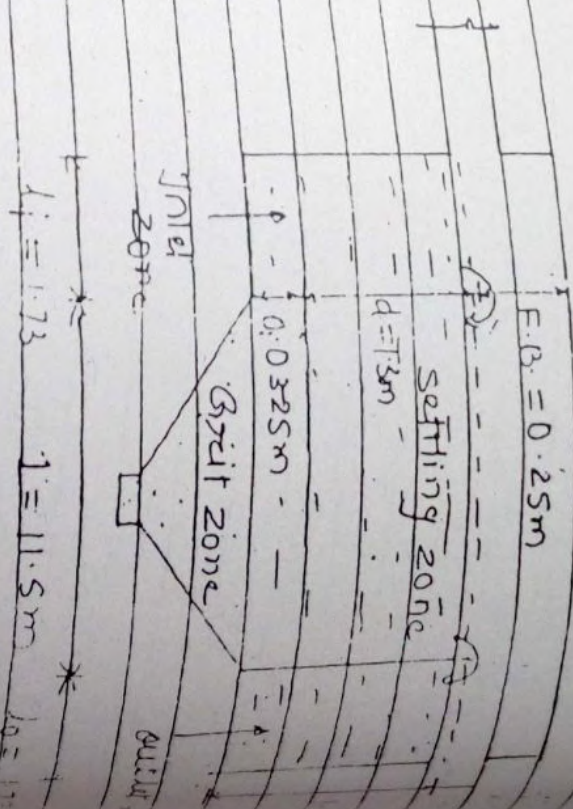
6) Inlet zone length (L_i):-

$$L_i = \text{outlet zone length } (L_o) = 1.5 \text{ m}$$

$$= 0.15 \times 11.5$$

$$= 1.725 \text{ m}$$

7) Design sketch:-



PRIMARY SEDIMENTATION TANK (PST).

It is the only one ~~primary~~ principle of sedimentation.

PRINCIPLE OF SEDIMENTATION:-

The main principle of sedimentation is to allow water to rest or to flow at a very slow velocity so that heavier particles can settle down due to gravity. Particles in water having specific gravity ranges from 1.04-2.65. Those particles having specific gravity more than 1.2 will settle down to the bottom & tank bed.

IDEAL SETTLING ZONE & DERIVATION FOR SOR:-

SOR is defined as the discharge per unit surface area of sedimentation tank.

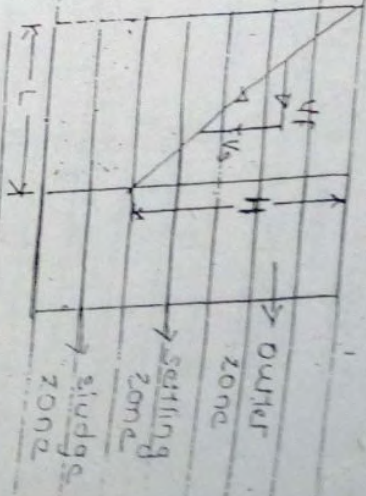
It is nothing but the surface overflow rate or surface loading (SL).

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

See

Q
A

Inlet zone



Consider an ideal settling zone as shown in figure having dimensions $L \times B \times H$. This ideal basin consists of four zones - Inlet zone

- settling zone
- sludge zone
- outlet zone

Let 'Q' be the discharge flowing through the basin, and if 'V_f' is the velocity of flow in the tank then,

$$V_f = \frac{Q}{A} = \frac{Q}{BH} \quad \text{--- (1)}$$

If the particle enters at the top of inlet zone it must be removed at the bottom corner of the sedimentation tank i.e., it flows a diagonal path as shown

in figure.

From the figure, considering the similarity of triangle,

$$\frac{H}{L} = \frac{V_s}{V_f}$$

$$V_s = \frac{H}{L} \times V_f$$

$$V_s = \frac{H}{L} \times Q$$

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

$$\Rightarrow V_s = Q$$

surface area of tank

V_s - design settling velocity (SOR)

Hence, from above derivation it is proved that the efficiency of the sedimentation does not depend upon the depth of basin but depends upon the surface area of basin. Hence, higher the surface area, higher is the efficiency.

* FACTORS AFFECTING SEDIMENTATION :-

○ Sedimentation process is affected by following factors :-

- velocity of flow (V_f)
- detention time
- Temperature & viscosity of sewage
- shape, size & surface area of particles.
- sewage current
- surface area of tank.

1) Velocity of flow (V_f) :-

Velocity of flow of sewage to from inlet to outlet is -

$$V_f = \frac{Q}{A} = \frac{1}{T} \quad (\text{Horizontal velocity})$$

Hence, more the velocity, less will settle down due to lesser time.

2) Detention time (t) :-

Detention time, $t = \frac{\text{Volume}}{\text{Flow rate}}$

Hence, greater the detention time, more efficient will be the process of sedimentation.

Temperature and viscosity of sewage:-

As the temperature of the sewage increases, viscosity becomes less. Hence, velocity of sewage increases and due to this less particle will settle down.

shape, size and specific gravity of the particle:-

Spherical particle settle down more quickly due to large surface area. Particles having high specific gravity will settle down quickly.

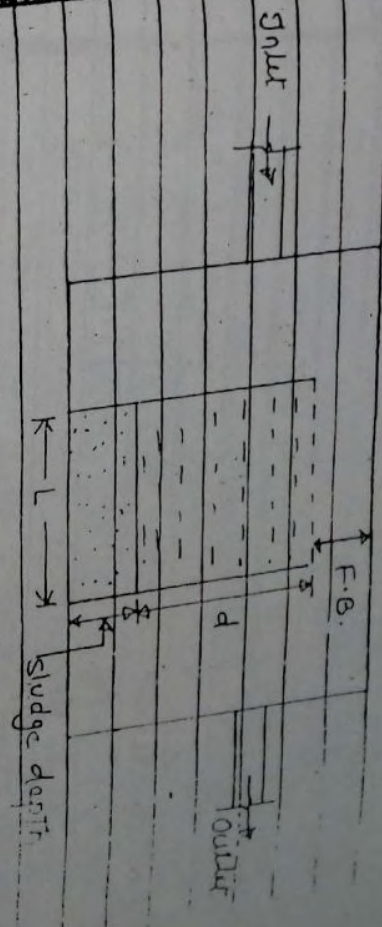
sewage current:-

Higher turbulence reduces the efficiency of sedimentation process due to higher current of sewage.

Surface area of tank:-

It varies directly with the process. Higher the surface area of tank, more efficient will the process.

* DESIGN CRITERIA FOR SEDIMENTATION TANK



1) Detention time (t) = 4-8 hrs. (range)
= 2-4 hrs. (chemicals)

2) Water depth $\approx 2.5 - 4.5$ m

3) L/B ratio $\approx 3-6$ m

4) Max. width $\neq 12$ m

5) SOR, = $0.5 - 0.75 \text{ m}^3/\text{hr}/\text{m}^2$ (plain)
= $1 - 1.5 \text{ m}^3/\text{hr}/\text{m}^2$ (chemicals)

6) velocity of flow = $0.15 - 0.6 \text{ m}/\text{min}$

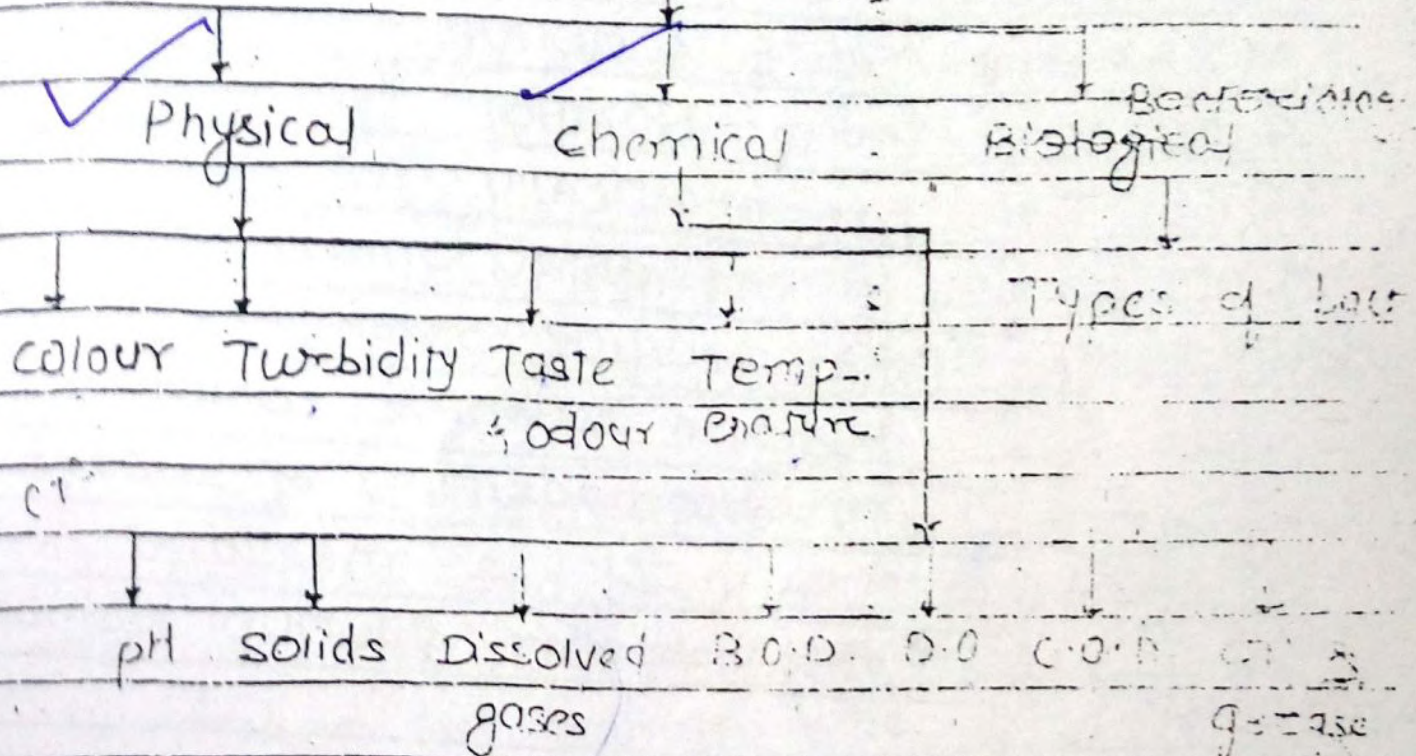
7) F.B. = $0.2 - 0.4$ m

8) sludge depth = 0.25 m water depth

9) Bottom slope = $1:100$

CHARACTERISTICS OF SEWAGE

Characteristics of sewage



PHYSICAL TESTS:-

COLOUR:-

Colour of the sewage gives the information about the strength of sewage

| Colour | Polluted intensity |
|----------------------|--------------------|
| 1. Light gray/yellow | Lightly polluted |
| 2. Dark gray/brown | Medium polluted |
| 3. Blackish | Highly polluted |

2
Other colours of sewage are found formed due to presence of some types of industrial wastes. Also colour of the sewage changes with time. It is measured on platinum cobalt scale in tinctometer.

• TASTE & ODOUR:-

Normal fresh domestic sewage has practically no smell odour and it is not objectionable. But stale sewage has peculiar odour, especially that of hydrogen sulphide which is formed due to decomposition of sewage.

It is measured in terms of threshold or dilution number.

• TEMPERATURE:-

The temperature of normal domestic sewage is slightly greater than that of water. It affects various biological activity, viscosity of sewage.

The temperature of sewage below normal shows the presence of ground water and above normal shows industrial water mixing.

• TURBIDITY:-

Sewage is generally turbid. It is caused by the presence of organic and inorganic matter, organic and inorganic. It can be determined by using 200 or Jackson's turbidimeter. It is generally measured on silica.

2) CHEMICAL PROPERTIES:-

• pH value:-

It is the important characteristic of the sewage. It decides acidic or alkaline nature of sewage.

pH value is defined as reciprocal of hydrogen mathematically.

$$pH = \log \left[\frac{1}{[H^+]} \right]$$

Fresh sewage is good in nature and it becomes passes. The pH value of should be 6.5-9.5 for the bacteria.

DISSOLVED GASES:-

Various unwanted gases are present in sewage. The concentration of these gases depend on the strength of pollution. They are produced due to the decomposition of organic matter in the sewage.

OIL & GREASE:-

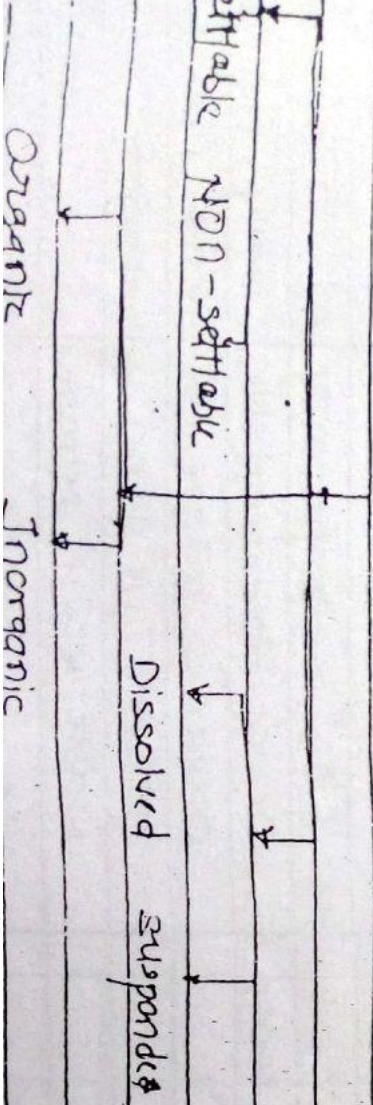
Oil and grease are undesirable in sewage as it tends to fill up the voids of filtering media and it forms objectionable deposits on sewers and other treatment units.

In the industrial waste, concentration of oil and grease is more & in domestic waste it is negligible.

TOTAL SOLIDS:-

They are divided as follows:-

Total solids:-



The organic solids are also called as volatile solids while inorganic solids are termed as non-volatile or fixed solids. Organic particles undergo decomposition. For their decomposition, for determining their proportion, sample of sewage is heated till dry residue is obtained. Its total weight is nothing but total solids. The dry residue is again heated. The loss of weight represents volatile matter while the residue represents fixed solids.

For the determination of settleable and non-settleable solids, the sewage sample is taken in Imhoff cone which is shown in the figure.

The sample is graduated & determined for 30 minutes. In ml

The solids which settled down

are called as settleable solids. Those which remain in dissolved form are called as non-settleable solids.

When the sewage sample is filtered through asbestos paper filter, suspended solids retained on the filter and dissolved solids remained in solution

1. DISSOLVED OXYGEN:-

It is the important characteristics of waste water. Dissolved oxygen indicates the strength of sewage. As the time passes, DO concentration in sewage decreases due to the consumption of oxygen by aerobic bacteria and decomposition process. It may fall up to zero. In fresh sewage, DO concentration is about 4-5 mg/L. But as the time, DO concentration decreases and sewage becomes strong and in this environment aquatic life will not survive.

BOD + COD:-

B.O.D. -

It stands for biological oxygen demand.

It is defined

as the amount of oxygen required by the aerobic bacteria for aerobic decomposition of organic matter under specified temp and time.

Temp. - 20°C

Time - 5 days

C.O.D. -

It stands for chemical oxygen demand.

It is defined

as the amount of oxygen required for the chemical decomposition of organic matter by using certain oxidising agent.

2. It indicates the amount of carbones organic matter present in the given waste water sample.

3. It requires 5 days for result.

4. It does not indicate complete decomposition of organic matter. But it indicates 10-85% decomposition of carbonaceous matter.

8. It indicates the amount of carbonaceous and nitrogenous matter present in the given waste water sample.

8. It requires 3 hours for result.

4. It indicates 100% decomposition of both carbonaceous and nitrogenous matter present in the given waste water sample.

5. BOD test is not useful for determination of nitrogenous matter.

5. COD test is useful for the determination of nitrogenous matter.

6. BOD is useful only when biodegradable organic matter is present in given waste water sample.

6. This test is only for biodegradable and non-biodegradable matter.

1. BOD is cheap method.

2. COD is costlier than B.O.D.

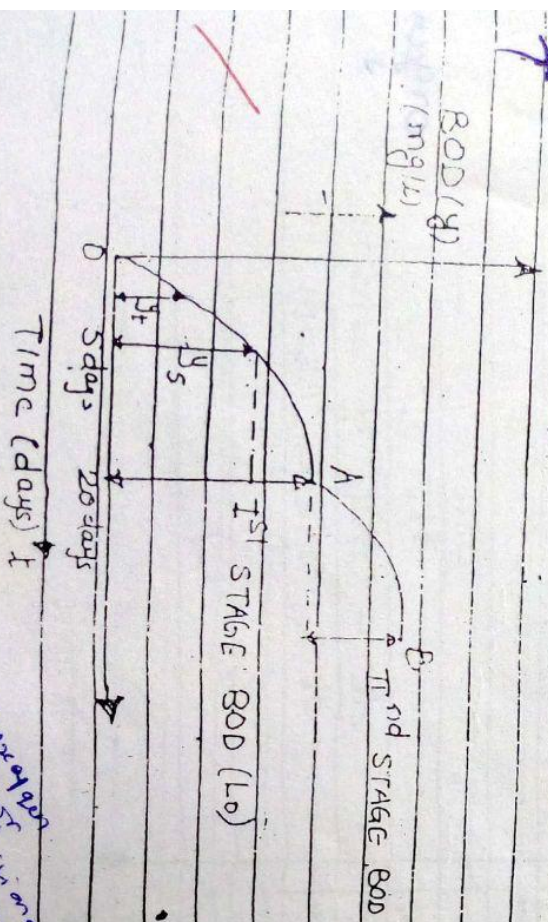
For domestic waste 8 For domestic waste BOD is less than COD is greater than BOD.

3. B.O.D. ~~is not combined~~

DEFINITION:- ~~not combined~~ 50%, 100%, 510

The amount of oxygen required by the bacteria for aerobic decomposition of organic matter under specified temperature at 20°C and time duration of 5 days is called as B.O.D.

STANDARD BOD CURVE:-



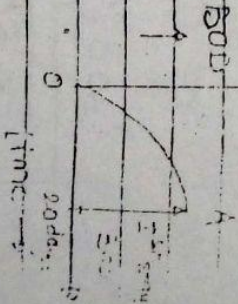
The std. BOD curve is divided into two stages, OA and AB.

OA - 1st stage BOD due to carbonaceous matter. It is also called as an ultimate BOD (Lo).

AB - 2nd stage BOD due to nitrogenous matter.

For the domestic waste water, nitrogenous compounds are negligible. Hence, 2nd stage BOD can be neglected i.e., BOD due to carbonaceous matter is considered only.

If it is observed that after upto 5 days about 75-80% decomposition of organic matter takes place. After 5 days, the rate of decomposition of organic matter decreases and it is continued upto 20 days. In this period, only 15-25% decomposition takes place. Hence, it is not possible practically to wait for 20 days till 20 days. Therefore, only 5 days BOD is considered.



Max BOD (y) = $L_0 \Rightarrow$ due to carbonaceous matter.

BOD remain = $L_0 - y_t$

y_t - BOD at any time.

L_0 - original BOD present initially.

* SIGNIFICANCE OF BOD:-

BOD has following significance.

1. BOD indicates the amount of oxygen required for biological decomposition of organic matter.
2. BOD is useful in order to determine the size of sump of units of treatment plant.
3. BOD is useful for determination of efficiency of sump of units of T.P.
4. By knowing BOD, the waste can be classified accordingly as follows:-

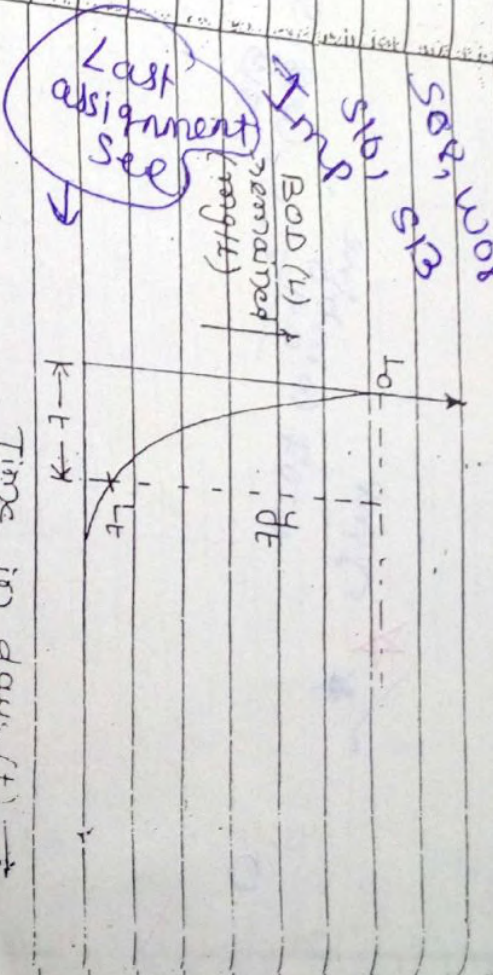
| | |
|------------|--------------------------------|
| 300 (mg/L) | Waste water |
| < 30 | Treated waste water |
| 30-300 | Slightly polluted waste water |
| 300-450 | Mediately polluted waste water |
| > 450 | Highly polluted waste water |

5. BOD gives idea of quantity of clean water required or disposal of effluent by the method of dilution.

* BOD EQUATION:-

Consider a std. BOD curve shown in figure.

IMP:- 7-8



From the graph, the rate change of BOD is given by-

$$\frac{d(L_0 - y)}{dt} = \frac{dL_0}{dt} - \frac{dy}{dt}$$

organic

L - BOD remained at any time t

$$\frac{dL}{dt} = -kL \quad \text{--- (2)}$$

k - BOD rate constant (deoxygenating rate constant)

Negative sign shows that the rate of change of BOD decreases with time.

$$L \frac{dL}{dt} = -k \cdot dt \quad \text{--- (3)}$$

At $t=0$, $L=L_0$

& $t=t$, $L=L_t$

Integrating,

$$\int_{L_0}^{L_t} \frac{dL}{L} = -k \int_0^t dt$$

$$\ln \left(\frac{L_t}{L_0} \right) = -k(t) \quad \checkmark$$

$$\ln \left[\frac{L_t}{L_0} \right] = -kt \quad \checkmark$$

$$\frac{L_t}{L_0} = e^{-k(t)} \quad \checkmark$$

$$L_t = L_0 \cdot e^{-kt} \quad \text{--- (4)}$$

OR

$$L_t = L_0 \cdot 10^{-kt}$$

(k is to the base 10)

Again from graph,

$$y_t = L_0 - L_t \quad \checkmark$$

$$\therefore y_t = L_0 - L_0 e^{-kt}$$

$$y_t = L_0 (1 - e^{-kt}) \quad \text{--- (5)} \quad \checkmark$$

OR, This is called as BOD eqn

$$y_t = L_0 (1 - 10^{-kt}) \quad \checkmark$$

L_0 - ultimate BOD

t - time in days

● EFFECT OF TEMPERATURE ON BOD RATE CONSTANT (K) :-

As the temperature increases, bacterial activity also increases. Hence, BOD rate constant (K) also increases with time.

I) If $T = 20^\circ\text{C}$,

$$K = 0.23 \text{ day}^{-1} \text{ (to the base 'e')}$$

$$K = 0.1 \text{ day}^{-1} \text{ (to the base '10')}$$

II) If $T > 20^\circ\text{C}$,

$$K(T) = K(20) (1.054)^{T-20}$$

III) If $T < 20^\circ\text{C}$,

$$K(T) = K(20) (1.135)^{T-20}$$

● NUMERICALS :-

1) 5 days BOD at 20°C for a given water sample is 180 mg/L. Find -
 - 3 days BOD at 25°C
 - 8 days BOD at 15°C
 Assume, $K = 0.23/\text{day}$.

Given :-

$$y_5 \text{ at } 20^\circ\text{C} = 180 \text{ mg/L}$$

$$K \text{ at } 20^\circ\text{C} = 0.23/\text{day}$$

Solution :-

I) Ultimate BOD (L_0) :-

$$y_t = L_0 (1 - e^{-Kt})$$

$$y_5 = L_0 (1 - e^{-5K})$$

$$180 = L_0 (1 - e^{-0.23 \times 5})$$

$$\Rightarrow L_0 = 263.6 \text{ mg/L} \quad \checkmark$$

II) y_3 at 25°C :-

$$K_{25} = K_{20} (1.054)^{25-20}$$

$$= 0.23 \times (1.054)^5$$

$$= 0.3/\text{day}$$

18 at 18°C :-

$$K_{15} = K_{20} (1.135)^{15-20}$$

$$= 9.12$$

$$y_3 \text{ at } 25^\circ\text{C} = 10(1 - e^{-kt})$$
$$= 263.4(1 - e^{-0.3 \times 3})$$
$$= 156 \text{ mg/L}$$

1) $48^\circ \text{ at } 15^\circ \text{C} :-$
 $1.000 (1.135) 15-20$

$$K_{15} = K_{20} (1.135)^{15-20}$$

$$= 0.12 \text{ /day}$$

$$= 0.12 \text{ /day}$$

$$= 263.4 (1 - e^{-(0.12 \times 8)})$$

For a given waste water sample, 5 day BOD at 17°C is 200 mg/L. Find -

Assume $k = 0.07/\text{day}$ at 17°C .

Guiden :-

$$k = 0.07 / \text{day} \quad \text{at } 17^\circ\text{C}$$

~~$\frac{1}{2} \text{ of } 17^\circ\text{C} = 200 \text{ mg/L}$~~

Solution :-

I) ultimate BOD:-

$$y_T = L_0(1 - 10^{-kt})$$

$$200 = L_0 (1 - 10^{-0.07 \times 5})$$

$$7,161.46 \text{ mg/L} = 0.716146\%$$

II) Y_f at $30^\circ\text{C} :-$

$$K_T = K_{20} (1.08^{\frac{T-20}{10}})$$

$$0.07 = 1620 (1.085)^{17-20}$$

~~$\therefore k_{20} = 0.102 / \text{day}$~~

$$K_{30} = 0. K_{20} \times (1.054)^{T-20}$$

$$= 0.102 \times (1.054)^{80-20}$$

$$K_{30} = 0.173/\text{day}$$

$$Y_f \text{ at } 30^\circ \Rightarrow$$

$$y_t = \ln(1 - 10^{-kt})$$

$$= 361.46 (1 - 10^{-0.173 \times 57})$$

$$= 339.2 \text{ mg/L}$$