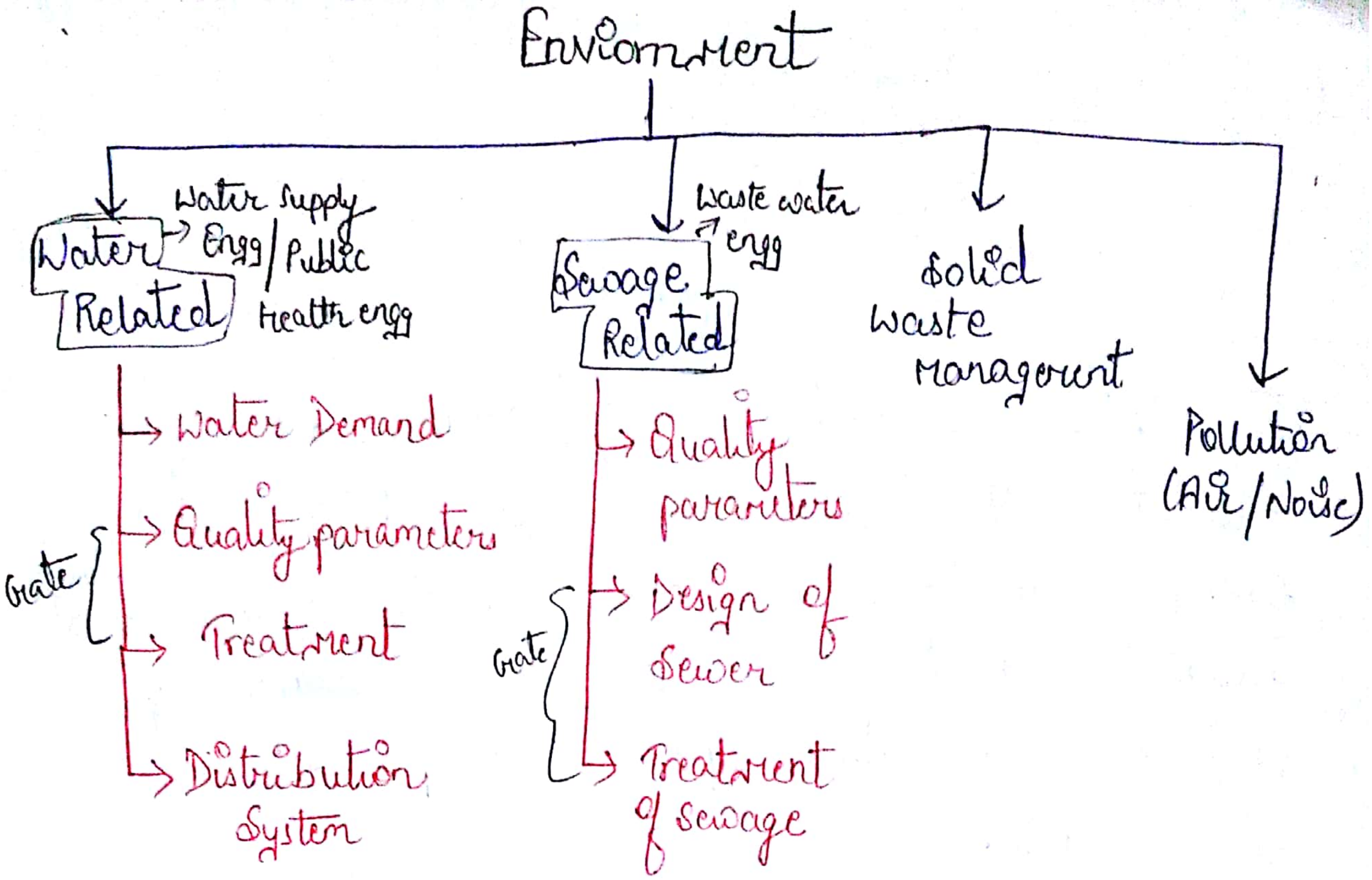
ENVIORNMENT

ENGG.

NITISH SIR

(Mayank Singh)



Enviornment

- Water Related → Water supply Engg / Public Health engg
  - Water Demand
  - Quality parameters
  - Treatment
  - Distribution System
  - Gate: Quality parameters, Treatment
- Sewage Related → Waste water engg
  - Quality parameters
  - Design of Sewer
  - Treatment of sewage
  - Gate: Design of Sewer, Treatment of sewage
- Solid waste management
- Pollution (Air/Noise)

Gate- 10-12 marks

ESE - 30 marks obj

SSE JE - 8-10 marks

Water Supply Engineering

Ch-1 Water Demand, its source ~~of~~ & conveyance

To design a water supply scheme, we first estimate the population for which the scheme should be designed. The scheme ~~first~~ once installed must cater for the demand of projected population upto some pre-determined future date.

Design Period

The future period or no. of years for which a provision is made in designing the capacities of various components of water supply scheme is called design period. The design period should neither too long nor to short.

It should not exceed the useful life of component structure. The design period is recommended by Government of India manuals on water supply are-

Imp

UNITS	Design Period
1.) Water treatment units	15years
2.) Pipe connections to several treatment units (Pipe mains)	30years
3.) Service reservoir (पानी की टंकी Colony में)	15years
4.) Distribution system	30 years

Population Forecasting

Water Demand is assessed on the basis of future population. There are 3 main factors responsible for change in future population—

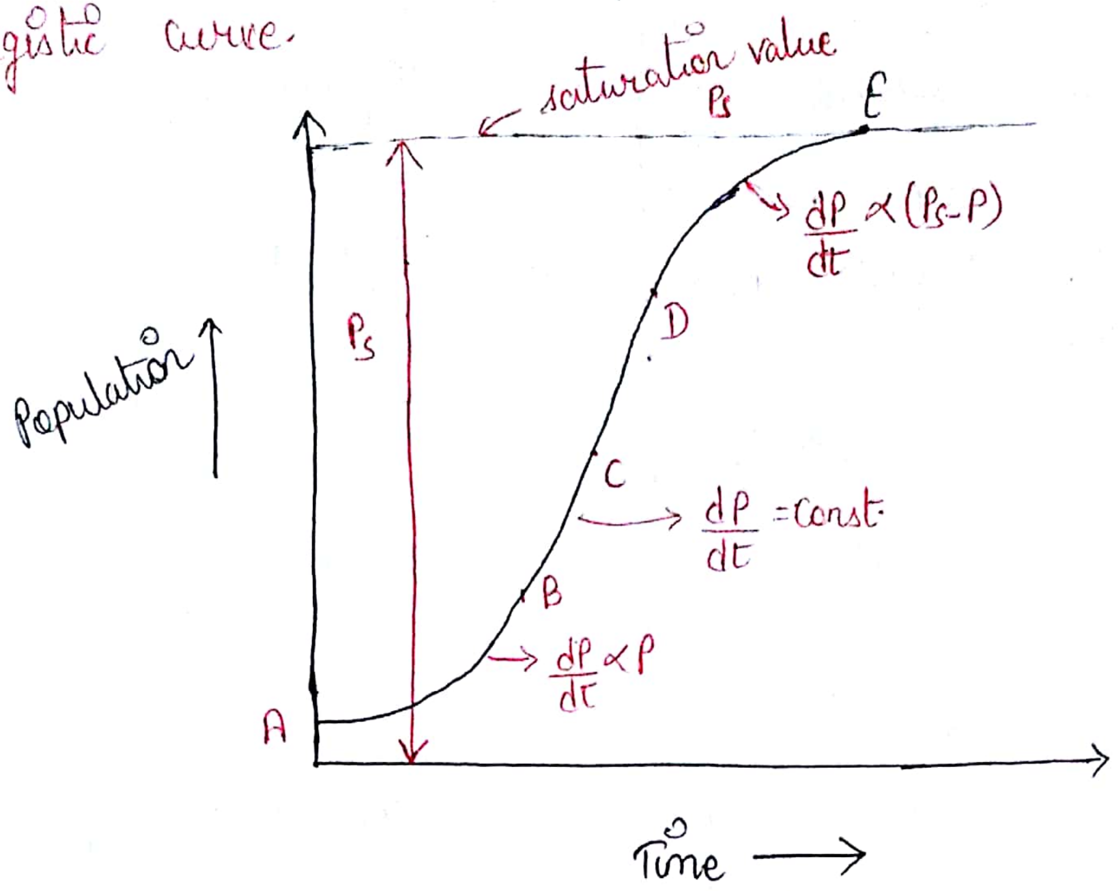
(i) Birth
(ii) Death
(iii) Migration

Population forecasting is done by mathematical formulas & by graphical representation by the previous population record.

Growth curve

The population would probably follow the growth curve, characterstics of living things within limited space & limited economic opportunities.

The curve is S-shape & is known as logistic curve.



→ The curve represents early growth AB at an increasing rate & late growth DE at a decreasing rate (as the saturation value approached)

→ The transitional middle curve BD follows an arithmetic increase (const).

Population Forecasting Methods

1) Arithmetic increase method

$$\boxed{P_n = P_0 + n\bar{x}}$$

P_n = prospective population after 'n' decade

P_0 = present population

n = no. of decades b/w now & future

$\bar{x}$ = (arithmetic mean) ave of population increase in known decades

Avg of

Q The population of 5 decades from 1930 to 1970 are given in table find out population after 1 & 6 decades beyond the last known decade by using arithmetic increase method.

Year	Population	Increase
1930	25,000	3000
1940	28,000	6000
1950	34,000	8000
1960	42,000	5000
1970	47,000	

$$\bar{x} = \frac{3000 + 6000 + 8000 + 5000}{4} = 5500$$

Sol^n

$$P_n = P_0 + n\bar{x}$$

$$P_{1980} = 47000 + 1 \times 5500$$

$$P_{1980} = 52{,}500$$

$$P_{2030} = P_0 + n\bar{x}$$

$$= 47000 + 6 \times 5500$$

$$P_{2030} = 80{,}000$$

2.) Geometric increase method

$$\boxed{P_n = P_0\left(1 + \frac{r}{100}\right)^n}$$

r = Geometric growth rate

$$r = (r_1 \times r_2 \times r_3 \times \dots r_n)^{1/n}$$

Eg

Pop	Inc	r
X_1	x_1	$\frac{x_1}{X_1}$
X_2	x_2	$\frac{x_2}{X_2}$
X_3	x_3	$\frac{x_3}{X_3}$
X_4		

In this M/d percentage increase or percentage growth rate is assumed const. & the increase is compounded over the existing population every decade.

This M/d is also known as uniform increase M/d. This m/d is suitable for young cities.

Q

Year	1951	1961	1971	1981	— —	2011
Population in thousand	93	111	132	161	——	?

Sol^n^

Year	Population	Increase	~~r/1000~~ $r = \frac{x_1}{x_1} \times 100$
1951	93000		
1961	111000	18,000	19.35 %.
1971	132000	21,000	18.91 %.
1981	161000	29,000	21.98 %.

$r = 20.03\%$

$r = \{ 19.35 \times 18.91 \times 21.98)^{1/3}$

$$P_n = P_0 \left(1 + \frac{r}{100} \right)^n$$

$$P_{2011} = 161000 \left(1 + \frac{20.03}{100} \right)^3$$

$$\boxed{P_{2011} = 2{,}78{,}416}$$

Q Calc. the population of the year 2000 & 2006 for a city whose population in year 1930 was ~~₹~~ 25000 & in year 1970 was 47000 make use of Geometric increase method.

Solⁿ

Yr ~~Pop~~	Pop	Inc	r
1930	25000	22000	$r_{40} = 88\%$
1970	47000		
2000	?		
2006	?		

r for 40yrs = 88%

r for 10 yrs $= \frac{88}{4} = 22\%$

r for 1yrs $= \frac{22}{10\text{yrs}} = 2.2\%$

$$P_{2000} = 47000 \left(1 + \frac{\boxed{22}}{100}\right)^{\boxed{3}} = 85.34K$$

यहाँ Decade lenge

$$P_{2006} = 47000 \left(1 + \frac{\boxed{2.2}}{100}\right)^{\boxed{36}} = 9\cancel{7.2}\,K$$

ये भी साल में लेगें

यहाँ साल लेगें Decade possible नहीं है

3) Incremental Increase method

$$P_n = P_0 + n\bar{x} + \frac{n(n+1)}{2}\bar{y}$$

$\bar{x}$ = avg increase of population of known decades

$\bar{y}$ = avg of incremental increase.

Q

Year	Population	Increase $\bar{x}$	~~x~~y
1940	23798624		
		23179701	9[illegible]%.
1950	46978325		
			−15371589
1960	54786437	7808112	
1970	63467823	8681386	873274
1980	69077421	5609598	−3071788
2010	?		

avg ← $\bar{x}$ = 11319699.25

avg ← $\bar{y}$ = −5856701

$$P_{2010} = 69077421 + 3 \times 11319699.25 + \frac{3(3+1)}{2} \times (-5856701)$$

$$P_{2010} = 67896312.75$$

4.) Decreasing rate of Growth

This m/d is applicable only when ∫ rate of growth shows downward trend.

Q

Yr	Population	Increase	% increase	% decrease
1960	55,500			
		8200	14.77	
1970	63700			2.84
		7600	11.93	
1980	71300			0.43
		8200	11.50	
1990	79500			
			r = 12.65	r' = $\frac{3.27}{2}$ = 1.63

$$P_{2000} = P_{1990} + \left(\frac{r-r'}{100}\right) \times P_{1990}$$

अगार r

$$P_{2010} = P_{2000} + \left(\frac{r-2r'}{100}\right) \times P_{2000}$$

$$P_{2020} = P_{2010} + \left(\frac{r-3r'}{100}\right) \times P_{2010}$$

$$P_{2000} = 79500 + \left(\frac{12.73-1.63}{100}\right) \times 79500 = 88324.5$$

$$P_{2010} = 88324 + \left(\frac{12.73-2\times1.63}{100}\right) 88324 = 96688$$

$$P_{2020} = 96688 + \left(\frac{12.73-3\times1.63}{100}\right) 9668 = 104269$$

Water Demand

After knowing the population of a particular area, it is mandatory to estimate per capita demand to design a water supply scheme.

Various types of Demand are-

1) **Domestic water demand**

This includes water require in private buildings for cooking, drinking, bathing, gardening purpose etc & may vary according to the living condition. The total domestic water consumption takes into account about 50-60% of total quantity of water supply.

As per IS code the Domestic water consumption limits b/w $\boxed{135-225}$ lpcd (litre per capita per day)

Under ordinary conditions water requirement must be 200 lpcd, but it can be reduced to ~~130s~~ 135 lpcd for LIG (low income group), depending upon the prevailing condition.

2) Institutional & Commercial Water Demand

(i) School | College - 45-135 lpcd
(ii) Offices - 45 lpcd
(iii) Restaurents - 70 lpcd
(iv) Cinema & theatre - 15 lpcd
(v) Hotels - 180 lpcd
(vi) Hospitals - 340 lpcd (when beds are less than 100)
450 lpcd (when beds are more than 100)

3) Industrial Water demand

Name of industry	Unit of production	Approximate quantity of water required for per unit production
1) Automobiles	Vehicle	40
2) Fertilizers	Tonne	80-200
3) leather	Tonne	40
4) Paper	Tonne	200-400
5) Petroleum Refinary	Tonne (Crude)	1-2
6) Sugar	Tonne	1-2

4) Demand for Public Use

This includes water demand for parks, gardening, roads etc. A nominal amount not exceeding 5% of total consumption may be provided for this.

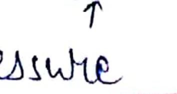
5) Fire demand

The quantity of water required for fire is not very large but for a total amount of water consumption for a city of 50 lacs population hardly amount to ~~be~~ per lpcd.

But this water should be easily available & kept stored always in reservoir.

Following requirement must be met for this demand-

a) 3 Jet streams are simultaneously ~~from~~ thrown from each hydrant one on ~~a~~ burning property & one each ~~&~~ ends of adjcent property.
The discharge of each stream should be about
$\boxed{1100 L/min}$



b) The min. water pressure available at fire hydrants should be $\boxed{100 - 150 kN/m^2}$ (10-15m of water head) & should be maintain even after 4-5 hrs of constant use of hydrants

Some other formula's are also used-

(i) Kuchling's Formula

$$\boxed{Q = 3182\sqrt{P}}\ l/min$$

P = Population in thousand

(ii) National board of fire under writer's formula.

$$\boxed{Q = 4637\sqrt{P}\left(1 - 0.01\sqrt{P}\right)}\ l/min$$

P in thousand

Q Compute the fire demand for a city of 2lacs population by writer's & Kuchling formula.

Soln:

P = 2lacks

Per thousand = ~~20,000~~ 200

$Q = 3182\sqrt{200} = 45000\ l/min = \boxed{64.8\ mld}$

$Q = 4637\sqrt{200}\left(1 - 0.01\sqrt{200}\right)$

$Q = 56303\ l/min \Rightarrow \frac{56303 \times 24 \times 60}{10^6}$

= ~~81.07 million/day~~

= $\boxed{81.07\ mld\ (million\ litre/day)}$

Water Demand for losses & theft

This includes loss in leakage & other ~~loss~~ losses & wastage. This amount may be 15% of total demand.

Per Capita Demand (q)

$$\text{Per capita demand} = \frac{\text{Total yearly water requirement for city in litres}}{365 \times \text{Population}}$$

Factor affecting per capita demand

1) **Size of city** - Demand increases with the size of city.

On an avg the per capita demand for Indian towns may vary with population.

Observations

Population	Per capita demand (lpcd)
<20,000	110
20 - 50K	110 - 150
50 - 200 K	150 - 240
2lac - 5lac	240 - 275
5lac - 10lac	275 - 335
>10lac	335 - 360

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Imp **Note** For Indian condition IS code permits max. value of $\boxed{335 \text{ lpcd}}$

2) Climatic Conditions

3) Type of Country & habit of People

4) Industrial & commercial activities

5) Quality of water

6) Development of sewage facilities

7) Cost of water & method of charging

8) Pressure in distribution system

9) System of supply

- → continuous - हर दिन
- → Intermittent - एक दिन एक छोड़ के

The water supply may be continuous or intermittent. In intermittent supply water consumption will be less.

Imp
Variation in demands

1) Maximum daily consumption = 1.8 × avg daily consumption (q)

Avg daily consumption $= \frac{\text{Total}}{365 \times P}$ → $q = \boxed{1.8q}$

2) Maximum hourly consumption - 1.5 × avg hourly consumption of maximum day

$$= 1.5 \times \frac{1.8q}{24}$$

$$\boxed{= \frac{2.7q}{24}}$$

3) Maximum weekly demand = 1.48 × avg weekly demand

4) Maximum monthly demand = 1.28 × avg monthly demand.

Peak Factor

Per capital rate of water supply indicates only the avg consumption of water per day over a period of 1 year, However this consumption vary with season, month, days & hour.

The fluctuation in consumption is accounted for by considering the peak factors. Following peak factors are recommended–

Imp

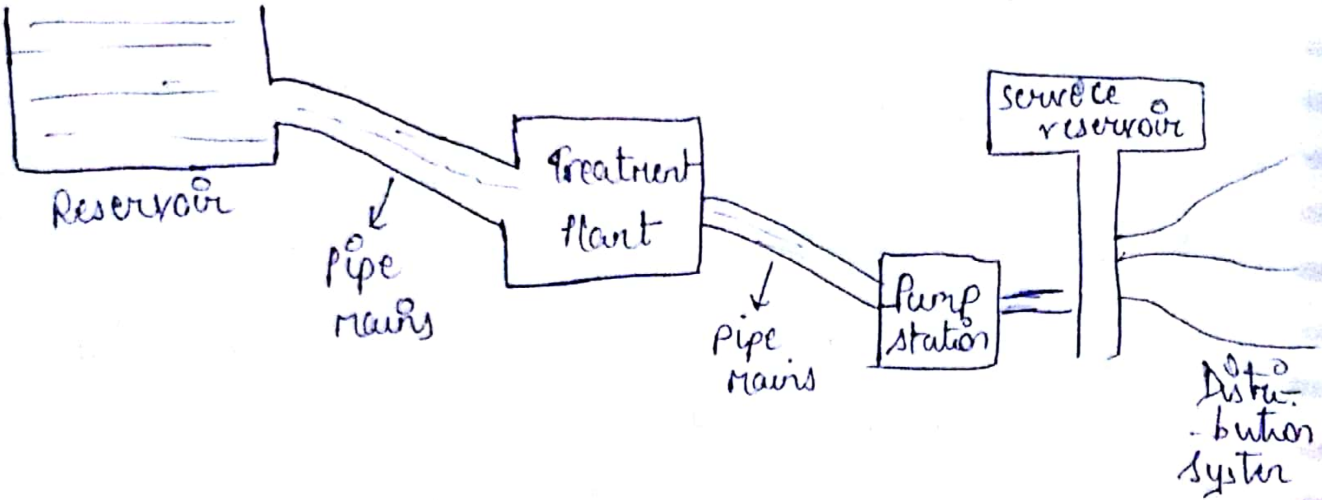
Populations	Peak Factor
(i) up to 50,000	3
(ii) 50,000 - 2,00,000	2.5
(iii) > 2,00,000	2
(iv) For small water supply schemes	3

Coincident Draft

It is sum of maximum daily demand & fire demand.

$$\boxed{\text{Coincident Draft} = 1.8q + \text{Fire demand}}$$

Design capacity of various components of water supply scheme



1) The source of supply may be designed for max daily consumption.

2) The pipe mains filters an other treatment units are designed for ~~coincident draft~~ max daily draft or max. daily demand.

3) The pumps may be designed for max. daily draft + some additional reserve for break down

4) The distribution system should be designed for max. ~~hour~~ hourly demand ~~of~~ for max. day or coincident draft whichever is more.

5.) The service reservoir is design to take care of hourly fluctuations, fire demand & emergency reserve.

Note <u>Pipe Mains</u> – These are used to take water from reservoir to service reservoir & <u>distribution system</u> – is used to take water from service reservoir to house-hold.

most imp

Various Sources of water for supply

1.) Surface water source – Reservoir
2.) Ground water source – Tubewell, well

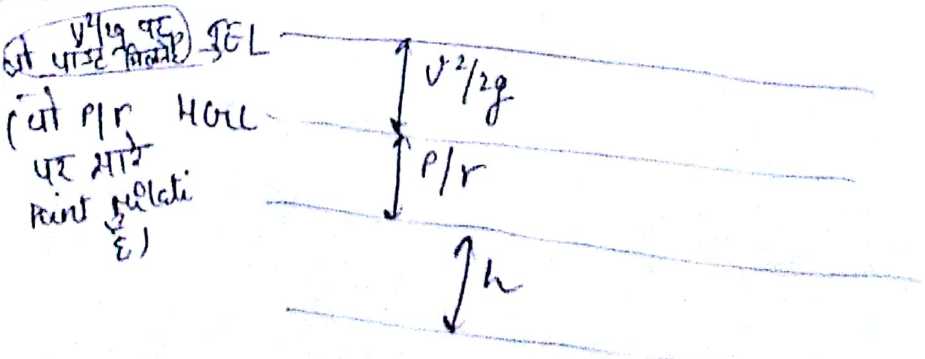
Conduits for water supply

Depending upon the condition & characteristics of flow the conduits may be divided into –

1.) Gravity conduits
2.) Pressure conduits

1.) Gravity Conduits

Water flows under the action of gravity. HGL will coincide with water surface & parallel to bed.



In such a flow water is all along at atmospheric pressure.

Gravity conduits can be in the following form – 1) Canal

2) Flumes → open channel supported above ground surface. They are used to ~~contain~~ convey water across vally & small depression.

3) Aquiduct → These are closed rectangular or circular section built up of masonary & RCC.

2) Pressure Conduits

→ Water flows under the pressure above atmospheric pressure. It can be used for rising gradient above Hydraulic gradient line.

→ The measure head loss caused by friction, can be given by following formulaes

a) Darcy Weisbach Formula
b) Manning's Formula
c) Hazen William's formula

Various types of Pressure Pipes

1) Metallic Pipe
2) Non-metallic Pipe

1) Metallic Pipe

a) Cast Iron - Most widely used in water supply for pipe ~~zones~~ mains & distribution mains.

b) Steel Pipe - Used as water mains, over bridge & culverts. life is taken around 40yrs under normal conditions.

Note → Steel pipes are having the coats of cement, mortar are known as Hume steel Pipes.

c) Wrought Iron Pipe.

d) Copper, lead brass pipe.

2) Non-metallic Pipes

a) PSC Pipes - They can be made to ~~resistant~~ withstand higher pressure, cheaper than other pipes above 300mm diameter.

→ These are corrosion resistant.

b) Reinforced concrete Pipes - Mostly used for water mains. These are available in diameter from 200mm - 1800mm.

- longitudinal Rft - 0.25% of Gross area

These are classified in 3 types as per IS code.

Class as per IS 458 : 1971	Total pressure
Class P_1	2 Kg/cm^2
Class P_2	4 Kg/cm^2
Class P_3	6 Kg/cm^2

c) Asbestos Pipe --> Silica & Cement are converted under pressure into a stiff material & called ~~as~~ asbestos.

-> These are not suitable for sulphate soils.

-> Due to expansion ~~of~~ & contraction of black cotton soil these pipes are generally avoided.

-> Highly resistant to corrosion

d) Plastic Pipes -

(i) UPVC Pipes - Unplatisized Poly vinyl chloride pipes

- These are used in water distribution system available in size from 15mm - 150mm (dia)
- They are join using socket or rubber ring or solvent.
- They do not have deterimental effect on composition of water passing through them.

→ The main advantage of UPVC pipe is that it is corrosion resistant.

→ lightness & resistant to wide range of chemicals fungi, bacterias are its main advantage.

→ They are not used under heavy water pressure

(ii) Polyethylene Pipes

→ It is a thermoplastic material which soften with heat.

→ They are light in ~~wt~~ weight & flexible resistant to abrasion & corrosion & have better impact resistance.

→ Two types of these pipes are ~~avalaible~~ available

(i) low density polyethylene Pipe (LDPE)

(ii) High density polyethylene Pipe (HDPE)

(iii) GRP (Glass Reinforced Pipes)

These are resistant to corrosion, light in weight, tough, rigid, low thermal conductivity

Forces acting on water Conduits

The structural design of Pressure pipes should be carried out to withstand the pipe under following forces —

1) Internal pressure of water including water Hammer → To be resisted by material stray in tension

2) Pressure due to external load due to backfill, traffic etc. → To be resisted by material strong in compression.

3) Longitudinal stresses, created due to unbalanced pressure at bends or at the point of change of cross-section → To be resisted by holding pipes in massive blocks of concrete or stone masonry.

Water Hammer

→ If the velocity of water flowing in pipe suddenly diminished. The energy given by the water ~~will~~ is divided b/w compressing the water itself & stretching the wall of ~~pi~~ pipe.

→ This pressure rise or water hammer generates a series of stokes sounding like hammer ~~blocks~~ blows which may have sufficient magnitude for rupture the pipe or damage of connected equipments.

→ It may be caused by instantaneous closure of valves.

→ The pressure wave due water hammer travels to back to upstream to the inlet ~~&~~ ~~oo~~ end of pipe where it reverse back to downstream & getting weaker at every reversal.

→ The excess pressure due to water hammer is additional to hydrostatic pressure & depends upon elastic properties of fluid & pipe magnitude of the change in velocity.

Computation of water hammer

Maximum water hammer pressure (which occurs at critical time T_c or time less than T_c)

$$\boxed{H_{max} = \frac{CV_0}{g}} \quad \text{If } T \leq T_c$$

H_{max} = max press. ~~due~~ rised in closed conduit (m)

C = velocity of pressure wave travel (m/s)

V_0 = Normal velocity in pipe line before sudden closure.

g = acc^n due to gravity

If the actual time of closure T is greater than T_c then

$$\boxed{H'_{max} = H_{max} \times \frac{T_c}{T}} \quad \text{If } T > T_c$$

If the pressure wave travel length l

$$\boxed{T_c = \frac{2L}{C}}$$

$$\boxed{C = \sqrt{\frac{E_w}{\rho_w}} \times \frac{1}{\sqrt{\left(1 + \frac{E_w \cdot d}{E \times t}\right)}}}$$

E_w = Bulk modulus of water = $\boxed{2.07 \times 10^8 \ Kg/m^2}$

E = modulus of elasticity of material (Kg/m^2)

ρ_w = density of water

t = wall thickness of pipe (m), d = dia. of pipe (m)

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Note Generally $\sqrt{\frac{E_w}{S_w}} \approx \boxed{1433 m/s}$

↳ velocity of pressure wave or a sound in water

Surge tank is provided in water pipe line to safeguards against water hammer.

Q A 0·5m diameter & 100m long pipeline carrying $0{\cdot}5 m^3/s$ discharge is fitted with a valve at downstream end calculate the rise of Pressure caused within the pipe due to the valve closure in (i) 1 sec
(ii) instantly

Neglecting the term $\alpha = \frac{1}{\sqrt{1+\frac{E_w \cdot d}{E_t}}}$ Take sonic velocity 1433m/s

Sol^n (ii) $Q = 0{\cdot}5 m^3/s$

$A = 0{\cdot}1962 m^2$

$V = 2{\cdot}54 m/s$

$$H_{max} = \frac{1430 \times 2{\cdot}54}{9{\cdot}81} = 370{\cdot}25 m$$

Water Pressure = ~~3650·55~~ 3632·2 KN/m²
370·25 × 9·81

(i) $$T_c = \frac{2L}{C} = \frac{2 \times 100}{1433} = 0{\cdot}13 sec$$

$$H'_{max} = H_{max} \times \frac{T_c}{T}$$

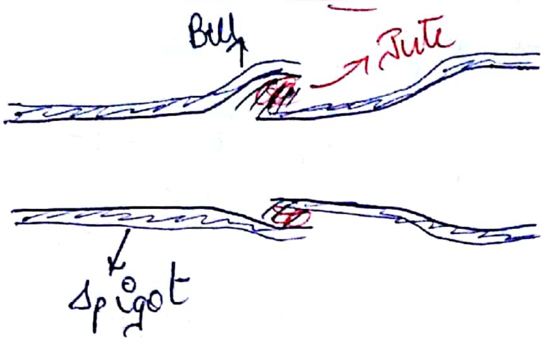
$$= 372.16 \times \frac{\cancel{1}}{\cancel{0.13}}\,0.13$$

$$= 51.54$$

water hammer pressure = 51.54 = 509.53 = 9 kN/m

Joints in water supply scheme

1) Spigot or socket Joint



→ The joint is also called bell & spigot joint

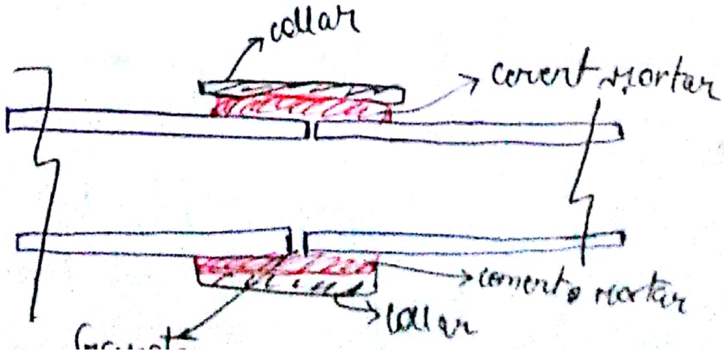
→ The plane end of pipe is called spigot end & expanded end is called Bell end.

→ The yarn (धागा) of jute is bound ~~aro~~ around the spigot & a rubber gascate is placed tightly over the jute.

→ The spigot now inserted into the ~~well &~~ bell end in such a way that it is properly in position.

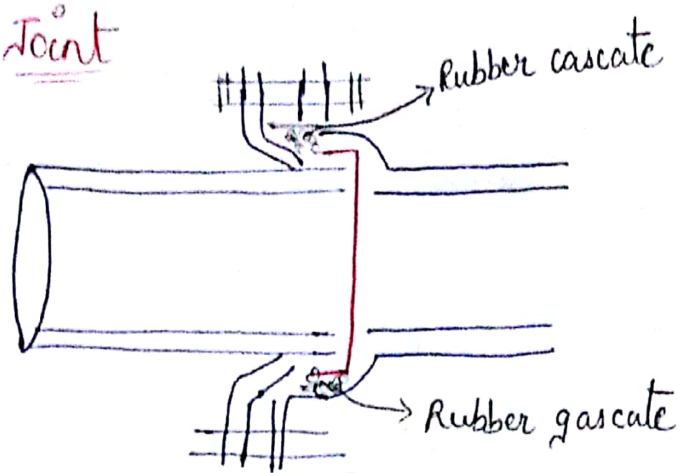
→ Mostly mains & submains of cast iron & steel pipes are joint with this pipe.

2) Collar Joint



Collar joint is recommended for joining RCC pipes or asbestos pipe.

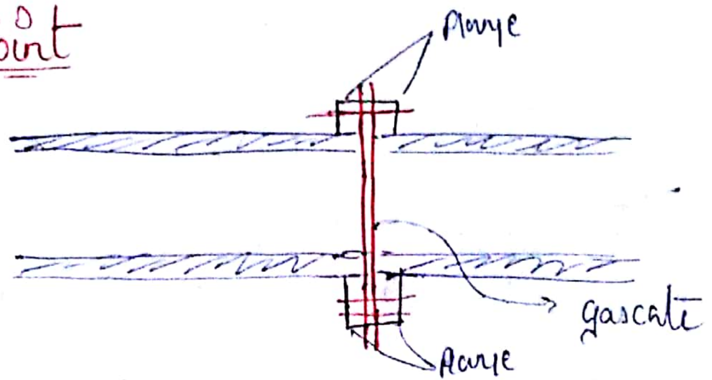
3) Expansion Joint



expansion joint

Expansion joints are provided in metal pipes at suitable intervals to take into account the change in pipe length due to temperature variation.

4) Flanged Joint

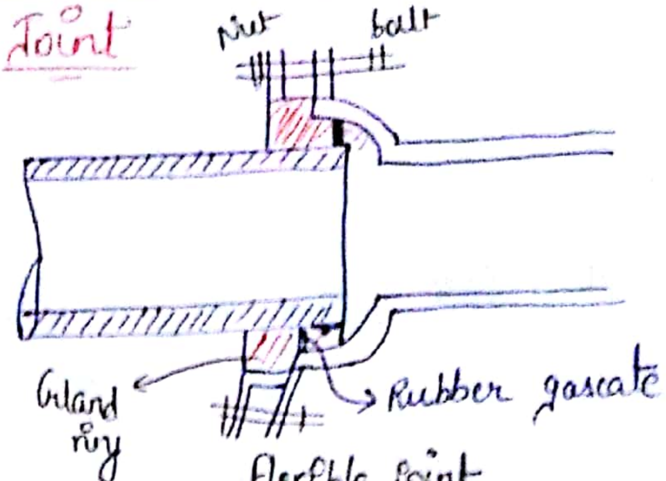


-> This type of joint is recommended for temporary works where the pipe line may be dismantled after work or may be shifted.

-> Mostly used for Cast Iron pipes & steel pipes, not used where deflection & vibration are expected.

-> Two flange are put together & a rubber gascate is kept b/w flanges & fixed by nut & bolts.

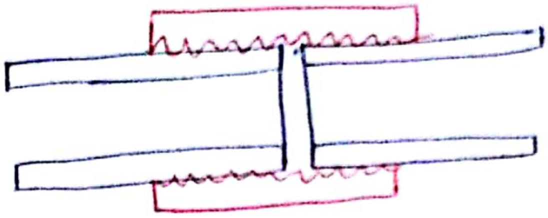
5) Flexible Joint



Flexible joint

Flexible joints are used in pipes where there may be chances of settlement.

6.) Threaded Joints



This pipe joint is recommended for connecting the Galvanised Iron joint.

Corrosion

When water flows through a metal pipes suggest cast iron & steel pipes, it attacks & disintegrate the surface of pipe. Thus metal can dissolve & rusted thereby producing life & carrying capacity of pipe. This phenomena is called as corrosion. It also imparts colour & odour ~~of~~ to the flowing water.

Corrosion Control

Imp

1) Quality of Water

→ By increasing (alkalinity) pH, corrosion can be reduced

→ Reducing CO_2

→ By adding Hexametaphosphate

2) Protective coatings

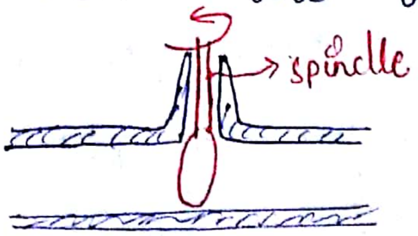
→ Paints, Galvanizing (zinc coating), cement lining etc.

Pipe Appur~~ant~~tenances

1) Sluice valve

→ These are also known as gate valves or shutt off valves

→ These are provided to regulate flow of water through the pipe & are essentials to divide the main line into several sections.



→ These valves are usually provided at summit of pressure conduits at location of low pressure

→ They are made up of iron with brass mounting

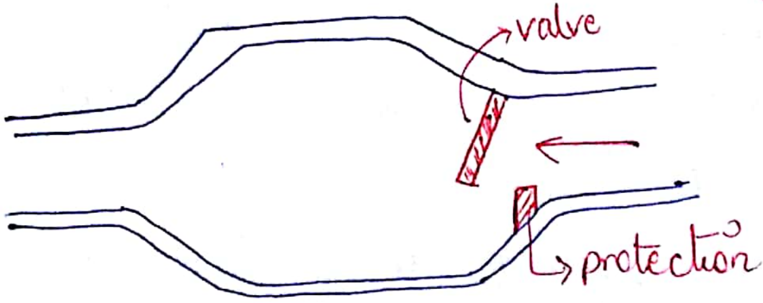
→ They are either slide wedge & ~~dis~~ double disk type

2) Air valves

→ Air valves are also known as air releif valves

→ The water flowing through the pipe line always carrying some amount of air with it, this air tends to accumulate at the summit of pipe line due to the accumulation of air a backward pressure is created which causes blockage to flow, so air relief is provided to releif the air pressure.

3) Check Valve / Reflux valve / Non-returning valve



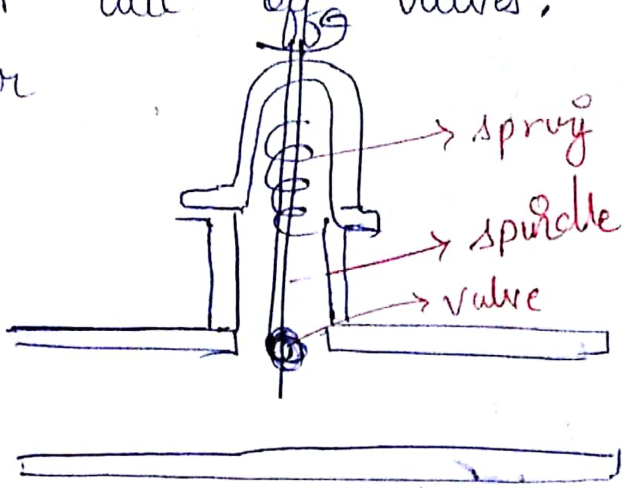
→ These are some automatic devices which allow the water to flow in one direction only. These are made up of brass or Gun metal. This valve is provided in the pipe line which draws water from the pump.

When the pump is operated, the valve is open but ~~the~~ when the pump is suddenly stop, the ~~is~~ valve is automatically closed & water is prevented to revert back to the pump.

4) Relief valve / Safety valve

→ These are also Known as pressure releif valve or cutt off valves.

~~The power~~



→ The power of the spring of the valve is so adjusted that the valve also remains in close condition upto some permissible water pressure in the pipe line.

→ When the pressure of water suddenly exceed the permissible value due to water hammer etc. then the valve is opened automatically & pressure is released, thus it prevents the pipe from busting.

5.) Scour valve

→ Scour valve is also Known as wash out valve

→ These are similar to sluice valve but function is different. These are provided at dead end of pipe line. The function of this valve is to remove sand, silt etc from pipe line.

6) Foot valve

Foot valve is used at the end of the suction pipe. The prevent entry of debris into pumping system.

V. Imp for obj.

Ch-2 Quality Parameters of water

Water Impurities are classified as physical, chemical & Biological impurities.

1) Physical Water Quality Parameters

a) Suspended Solids
b) Turbidity
c) Colour
d) Taste & odour
e) Temperature

a) Suspended Solids

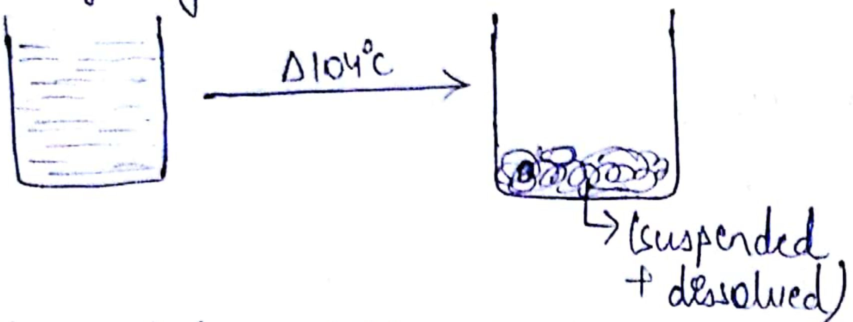
Source → Suspended solids come from inorganic particles like silt clay etc., imiscible liquids like oil & grease. Organic particles from fibres of plants etc; these are non biodegradable solid

Note Suspended solids are physical parameters where as dissolved solids are chemical parameters.

Objection → • These are objectionable because these are asthetically displeasing (दिखने में अच्छा नहीं)
- It provides adsorption site for chemical & biological agents.
- They may biologically active hence can cause disease.

Note Problem of suspended solids comes only in surface water not in ground water.

Measurement → ~~Mo~~ Suspended solids are calculated by weighing them. Total solids (suspended & dissolved) are calculated by evaporating the water at about 104°c & by weighing the residue.



Suspended solids is obtained by filteration.

$$\boxed{\text{Dissolved solids} = \text{Total solids} - \text{Suspended solids}}$$

The organic components of both (DS + SS) can be determined by firing residue at about 600°c

Organic fraction $\xrightarrow{600^\circ \Delta}$ CO_2 + water + some gases

Remaining solids are inorganic solids or fixed solids.

Permissible limit – As per EPA (Enviornmental protection agency) For ~~permis~~ suspended solids perm limit = 30mg/l

permissible limit = 500mg/l or ~~ppa~~ ppm=10^{-6}

- Cause for rejection = 2000 mg/l

b) Turbidity

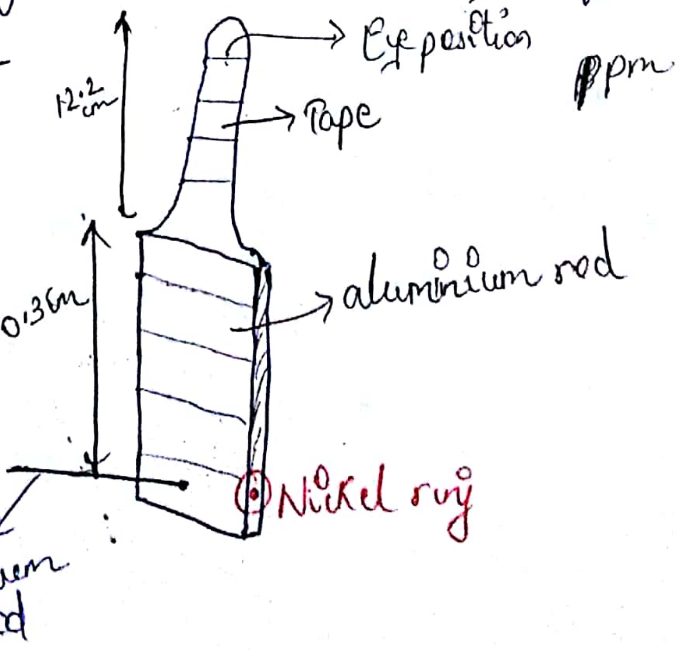
Turbidity is the measure of extent to which light is either scatter or absorbed by suspended materials in water.

Note → It is not the direct quantity ~~of~~ measure of suspended solids.

Objection →
- In natural bodies turbidity interferes the light penetration resulting in difficulty in photosynthesis.
- ~~The~~ disfection of turbid water is difficult.

Measurement → Measurement of Turbidity is done by – Turbidity Rod, Jackson turbidimeter, Bayli's turbidimeter, Nephleometer.

Turbidity Road –



ppm

⇒ Turbidity rod is used to measure the turbidity of water in feild. It consist of a graduated aluminium rod of about 20.3 cm length at, the upper end of rod attatched a graduated non-stretchable tape of about 12.2 cm length.

⇒ At the lower end of ~~plate~~ aluminium rod a screw containing platinum needle (of 1mm diameter & 2.5cm length) & a Nickel ring is inserted.

⇒ The graduated tape has a mark at its top & specifying the position of eye during testing.

⇒ In order to find turbidity the lower end of rod is immersed in water gradually.

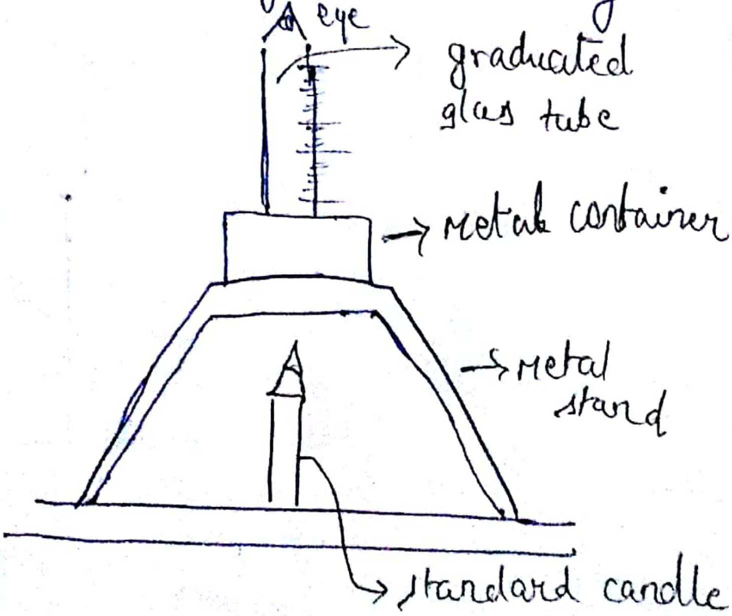
⇒ Eye is kept constantly at the mark position.

⇒ The platinum needle is immersed in water till the platinum needle just disappears.

⇒ The reading of graduated tape gives turbidity directly in ppm.

Jackson turbidimeter

1 JTU = 1ppm



⇒ It is a Laboratory apprattus which is used to measure turbidity of water it works only when turbidity is more than 25 ppm

⇒ It consist of a metal stand holding a metal container & a graduated glass tube in it.

⇒ A std. candle is placed below the stand, the water sample is taken in glass tube & the image of flame of candle seems through the turbid water, the level of water in glass tube is gradually increased the image of flame is invisible.

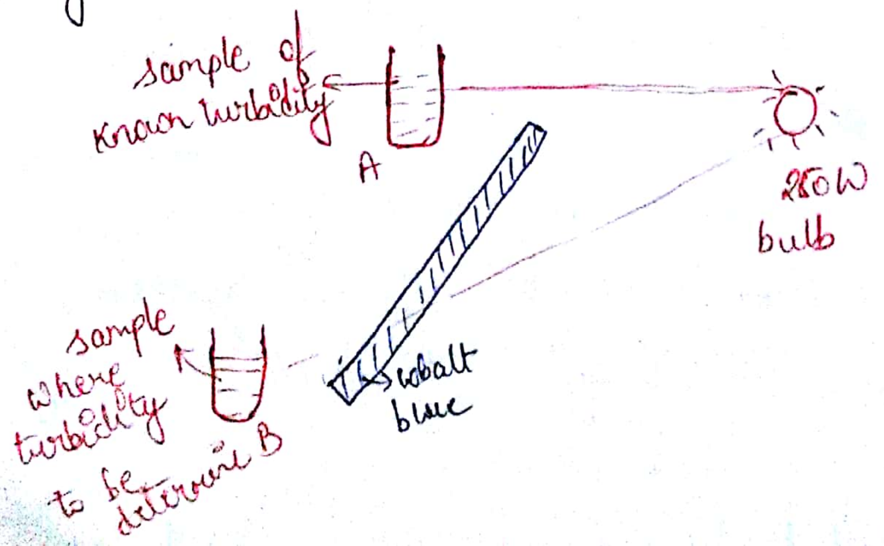
⇒ The height of a less water column measured in the graduated glass provides measure of turbidity longer the light path lesser will the turbidity.

Note A light path of 21.5 cm corresponds to 100 JTU (Jackson turbidity Unit) & about 8.6 cm light paths corresponts to 200 JTU 1 JTU = 1 ppm

Not suitable for Drinking water purpose 5-10 ppm

~~Bay~~

Bayli's turbidimeter & Nephelometer (based on colour matching technique)



⇒ It is a very accurate instrument which can measure ~~the~~ turbidity of less than 5 units.

⇒ It consist of galvanised iron box in which two glass tubes are kept at one end & on the other end a 250W bulb is kept with reflector

· 1 tube contains std. sol^n of known turbidity & other tube contain sample whose turbidity is to be determine.

⇒ The tubes are surrounded all sides (4 sides) by blue cobalt plate & at its bottom white glass plate bcoz of cobalt plate blue light is cast in both the tubes & the colour comparison is made to gave turbidity.

⇒ Photometer produces current after incident of light

Note To improve efficiency intensity of ~~color~~ current is noted instead of ~~inten~~ intensity of light.(color)

Imp

⇒ In Baylis turbidimeter intensity of light is measured in the dir^n of incident of light where as in Nephelometer intensity of light is measured in dir^n of right ∠s to the dir^n of intensity of light.

Note If formazine is used in place of SiO_2 is used, the ~~sometimes~~ turbidity is sometimes called formazine turbidity Unit (FTU) vimp

Note Baylis Turbidimeter is based on adsorption principle & Nephlometer is based on scattering principle.

The permissible limit is 5-10 mm
→ cause for rejectio

Government of India manuals give turbidity in NTU (Nephlometer turbidity unit)

acceptable limit = 1 NTU
cause for rejection = 10 NTU

Note We cannot see turbidity less than 50ppm by Naked eye

C) colour

Source - colour is caused by suspended & dissolved solids in water.

Humic acid - Yellowish brown colour
Iron oxide - Reddish colour
Heavy growth of algae - Greenish colour

Note → After suspended matter causing color are remove, then the color obtained is called True colour.

Objection → Colored water is not suitable for drinking purpose, organic compound causing colour may exert chlorine demands hence reduces the effectiveness of disinfactant

⇒ Phenolic compounds with chlorine produces taste & odour.

⇒ Some colour causing organic compounds with chlorine become carcinogenic (cancer causing substances)

Measurement ⇒ Measurement of colour is done by ~~the~~ colour matching technique (Tintometer)

⇒ Result is expressed in TCU (True colour unit)

⇒ $\boxed{1 \text{ TCU} = 1 \text{ mg/l}}$ of platinum in chloroplatinum ion (it is yellowish brown colour)

⇒ colour testing is done within 72 hour because if time period is more physical & biological characteristic may change.

Acceptable limit = 5 TCU

Cause for rejection = 25 TCU

Imp Note: for determination of colour of industrial waste spectro-photometric technique is used

d.) Taste & odour

Source - These are caused by dissolved gases like H_2S, methane, organic matter from certain living or dead microorganisms, decomposting organic matter, industrial liquid water containing phenolic compounds, ammonia, agricultural chemicals, high residual chlorine & chloro-phenols.

Objection - Taste & odour causing compounds may be carcinogenic.

Measurement

⇒ Odour is measured by an instrument

Osmoscope

⇒ Intensity of taste & odour is ~~generally~~ measured by TON (Threshold's ~~odour~~ odour Number)

It represents the dillution ratio

Acceptable limit = 1-3 TON

Note ⇒ TON testing is done in cold water because increase in temperature can change taste & odour.

Imp ⇒ odour can be removed by - mechanical aeration, oxidation by chemicals like chlorine & its compounds or ozone & adsorption of colour by activated carbon etc.

e.) Temperature

Temperature affects the chemical & biological reactions & increase in 10°c doubles the biological reaction, hence for water supply temperature of water kept at 10-25°c & more than 25°c is objectionable.

Chemical Properties of Water

1.) Total dissolved Solids
2.) Alkanity ✓ Imp
3.) Hardness = Imp
4.) Chroride content
5.) Nitrogen content - SSE, SE, AE
6.) Phosphorus
7.) Flourides
8.) Metals
9.) pH = Imp

Create Imp for Num.

Theoritical questn. ←

1.) Total dissolved Solids

The material remaining in the water after filteration is called dissolved solids.

A direct measurement of TDS ~~can be~~ is made by evaporising the sample of water (after filtration) the residual is ~~made~~ weighed & represented as TDS.

Approximate analysis of total dissolved solids is made by determining electrical conductance of water.

$$\boxed{\text{Electrical conductivity in } (\mu\text{mho/cm}) \text{ at } 25^\circ C \times 0.65 = \text{Dissolved solid in mg/l}}$$

Electrical conductivity is measured by Dionic Water Tester

Sources of total dissolved solids are —

Major sources → Na, Ca, Mg, HCO_3^-, SO_4^{2-}, Cl^-

Minor sources → Fe, K, CO_3^{2-}, NO_3^-, Flouride, boron silica etc

According to GOI manuals —

Acceptable limit of TDS — 500 mg/l

& cause for rejection — 2000 mg/l

Note → The ability of water to conduct electricity is called specific conductance

⇒ specific conductance & concentration of dissolved solids are not related directly one on one basis. Only ionised substances contribute specific conductance

2.) Alkalinity

Alkanity is defined as quantity of ions in water that will react to neutralised hydrogen ion (H^+)

Alkanity is thus a measure of ability of water to neutralized acidity.

Most common constituents of alkanity of water are CO_3^{2-}, HCO_3^-, OH^-.

Alkanity caused by CO_3^{2-} is called carbonate alkanity, caused by HCO_3^- is bi-carbonate alkanity & by OH^- is called caustic alkanity

Alkanity of water comes due to minerals or it may come due to CO_2 mixed with water or due to microbial decomposition of organic matter.

$$CO_2 + H_2O \rightleftharpoons H_2CO_3$$

$$H_2CO_3 \rightleftharpoons H^+ + \boxed{HCO_3^-}$$

$$HCO_3^- \rightleftharpoons H^+ + \boxed{CO_3^{2-}}$$

$$CO_3^{2-} + H_2O \rightleftharpoons \boxed{HCO_3^-} + \boxed{OH^-}$$

Alkanity imparts bitter in taste.

Measurement

Alkanity measurement are done by titrating a water with an acid & determining the hydrogen equivalent of alkalinity & it is expressed in terms of mg/l of $CaCO_3$.

If 0.02N H_2SO_4 is used in titration then 1ml of acid will neutralize 1mg of alkanity as $CaCO_3$.

1ml acid of 0.02N will have $\frac{0.02}{1000}$ gm equivalent of H_2SO_4

for Note

0.02N H_2SO_4 contains 0.02 gm equivalent/l of H_2SO_4

1gram equivalent of everythin will react 1g equivalent of other thing

$$\boxed{\frac{0.02}{1000}\ \text{g. eq. of } H_2SO_4 = \frac{0.02}{1000}\ \text{g equivalent of } CaCO_3}$$

$$\boxed{\text{Equivalent weight} = \frac{\text{molecular weight}}{\text{valency}}}$$

$$\boxed{\text{Gram equivalent} = \frac{\text{weight in gm}}{\text{equivalent weight}}}$$

molecular ~~eq.~~ wt $CaCO_3 = 100 = 40+12+48$ & eq. wt $CaCO_3 = 50$

Q A water contains 210g of CO_3^{2-}, 122g of HCO_3^- & 68g of OH^- what will be the total alkanity in the form of $CaCO_3$.

Soln

Total alkanity in form of $CaCO_3$ = Gram equivalent × equivalent wt of $CaCO_3$

$$\text{equivalent wt of } CO_3^{2-} = \frac{12+48}{2} = 30$$

$$\text{equivalent wt of } HCO_3^- = \frac{1+12+48}{1} = 61$$

$$\text{equivalent wt of } OH^- = \frac{16}{1} = 17$$

$$\text{Gram equivalent of } CO_3^{2-} = \frac{210}{30} = 7$$

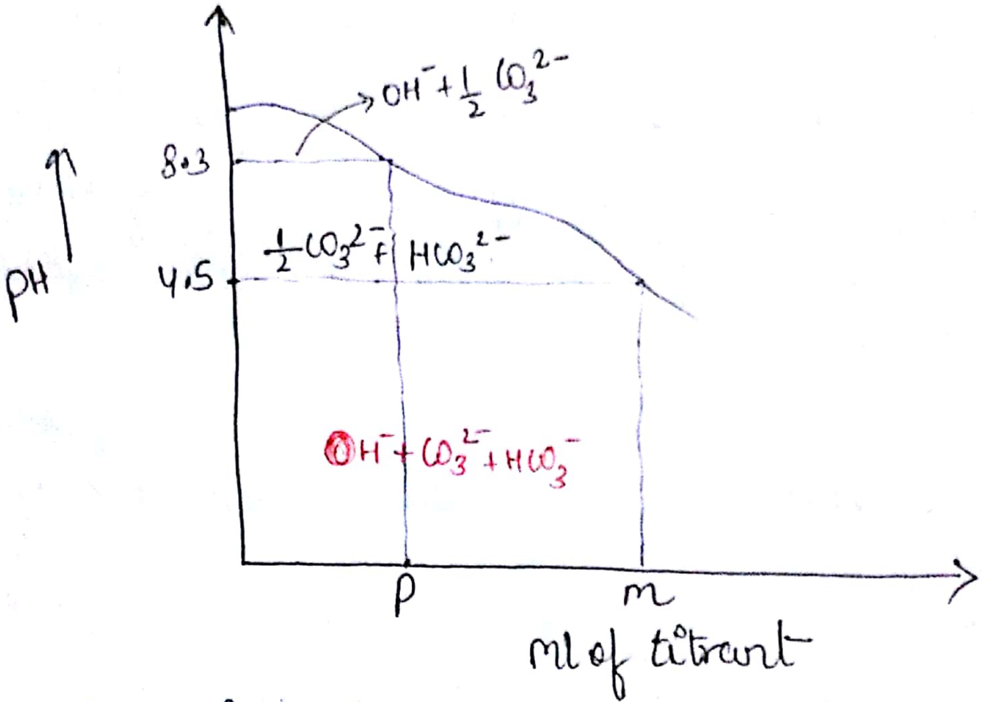
$$\text{〃 〃 〃 } HCO_3 = \cancel{\frac{61}{122}}\ \frac{122}{61} = 2$$

$$\text{〃 〃 } OH^- = \frac{68}{17} = 4$$

Total alkanity = (7+2+4) × 50

= 650 g/L { eq wt of $CaCO_3$ = 50 }

Relative quantity of alkalinity species are pH dependent.



⇒ During titration measurement of pH is done at every stage & titration curve is plotted

⇒ ~~conversion of CO_3~~ Titration -

$$H^+ + OH^- \rightarrow H_2O$$

$$CO_3^{2-} + H^+ \rightarrow HCO_3^-$$

$$HCO_3^- + H \rightarrow (H_2CO_3)$$ carbonic acid

Neutralised

⇒ Conversion of carbonate (CO_3^{2-}) to bicarbonate (HCO_3^-) is essentially complete at pH = 8.3. But resultant bicarbonate also requires acid hence $\frac{1}{2}$ of CO_3^{2-} is thought to be neutralize at pH of 8.3

⇒ Neutralization of OH^- is complete upto pH = 8.3
~~So~~ Hence $\left(OH^- + \frac{1}{2} CO_3^{2-}\right)$ is Neutralized at pH = 8.3

⇒ At pH value of 4.5 all bicarbonates (HCO_3^-) are converted into carbonic acid (H_2CO_3) i.e neutral.
Hence the amount of acid required to titrate sample of water at pH = 4.5 is equivalent to total alkanity of water.

P = phenopthlene alkalinity
m = methyl orange alkalinity
(m = represent total alkalinity)

⇒ phenophthene is color indicator which changes color pink to colourless

If ($P = m$) m = total alkanity
- All alkanity will caustic alkanity

If ($P = m/2$) or $m = 2P$
- All alkanity is carbonate alkanity

If ($P < m/2$)
- Predominant alkanity are carbonate & bicarbonate

carbonate alkanity = 2P
Bicarbonate " = m − 2P

If $(P > M/2)$

- Predominant species are Carbonate & hydroxide.

9) pH

$$\boxed{pH = -\log_{10}[H^+]}$$

where H^+ = moles/liter

It is measured by potentiometer. in which potential exerted by H^+ ions is measured.

permissible limit — 7 to 8.5

pH < 6.5 & > 9.2 is cause for rejection

⇒ It can be measured by color indicators color formed with compared with standard color.

⇒ Indicator used are methyl orange its original color is red & color produces is yellow pH range is $\boxed{2.8 \text{ \& } 4.4.}$

⇒ ~~Phep~~. Phenophthlene red as pH range is $\boxed{8.6 \text{ to } 10.3}$ & its original color is colorless & color produces red.

⇒ Acidic water cause corrosion.

Q A pH of water admitted into treatment plant was 6 in the morning consequent to inflow of raw order from a different source it changed to 8 in next 24hrs assuming linear variation of time of Hydrogen concentration. The time mean pH value of water after 24hrs is –

Solⁿ

Initial $pH_i = 6$

$H^+ = 10^{-6}$ mole/ l

Final $pH_f = 8$

$H^+ = 10^{-8}$ mole/l

avg $H^+ = \frac{10^{-6}+10^{-8}}{2} = 5.05 \times 10^{-7}$ mole/ l

$pH = -\log(H^+)$

$= -\log(5.05 \times 10^{-7}) = \underline{6.29}$

3) Hardness

Hardness is the concentration of multivalent cations in solution. It may induced due to Ca^{2+}, Mg^{2+}, Fe^{3+}, Fe^{2+}, Sr^{2+} etc.

Major constituents of hardness are Calcium & Magnesium

Hardness of water can be divided into—

Carbonate hardness & Non- Carbonate hardness.

It is measured by using spectro-photometric technique

a) Carbonate Hardness / Temporary Hardness

Hardness due to Carbonate (CO_3^{2-}) & Bicarbonate (HCO_3^-) of multivalent cations is called Carbonate Hardness. This hardness is also known as temporary hardness, as it can be removed by simple boiling or adding lime to water.

$$Ca(HCO_3)_2 \xrightarrow{\Delta} CaCO_3 \downarrow$$

$$Mg(HCO_3)_2 \xrightarrow{\Delta} \underset{(soluble)}{MgCO_3}$$

$$MgCO_3 + \underset{(lime)}{Ca(OH)_2} \longrightarrow Mg(OH)_2 \downarrow + CaCO_3 \downarrow$$

$$Mg(HCO_3)_2 + Ca(OH)_2 \longrightarrow Mg(OH)_2 \downarrow + Ca(HCO_3)$$

$$Ca(HCO_3)_2 + Ca(OH)_2 \longrightarrow 2CaCO_3 \downarrow + 2H_2O$$

b) Non-Carbonate Hardness / Permanent Hardness

Hardness due to chloride, sulphate, Nitrate etc of multivalent cations is known as NCH or permanent hardness as it can't be removed by ~~b~~ boiling.

Special techniques are used for removal of permanent hardness & these are called softening techniques.

Impact of Hardness

⇒ It leads to increase consumption of soaps because of lesser formation of foam.

⇒ Hardness leads to corrosion & incrustation in pipe.

⇒ Hardness of water makes the food tasteless.

Magnesium hardness due to sulphate ($MgSO_4$) causes induces effect (loose motion)
Hence its concentration is limited to $\boxed{50 mg/l}$

Note calcium hardness, however does not have any health problem.

Hardness is expressed as $CaCO_3$ equivalent of Ca^{2+} & Mg^{2+} present in water in $\boxed{mg/l}$

$$\text{Total Hardness} = (\text{Gram eq. of } Ca^{2+} + Mg^{2+}) \times \text{eq. weight of } CaCO_3$$

$$\text{eq. wt of } Ca^{2+} = \frac{40}{2} = 20$$

$$\text{eq wt of } Mg^{2+} = \frac{24}{2} = 12$$

$$\boxed{PH = \left\{ \frac{\text{wt in gm } Mg^{2+}}{12} + \frac{\text{wt in gm } Ca^{2+}}{20} \right\} \times 50}$$

as $CaCO_3$

Amount of Calcium & Magnesium in water is determine of by titrating with <u>versenate soluble</u> (<u>EDTA m/d</u>) Ethylene Diamine Tetra acetic acid. Using Eriochrome Black-T as (EBT) an indicator. EBT forms <u>Red colour</u> & titration changes it to blue

Permissible limit of hardness = 200mg/l
cause for rejection = 600mg/l

Q A 200 ml of sample of water has initial pH of 10, 30ml of 0.02N H_2SO_4 is required to titrate the sample to pH = 4.5. What is total alkanity of water in mg/l in the form of $CaCO_3$.

Sol^n

1ml of 0.02N of H_2SO_4 = 1mg of $CaCO_3$

200 ml sol^n ⟶ 30mg

1000 ml = $\frac{30 \times 1000}{200}$ = 150mg/l

Imp for obj

<u>Alkanity & Hardness</u>

Ca^{2+}, Mg^{2+} ———————— $Na^+, HCO_3^-, CO_3^{2-}, Ca^{++}, Mg^{++}$ Alkanity

Hardness $\{CO_3^{2-}, HCO_3^-\}$

CO_3^{2-}, HCO_3^{2-} Hardness

Bi-carbonate & Carbonate ions in water are generally induced by Calcium, magnesium & Sodium.

(i) If Na^+ alkanity is absent

~~Tot~~. $\boxed{\text{Alkanity = Carbonate Hardness}}$

(ii) If Na^+ alkanity is present in water ($NaHCO_3$, $NaCO_3^{2-}$)

Alkalinity > carbonate Hardness
Total hardness = Carbonate Hardness

(iii) Na^+ alkanity is absent only if Non-carbonate is present (SO_4^{2-}, Cl^- is present)

Total hardness = CH + NCH
= alkanity + NCH {From case 1}

Hence Carbonate Harness = Total hardness or alkalinity } whichever is less

Degree of Hardness on the basis of $CaCO_3$

Hardness	Degree
0 – 55	Soft water
56 – 100	slightly hard
101 – 200	Moderately hard
201 – 500	~~Ver~~ Very hard

Q If Total hardness & alkanity of sample are 300mg/l & 100mg/l (as $CaCO_3$) respectively. Then its carbonate & Non-carbonate hardness will be –

Solⁿ

Carbonate hardness = least of alkalinity / TH

CH = 100mg/l

TH = CH + NCH

NCH = TH − CH

= 300 − 100 = 200mg/l

Q Following chemicals were reported for a sample of water

~~$30\times5 = 1750$~~

Species	Concentration (milli equivalent/l) or gram equivalent
Chloride Cl^-	15
Sulphate SO_4^{2-}	15
Carbonate CO_3^{2-}	5
Bicarbonate HCO_3^-	30
Calcium Ca^{2+}	12
Magnesium Mg^{2+}	18
pH	8.5

Solⁿ

$TH = [Ca^{2+} + Mg^{2+}] \times 50 = (12+18)\times 50$

$= 1500$

$Alkalinity\ [CO_3^{2-} + HCO_3^-] \times 50 = (30+5)\times 50$

$= 1750$

सिर्फ Na & Ca से मतलब है
और CO_3 से
& HCO_3 से लेना

4) Chloride Content

Chlorides in water are mostly derived from Natural mineral deposit, agricultural or irrigation discharges.

Presence of chloride in high quantity indicates pollution of water due to sewage or industrial water.

Imp

Chlorides are estimated by Mohr's method in which water is titrated with standard silver Nitrate ($AgNO_3$ solⁿ) using potassium chromate (K_2CrO_4) as an indicator.

$$AgNO_3 + Cl^- \longrightarrow \underset{\text{white}}{AgCl\downarrow}$$

$$AgNO_3 + K_2CrO_4 \longrightarrow \underset{\text{Red brown}}{Ag_2CrO_4}$$

Acceptable limit → 200mg/l

Cause for rejection → 1000mg/l

5) Nitrogen Content

The presence of Nitrogen in water indicates presence of organic matter. Nitrogen may be present in any of the following form—

a) Free ammonia
b) organic ammonia (Albiminoid)
c) Nitrite (NO_2^-)
d) Nitrate (NO_3^-)

a) Free ammonia

The presence of free ammonia represents recent pollution. Free ammonia can be calculated by simple boiling of water & measuring the amount of ammonia gas liberated.

Permissible limit &
cause for rejection = 0·15mg/l

b) Organic ammonia

The presence of organic ammonia in water indicates the ~~amount~~ presence of Nitrogen before decomposition has started.

Organic ammonia can be calculated by boiling the already boil water & calculating the amount of ammonia liberated.

During this process a strong alkaline agent like $KMnO_4$ (potassium permagnet) is added so as to aid (मदद) the liberation of gas.

Note - Free ammonia with organic ammonia commonly called (Kjeldhas ammonia.)

Permissible limit of
organic ammonia = 0.3mg/l

c) Nitrite (NO_2^-)

⇒ The presence of Nitrite in water indicates the partial decomposition of organic matter.
Hence it is highly dangerous & not permitted in water.

⇒ Nitrite content can be determine by color matching technique.

Permissible / Acceptable limit = 0.

d) Nitrate (NO_3^-)

The presence of Nitrate in water indicates complete decomposition of organic matter thus it indicates old pollution of water.
The presence of Nitrate in water is not too harmful but in some cases if its concentration is in excess it affects the infants.

Note Nitrate is converted in Nitrite due to presence of acids in the intestine of infants which has very high affinity with (attracts) haemoglobin, hence replaces O_2 from it. Thereby leading to deficiency of oxygen from body & the disease is called Blue Baby disease

permissible limit / acceptable limit = 45 mg/l

Imp

Free ammonia → Recent pollution → 0.15 mg/l

Organic ammonia → N_2 before decomposition → 0.3

Nitrite → Partial decomposition → 0

Nitrate → Total decomposition (old pollution) → 45

6) Phosphorus

It is not toxic & do not represent direct health ~~&~~ impact, but is indirect threat to water quality because—(i) its facilitates rapid growth of aquatic plant.

(ii) It interferes with water treatment.

Even ~~at~~ a low concentration (0.2mg/l) interferes with water treatment.

7) Flourides

⇒ Upto 1mg/l ~~ppm~~ it helps to prevent dental cavities, during formation of permanent teeth, it combines ~~Dees~~ ~~wit~~ chemically with tooth enamel resulting in harder & stronger teeth.

⇒ Excess value (More than 1.5 ppm or 2ppm) can cause decolouration of teeth called Mottling of teeth

⇒ More than 5ppm can cause deformation of bones called bone flourisis

Acceptable limit = 1mg/l

Cause for rejection = 1.5mg/l

8) Metals

a) **Sodium** - • Excess of sodium concentration cause bad taste & it is harmful for Cardiac & Kidney patients.

• It is also corrosive to metal surface & in large conc it is toxic for plants.

Measurement Atomic absorption spectrophotometry

b) **Copper** - • High copper concentration affects lungs & respiratory system

• If $CuSO_4$ is more than 250mg/l it causes laxative effects.

Acceptable limit = 0.05mg/l

Cause for rejection = 1.5mg/l

c) **Iron & Magnese** - • They ~~cause~~ pose color problem.

• If concentration of Iron & Magnese are greater than 0.3mg/l & 0.05mg/l resp.

Note Some bacteria use Iron & Magnese compounds for their energy source resulting taste & odour.

rejection

Acceptable limit for iron - 0.1 - 1.0mg/l

" " " Magnese - 0.05 - 0.5mg/l

reject

d) **Calcium** - • It is essential for bones & teeth formation.
as(Ca)

Acceptable limit = 75mg/l

Cause for rejection = 200mg/l

e) **Sulphate**

Acceptable limit = 200 mg/l

Cause for rejection - 400mg/l

f) **Zinc**

Acceptable limit = 5mg/l

Cause for rejection = 15 mg/l

g) **Arsenic**

Acceptable limit = 0.01mg/l

Cause for rejection = 0.05mg/l

h) **Cadmium** - • It conc^n should not be more ~~Accept~~ than 0.01mg/l

• It is highly toxic for humans & for living thing.

(i) **Chromium** • should not be greater than 0.05mg/l

• Toxic for both plants & humans.

(j) **Cynide** • It should not be more than 0.05mg/l

k) **Lead** • It should not be greater than 0.05mg/l

• It is toxic to organs & tissues including ~~hard~~ heart, bones, kidney, intestines etc.

l) Mercurry - It should not be greater than 0.001 mg/l.

m) Dissolved Gases

(i) CH4 - which is study for its explosive tendency.

(ii) H_2S - imparts bad taste & colour

(iii) CO_2 - indicates biological activities imparts bad taste & water become corrosive.

(iv) Oxygen less than saturation level indicates oxygen deficiency.

Note → saturation dissolved Oxygen → 9.2mg/l
If less than 4ppm → fishes can't survive

Biological Water Quality Parameters

→ The most important organisms are called pathogen. These are capable of transmitting disease.
Eg- protozoa, virus, bacteria.

→ Bacteries are group under 4 categories, according to their shape.

1.) Spherical → Coccus (Cocci)
2.) Rod shaped → Bacillus (Bacilli)
3.) Comma shaped → Vibrium (Vibro)
4.) Spiral → Spirillum (Spirilla)

⇒ Pathogen bacteria can be tested & counted in laboratory but with only great difficulty

⇒ Hence these tests are not perform generally in routine.

⇒ The usually test are conducted to detect coliforms which themselves are harmless but their presence indicates presence of pathogens.

The test for coliforms are –

1) Membrane filter technique
2) MPN Test (Most Probable No)

1) Membrane Filter Technique

⇒ Water sample is poured on the sterile membrane filter. Membrane is then put in contact with nutrients that permits the growth of coliforms only.

⇒ After incubation for 20hrs no of visible colonies are counted.

2) MPN Test (Most Probable No)

This test is conducted in 3 phase –

1) Presumptive Test
2) Confirmed Test
3) Complete Test

1> **Presumptive test**

⇒ +ve ~~rs~~ tests results indicates likely presence of coliforms

2> **Confirmed test**

Sample of +ve tubes is used to conform the presence of coliforms.

3> **Complete test**

Used for obtain isolated colonies

⇒ Mix different dilutions of sample of water with lactose broth & incubate then at 37°C for 48 hrs (other substance which enhance the growth of coliform may be added)

⇒ The presence of acid or carbon dioxide in the test tube indicates presence of coliforms.

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Ch-3 Treatment of Water

Method of treatment employed depends on nature of raw water & desired standard of water supply.

The unit operation ~~of~~ in water treatment includes – Screening, aeration, Flocculation, Filteration, Disinfection, softening, deferization, desalination, sedimentation (plain), sedimentation with coagulation, Deflouridation.

The choice of particular sequence of treatment unit depend on quality of raw water & desired quality of treated water.

1) In case of ground water & surface water with storage having turbidity less than 10 NTU & free from color & odour. **Plain disinfection** (अकेला & एकबार) is sufficient for drinking purpose.

2) For ground water containing excessiv iron, dissolved CO_2 & odourous gases the treatment process will be – Aeration → Flocculation → sedimentation → Rapid gravity filter → disinfection → supply

~~3) If ground wa~~

3) If surface water contains turbidity less than 50NTU & where sufficient land is available then treatment process will be -

Sedimentation → Slow sand filter → Disinfection → supply

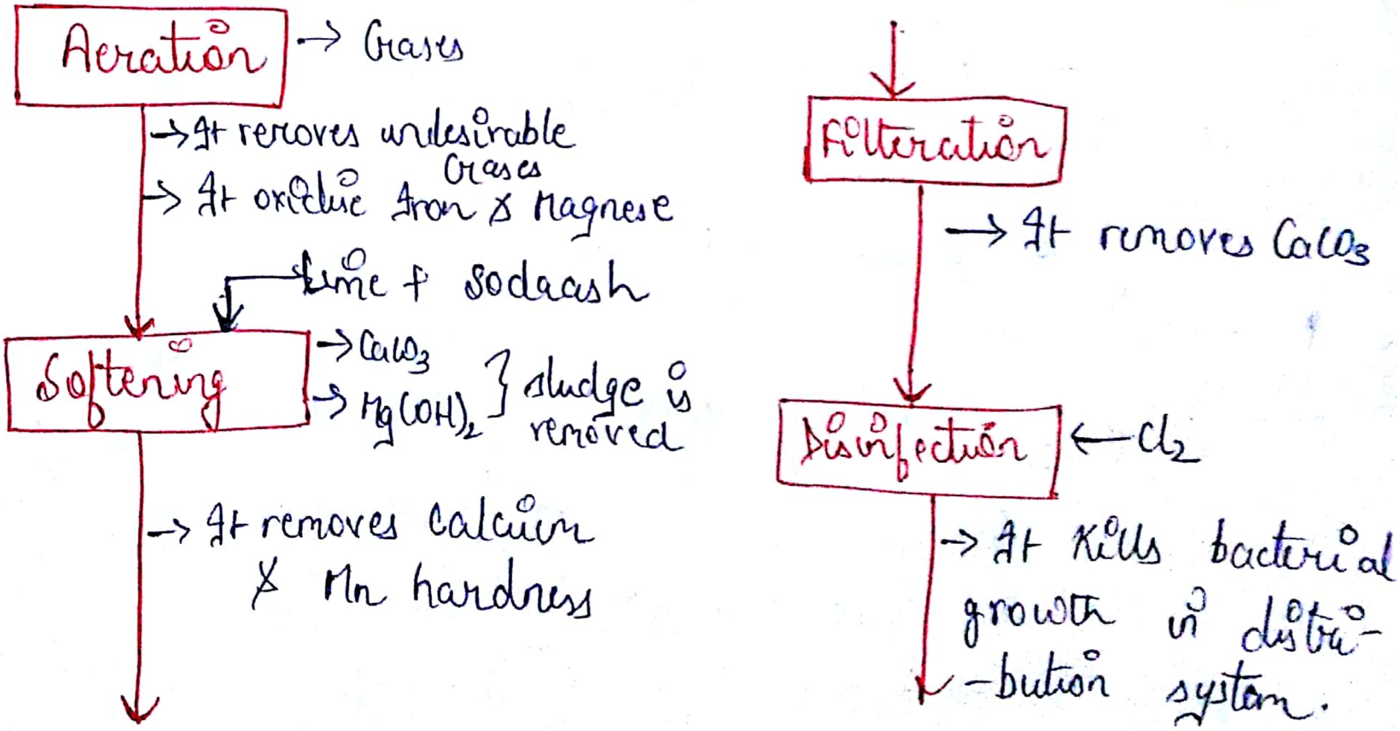
4) If Ground water contains only CO_2 & Gases, then Aeration & disinfection is sufficient.

5) Highly polluted surface water laden with Algae & other micro-organism the sequence of treatment will be -

Pre-chlorination → aeration → flocculation & sedimentat → Rapid Gravity filter/slow sand filter → disinfecti

Most Imp

Typical Plant for Treating hard Ground Water



→ It removes undesirable Gases

→ It oxidise Iron & Magnese

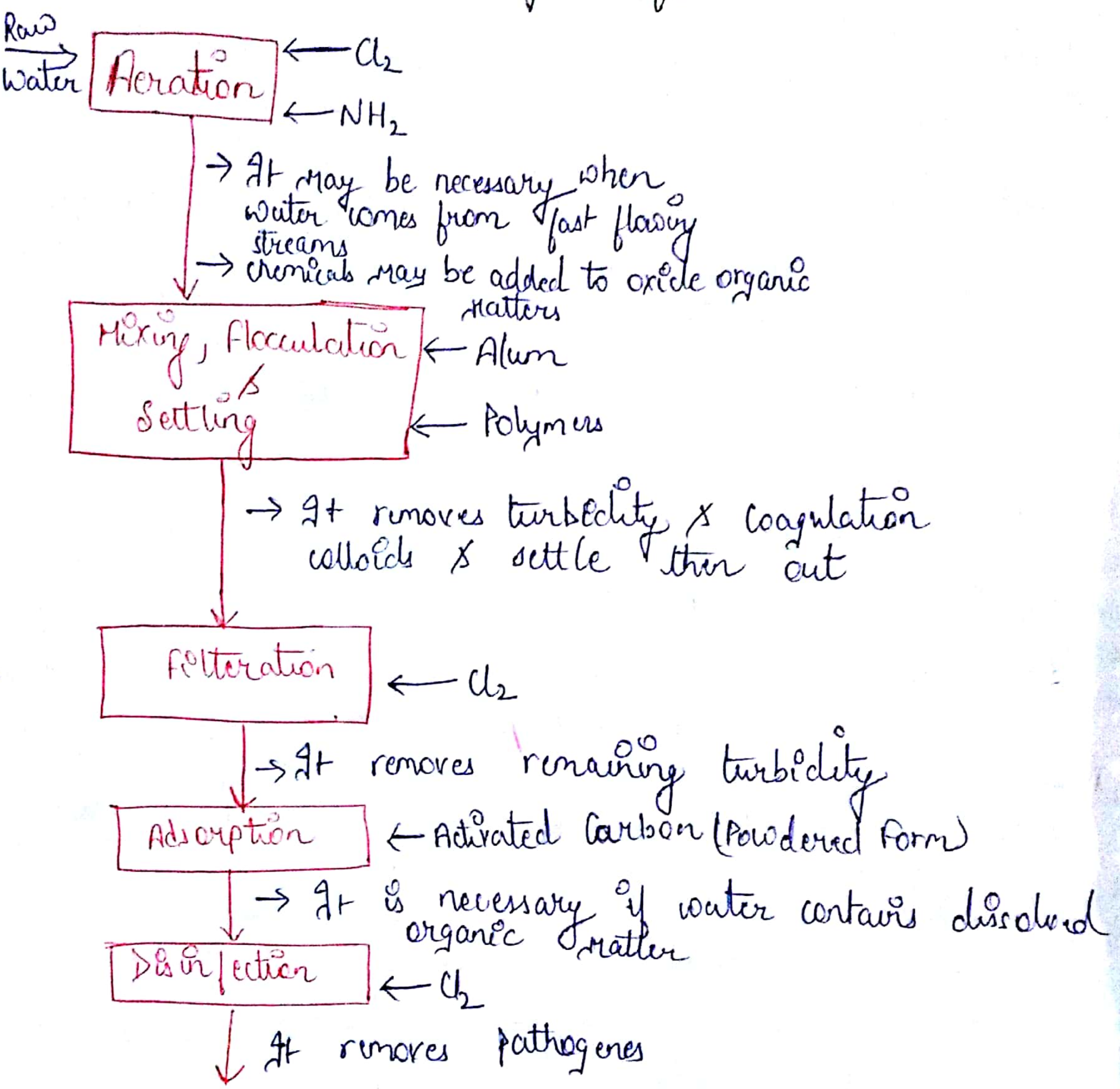
$CaCO_3$, $Mg(OH)_2$ } sludge is removed

→ It removes calcium & Mn hardness

→ It removes $CaCO_3$

→ It kills bacterial growth in distribution system.

Treatment Plant treating surface turbid water



Raw water → Aeration ← Cl_2 ← NH_2

→ It may be necessary when water comes from fast flowing streams

→ chemicals may be added to oxide organic matters

Mixing, Flocculation & Settling ← Alum ← Polymers

→ It removes turbidity & coagulation colloids & settle them out

Filteration ← Cl_2

→ It removes remaining turbidity

Adsorption ← Activated Carbon (Powdered Form)

→ It is necessary if water contains dissolved organic matter

Disinfection ← Cl_2

It removes pathogenes

Screening

→ These are classified as coarse screen & fine screen. Coarse screen are in form of bar & spaced at about 20-100mm c/c.

→ It is kept inclined & 3-6V : 1H.

Inclined screen helps in racking (velocity कम & particle ज्यादा अटकेंगी)

→ Coarse screen are sometimes also called Trash Rack

→ Fine screen is in the form of wire mesh having opening generally less than 10mm.

→ As the fine screen gets clogged easily so head loss increases hence we tried to avoid fine screens. Fine particles may be settled in sedimentation tank.

Prechlorination

→ Pre-chlorination refers to the ~~practice~~ process of infecting chlorine into raw water when it is not so turbid but has high bacterial count (No. of bacterias)

→ High dose of chlorine is used (2-5mg/L)

→ Pre-chlorination also kills ~~all~~ algaes & bacterias & it reduces color.

→ If excessive silt is present, prechlorination is not so effective, because silt absorb chlorine & make to it settled.

Aeration

→ Water is brought in contact with air.

→ It is used to remove undesirable gases which are dissolve in water like CO_2, H_2S etc.

→ It is also used to add oxygen to oxid~~ies~~ation of undesirable substances like oil, decomposting product of algae etc.

Note Aeration is generally practiced for ground water because ~~ground~~ surfaces water is in already contact with air.

→ It can also remove volatile liquids like phenolic compound, humic acid, but the rate is very slow.

→ It remove iron & magnese, ~~because~~ as iron & magnese are soluble in form Fe^{2+}, Mn^{2+}.. They are oxidise to $Fe(OH)_3$ & MnO_2 which are precipitate

$$4Fe^{2+} + O_2 + 10H_2O \rightarrow 4Fe(OH)_3 \downarrow + 8H^+$$

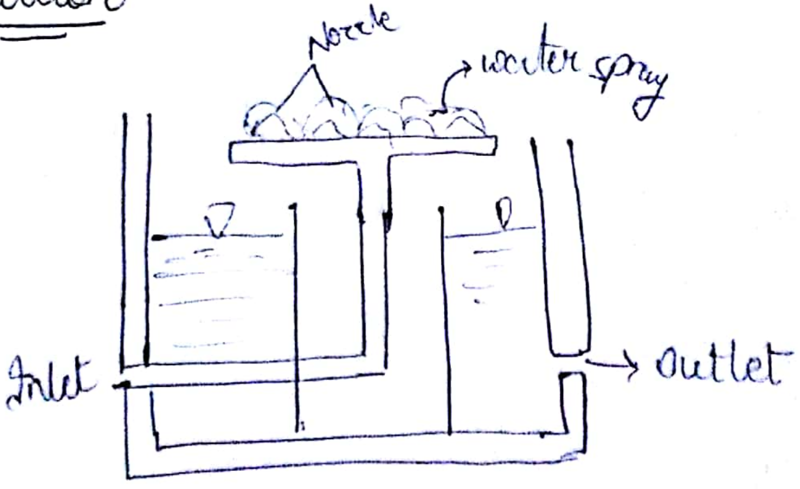
ppt.

$$2Mn^{2+} + O_2 + H_2 \rightarrow 2MnO_2 \downarrow + 4H$$

ppt.

Process of Aeration

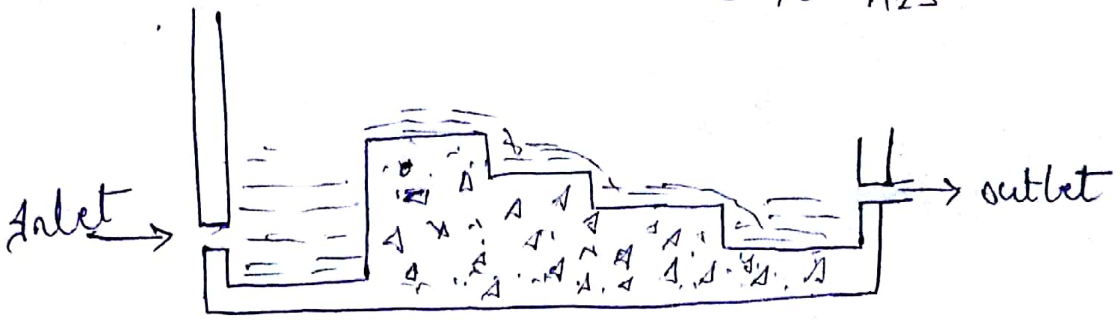
1) Spray Nozzle



It removes 90% CO_2 & 99% H_2S

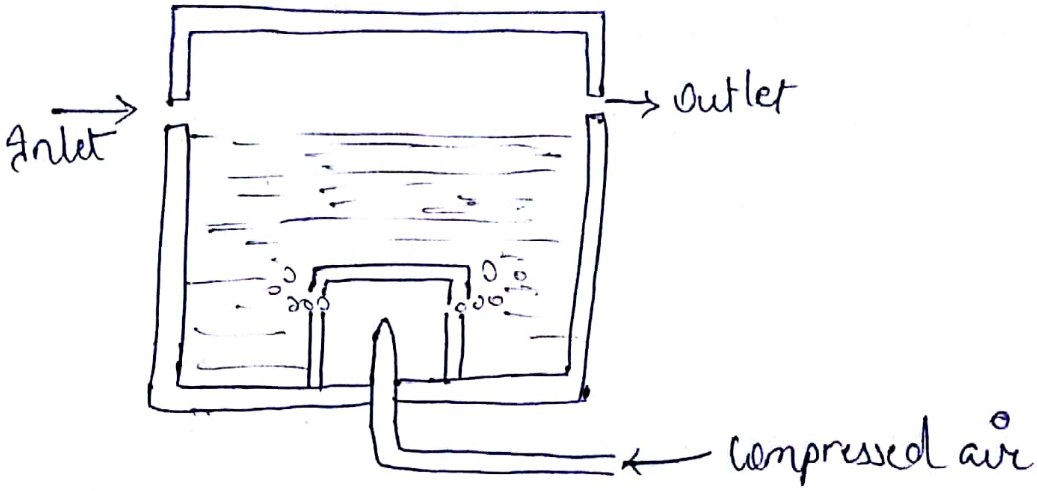
2) Cascade aerator

It removes 20-25% CO_2 & 35% H_2S



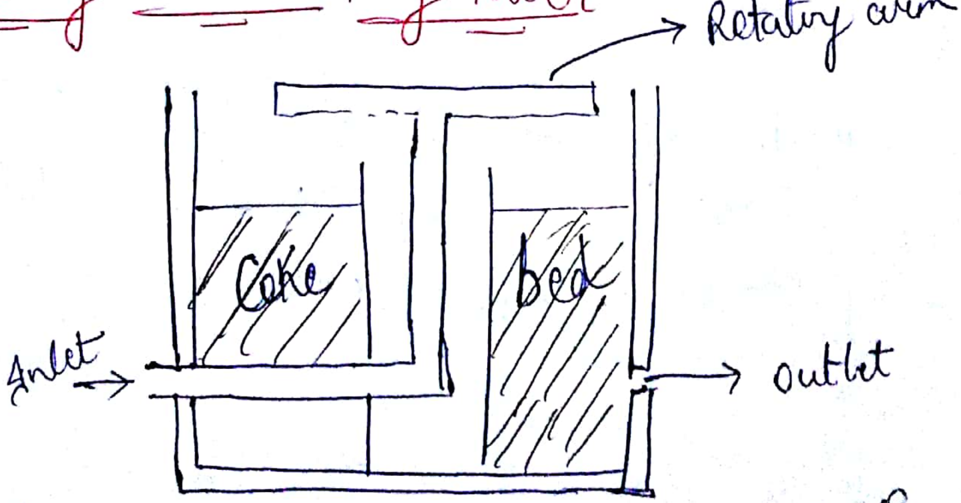
3) Diffused Air Aerator

~~It removes 20- 25% CO_2 & 35% H_2S~~



Water absorbs oxygen & colour, odour & taste are removed.

4) Trickling bed or Tray Tower



It is used mostly for removal of iron & Magnese (Fe^2 & Mn^{+2})

→ To help oxidation $KMnO_4$ may be added

→ This method is one of the best method for removal of CO_2

Algal Control

Algaes are minute organisms → single celled

→ The building of an impounding reservoir on an unclear stream encourage the growth of algae.

→ Alkaline water contains more nitrates & phosphates & this leads to growth of algae.

→ Algae tend to float & it is not easy to remove.

Imp Note Excessive oxygen causes corrosion

→ Thes best way to kill /remove algae is by ~~pre~~ pre-chlorination where organism are ~~Known~~ more a heavy dose of copper sulphate (2mg/l) or chlorine (3-5mg/l) is used.

Pre-settlement basin

The requirement of pre-settlement basin are there where water reaching main settling basin contains suspended matters greater than 1000 mg/l

Sedimentation → suspended

→ It is a natural process by which solids with higher density than fluid in which they are suspended by the action of gravity.

→ The purpose of sedimentation tank is to remove suspended solids.

Sedimentation is classified into 2 categories —

1) Plain sedimentation
2) Sedimentation with ~~cogut~~ coagulation

Sedimentation tank are classified as —

1) Quiscent Type (Fill & draw)
2) Continuous type

→ The Quiscent type tank will have detention time / Retention time 24hrs.

→ 3 No of tanks are required for Quisent type

→ Tank will be design for maximum Daily flow.

Max. daily flow = 1.8 × Avg daily flow.

→ Settling velocity is determined by stoke's law if not given —

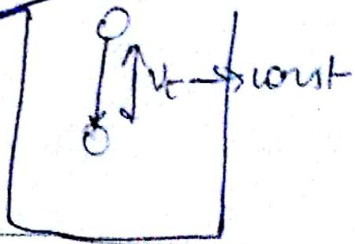
for 0.02mm to 0.0002mm

Terminal velocity

$$\boxed{V_t = \frac{(\gamma_s - \gamma_L)D^2}{18\mu}}$$

or

$$\boxed{V_t = \frac{(\rho_s - \rho_L)gD^2}{18\mu}}$$

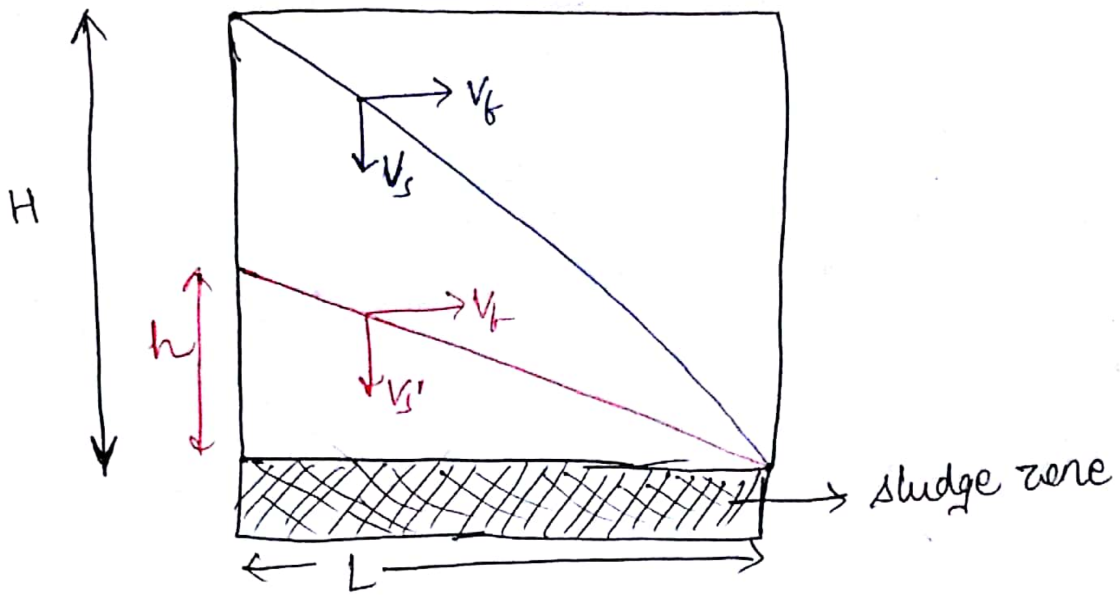


$$\boxed{V_t = \frac{(G-1)gD^2\rho_w}{18\mu}}$$

Assumptions of Stoke's Law

1) Conc^n of suspended particle of each size is same of all points of the vertical c/s.

2) A particle is remove when it reaches the bottom of settling zone.



Time taken in horizontal plane

$$= \frac{\text{length of tank}}{\text{velocity (Horizontal)}}$$

$$\text{velocity} = \frac{Q}{A} = \frac{Q}{BH}$$

$$\text{Time} = \frac{L}{Q/BH} \Rightarrow \cancel{LQ} \; \frac{LBH}{Q}$$

Detention time

$$\boxed{\text{Time} = \frac{LBH}{Q} = \frac{\text{Volume}}{\text{Discharge}}}$$

Time taken from height H

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

If the a particle of settling velocity V_s is introduced at top most point of inlet. It will be assumed to be removed if the time of falling, through a height H is equal to detention time (time taken by horizontal flow)

$$\frac{H}{V_s} = \frac{LBH}{Q}$$

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

→ V_s does not depend on H

→ This quantity $V_s = Q/BL$ is called overflow rate.

→ Hence surface overflow rate can be thought of at settling velocity of that particle which is introduced at the top point of inlet will reach the bottomost outlet point.

→ This implies that the particle having size greater than the particle for which settling velocity is equal to over flow rate is removed 100%

→ Particles having settling velocity less than over flow rate will not get 100% removed.

Percentage removal of particles having settling velocity $\boxed{(V_s')\ \%\ \text{for removal} = \frac{V_s'}{V_s} \times 100}$

Q A water sample having particle distribution as shown below.

Size	0.1mm	0.2mm	0.3mm	0.4mm	0.5mm	0.6mm
Quantity	10% (100g)	20% (200g)	15% (150g)	5%	30%	20%
V_s' (mm/s)	0.2	0.25	0.3	0.35	0.4	0.5

Total solid in suspension = 1000g

over rate flow of tank = 0.35mm/s

Find efficiency of tank.

85%

71%

57%

Sol^n^

particle $V_s' \geq V_s$ – completely removed.

As aboree over flow rate the particles are ~~100~~ totally removed i.e = 5+20+30
= 55%
= $\boxed{550g}$

Now for 0.3mm = $\frac{0.3}{0.35} \times 100 = 85\%$

mass get removed = $\frac{85.7 \times 150}{100} = \boxed{128.56\,g}$

particle removal for 0.2mm size = $\frac{0.25}{0.35} \times 100 = 71.42\%$

Mass get removed = $\frac{71.42}{100} \times 200 = \boxed{142.48\,g}$

particle removal for 0.1mm size = $\frac{0.2}{0.35} \times 100$
$= 57.14\%$

mass get removed = $\frac{57.14}{100} \times 100 = \boxed{57.14\,g}$

Total mass of ~~Afto~~ solid removed = 550 + 57.14 + 142.98 + 128.56
= 878.4g

$$\eta = \frac{878.4}{1000} = 87.8\%$$

Q Determine the surface area of settling tank for $0.5\,m^3/s$ flow. Using design over flow rate (velocity) as $32.5\,m^3/m^2/day$ also find depth of tank if detention time 95min. Assume $L/B = 2:1$ to $5:1$ & length of tank should not exceed 100m.

Sol

$Q = 0.5\,m^3/s$

~~$V_s =$~~ ~~$\frac{Q}{BL}$~~ $\frac{Q}{\text{surface area}}$ = Over flow rate.

~~LBH~~ Surface area $= \frac{0.5\,m^3/s}{32.5\,m^3/m^2/day}$

$$= \frac{0.5}{\frac{32.5}{24\times60\times60}} = 1329.23\,m^2$$

Volume $= T_d \times$ discharge

$= 95\times60\times0.5 = 2850\,m^3$

Depth $= \frac{\text{Volume}}{\text{Surf area}} = \frac{2850}{1329.23}$

Take $\frac{L}{B} = \frac{4}{1}$

Depth = 2.14m

LB = ~~Volume~~ area

$4B^2 = 1329.23$

$B = 18.3\,m$

$L = 18.3\times4 = 73.1\,m < 100$ APQ OK

Q A continuous flow sedimentation tank 3.5m deep & 65m long. The flow velocity observed is 1.22 cm/s what size of particle of of sp. gravity 2.65 may be effectively removed Kinematic viscosity of water is 0.01 cm²/s

Sol^n

$H = 3.5m$

$L = 65~~c~~m$

$V_f = 1.22 cm/s$

$G_s = 2.65$

ν ~~α~~ $= 0.01 cm^2/s$

$\nu = \frac{\mu}{\rho}$

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

$$V_s = \frac{V_f \times H}{L} = \frac{1.22 \times 3.5 \times 100}{65 \cancel{\times 0.01} \times 100}$$

$V_s =$ ~~6.56cm~~ ~~6.56m/s~~ ~~6.56cm/s~~ $\boxed{0.065 cm/s}$

$\rho_s = 2.65 \times 1000 = 2650$

$\mu = \nu \times 1000 = 0.01 \times 1000 \times 10^{-4}$

$$0.065 \times 10^{-2} = \frac{(2.65-1) \times 9.81 \times \overset{\rho_w}{1000} \times d^2}{18 \times 0.01 \times 10^{-4} \times \underset{\hookrightarrow \rho_w}{1000}}$$

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

$d^2 = 2.68 \times 10^{-5}$ m

$d = 0.026$ mm

Q A rectangular settling tank without mechanical equipment is to treat 1MLD of raw water. T_d = 2.5 hr, overflow rate = 8m/min. depth of water + sediment = 4.5m find length of tank & width of tank if an allowance of 1.5m is to be made for sediment.

Sedimentation with Coagulation (पहला तोड़ना फिर जोड़ना)

→ Efficiency of sedimentation tank is very low when water contains very fine suspended particles. Coarse solids are effectively removed by sedimentation process.

→ Colloidal particles pass along with the effluent.

→ A colloidal dispersion is defined as stable ~~when~~ as the dispersion shows little or no tendency to aggregate.

→ For settlement of colloidal particles aggregation is essential, The aggregation of colloidal particle require —

(i) Particle destablization to permit attachment.

(ii) Agglomeration (Aggregation) of destablized particles.

जुड़ना

Coagulation is defined as addition of chemicals to a colloidal dispersion which results in particle destabilization which tends to keep particle apart, the ~~coag~~ chemicals are called coagulants

→ The zetapotential is the measure of stability of particles

→ Higer the zetapotential more stable is the particle.

→ The purpose of coagulation is to reduce this zetapotential.

→ Suitable ions are usually provided by aluminium salt, iron salt, lime & various polymers which are common coagulants.

Mechanism of Coagulation

A.) Ionic layer Compression
B.) Adsorption & Charged neutrilization
C.) Sweep Coagulation
D.) Inter-particle bridging.

A. Ionic layer Compression

→ The quantity of ions in water surrounding a colloid has an effect of reducing the repulsive force.

→ A high ion concentration compresses the layer of positive charge ions towards the surface of colloids.

→ If this layer is sufficienty compressed then the particles will grow in size & settle down.

B. Adsorption & charge utilization

→ Nature rather than quantity of ions is important in sedimentation.

→ If Alum is added in water it will form Al^{3+} & SO_4^{2-}.

→ The sulphate ion may remain in this form or can be combined with other cations like Mg^{2+}, Na^+

→ However Al^{3+} will not react immediately

→ Once the charge is neutralize, free contact of particle can occur so it will lead to grow in size & thus settling take place.

C Sweep Coagulation

→ The aluminium hydroxide $Al(OH)_3$ is formed when alum is added water is sticky ppt.

→ These are heavier than water & get settled by the action of gravity.

D Inter-particle Bridging

→ large molecules may be form when aluminium & ferric sulphate dissociate in water.

→ Many colloids can attatch on these molecules resulting in settling of mass.

Common coagulants added in water

1.) Alum
2.) Copperas
3.) Chlorinated Copperas
4.) Sodium aluminate
5.) Lime

1.) Alum

Chemical formula – $Al_2(SO_4)_3 \cdot 18H_2O$

$Al = 27,\ S = 32,\ O = 16,\ H = 1$

$= 27\times2 + 3[32 + 4\times16] + 18\times18$

$= 666$

→ Alum reacts with HCO_3^- alkalinity to form $Al(OH)_3$ which is ppt.

→ This attracts other & fine particles & suspended matter thus grow in size & settles down.

$$\underbrace{Al_2(SO_4)_3\cdot 18H_2O}_{\boxed{666}} + 3Ca(HCO_3)_2 \longrightarrow \boxed{3CaSO_4} + \overset{[27+17\times3]\times2\ \boxed{156}}{2Al(OH)_3}\downarrow + \underbrace{6CO_2}_{\boxed{264}}\uparrow + 18H_2O$$

~~$+6CO_2 + 18H$~~

permanent Hardness

Al 3

$6[12+32]$

Imp

666 g of Alum forms – 156 g of $2Al(OH)_3\downarrow$

1 g alum forms $\frac{156}{666} = \boxed{0.23g}$ Aluminium Hydroxide

→ This reaction introduces permanent hardness in water & water become corrosive.

→ Amount of coagulant (alum) depends on turbidity & color of water.

→ Alum requires alkalinity of water

→ Normal dose is 10 - 30 mg/l of water

→ It is effective in pH range - 6.5 to 8.5

→ It is cheap flocc's formed are stable.

→ Disadvantages - It is difficult to dewater the sludge formed.

→ Alum also reduces color, taste & odour.

IMP Note 1 g of alum gives - 0.24g $Al(OH)_3$

666 g of alum gives → 264g CO_2

1g alum gives $\frac{264}{666}$ = 0.4g of CO_2

Q. At a water Treatment plant 12 million litre of water is treated daily using alum dose of 16mg/l find total quantity of alum used daily, quantity of alluminium hydroxide daily & amount of CO_2 released.

Quantity of alum = Quantity of water in day × total alum

Sol^n Water → 12 MLD

Alum → 16 mg/l

Total quantity of alum $= \frac{16\,mg}{L} \times \frac{12 \times 10^6\,L}{10^6\,kg}$

$= 192 \times 10^3$ g

1 gm → ~~0.24~~ 0.234 g of $Al(OH)_3$

192×10^3 g → $0.234 \times 10^{-3} \times 192$ kg

$= 44.92$ Kg of $Al(OH)_2$

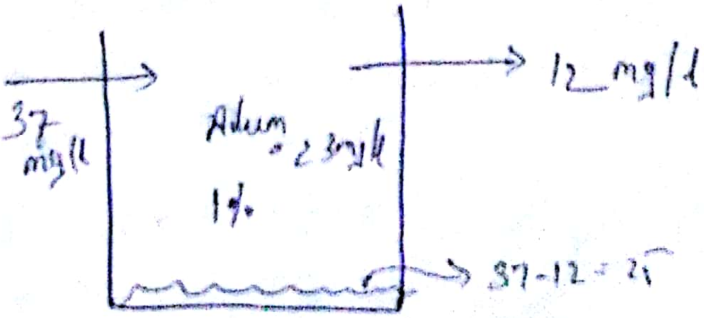
1 gm alum → 0.4 g CO_2

$192\,kg \times 10^3$ → $0.4 \times 10^{-3} \times 192$

$= 76.8$ kg

Q A coagulation treatment plant with a flow of 0.5 m³/s is dosing alum at 23 mg/l. No chemical is added the raw waste water suspended solid conⁿ is 37 mg/l. The effluent suspended solid conⁿ is 12 mg/l. Sludge content is 1% solid (by volume) & a specific gravity of sludge solid is 3.01, what vol. of sludge must be discarded per day.

Sol^n



Soln Discharge – 0.5 m^3/day

Quantity of water in 1 day ⇒ 0.5 m^3 × 86400

[$1m^3 = 1000 l$]

= 43200 m^3/day

= 43200 × 1000 l/day

= $\frac{432000 \times 10^3}{10^{-6}}$ MLD

= 43.2 MLD

suspended solids settled = $25 \times 10^{-6} \times 43.2 \times 10^6$

SS = 1080 kg

Total alum per day = ~~43.2 × 23 = 993.6~~ kg

= $43.2 \times 10^6 l \times 23 \times 10^{-6} \frac{kg}{l}$

= 993.6 kg

1 kg of alum ⟶ 0.234 kg of alum

993.6 kg ⟶ 993.6 × 0.234

= 232.5 kg ↓ ppt

Total solid = 1080 + 232.5

= 1312.5 kg

AQ sludge contains 1% solid

sludge = solid + water

1% + 99%

~~Quantity of water = 99 × 1312.5 = 129937.5 kg~~

अगर by wt में होता तो wt में

लेकिन यहाँ by vol. कहा है तो

Vs निकालना है।

$$V_s = \frac{W_s}{G_s \gamma_w} = \frac{1312.5}{3.01 \times 1000}$$

$$V_s = 0.436 m^3$$

$$V_w = 99 \times 0.436 = 43.16 m^3$$

Total sludge volume $= 43.16 + 0.436$
$= 43.59 m^3/day$

Alkalinity-Coagulation Relationship

⇒ The coagulation of a metallic salt release H^+ ion. These H^+ neutralize alkanity. If the initial alkanity of water is low, coagulation results in further reduction of pH.

⇒ Since optimum pH value must be maintained for effective coagulation so alkalinity must be present low alkaline water must be added with lime, $Ca(OH)_2$ or soda ash Na_2CO_3 to increase alkalinity of water.

$$Al_2(SO_4)_3 \cdot 18H_2O + 3Ca(OH)_2 \longrightarrow 3CaSO_4 + \underset{ppt}{2Al(OH)_3}\downarrow + 18H_2O$$

$$Al_2(SO_4)_3 \cdot 18H_2O + 3Na_2CO_3 \longrightarrow 3Na_2SO_4 + \underset{ppt}{2Al(OH)_3}\downarrow + 3CO_2 + 15H_2$$

2) Copperas ($FeSO_4 \cdot 7H_2O$)

→ It is added in water with lime

$$FeSO_4 \cdot 7H_2O + Ca(HCO_3)_2 \longrightarrow CaSO_4 + \underset{\text{soluble in water}}{\overset{Fe(HCO_3)_2}{\cancel{Fe(OH)_2}}} + 7H_2O \text{ — ② react}^n$$

→ When copperas is added first reaction is ~~above~~.

~~$FeSO_4$~~ .

Note When lime is added in copperas then reac^n is –

$$FeSO_4 \cdot 7H_2O + Ca(OH)_2 \longrightarrow CaSO_4 + \underset{\text{soluble in water}}{Fe(OH)_2} + 7H_2O \rightarrow \text{① react}^n$$

$$Fe(HCO_3)_2 \xrightarrow{O_2} Fe(OH)_3 \downarrow \longrightarrow \text{③ react}^n$$

1 mol of copperas will form 1 mol of ferric hydroxide $Fe(HCO_3)_2$

→ Copperas is used for water, which is not coloured

→ It works in pH range of 8.5 and above

→ Quantity used is same as that of alum (10-30 mg/l)

→ It forms heavy flocs.

3) Chlorinated Copperas

→ It is form by the addition of chlorine in copperas.

→ Chlorinated copperas is an effective coagulant as it works with large pH range.

$$FeSO_4 \cdot 7H_2O + Cl_2 \longrightarrow Fe_2(SO_4)_3 + FeCl_3$$

$$Fe_2(SO_4)_3 + Ca(HCO_3)_2 \longrightarrow 2Fe(OH)_3 \downarrow + CaSO_4$$

$$FeCl_3 + Ca(HCO_3)_2 \longrightarrow Fe(OH)_3 \downarrow + CaCl_2$$

→ Chlorinated Copperas with lime is effective coagulant.

4) Sodium aluminate ($Na_2Al_2O_4$)

→ It reacts with Calcium & Magnesium present in water & will form sticky ppt of calcium & Magnesium Aluminate.

$$Na_2Al_2O_4 + CaSO_4 \longrightarrow \underset{\text{calcium aluminate}}{Ca(Al_2O_4)_2} \downarrow + Na_2SO_4$$

$$Na_2Al_2O_4 + CaCl_2 \longrightarrow \underset{\text{Calcium aluminate}}{Ca(Al_2O_4)_2} \downarrow + 2NaCl$$

Similarly for Mg

⇒ It reduces temporary & permanent hardness but it is costlier than alum.

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Note Sodium aluminate is very useful for water which does not require alkalinity.

⇒ Its pH range is 6–8.5

5.) Lime

⇒ It is a unique coagulant in cases where phosphorus removal is ~~desired~~ required, also it may be desired to reduce ammonia.

Imp
Comparison b/w alum & Iron salt

1.) Iron salts produces heavy flocs

2) Time required for floc formation is smaller in Iron.

3) Iron salts works in large pH range.

4.) Disadvantage It produces Iron bacteria & also imparts corrosiveness

5.) It also impart reddish or brown ~~tint~~ tinge

Note For drinking water purpose alum is used. & iron salts are used in case of sewage treatment.

Coagulation Aid

→ It is a chemical which is used with main coagulant to accelerate the action of coagulant.

→ Finely divided clay, bentonite & activated carbon powder are most commonly used as coagulant aid.

→ Activated silica & ~~ploo~~ polyelectrolytes are other examples.

→ The study of relationship among pH, colloidal concentration of water & optimum dose of coagulant is necessary of satisfactory operation of sedimentation process.

There may be 4 cases –

1) High turbidity – low alkalinity

This type of water requires low dose of coagulants.

2) High turbidity – High alkalinity

This type of water is unaffected by the addition of coagulants. Higher coagulant dose should be used to ensure sweep of the flocs.

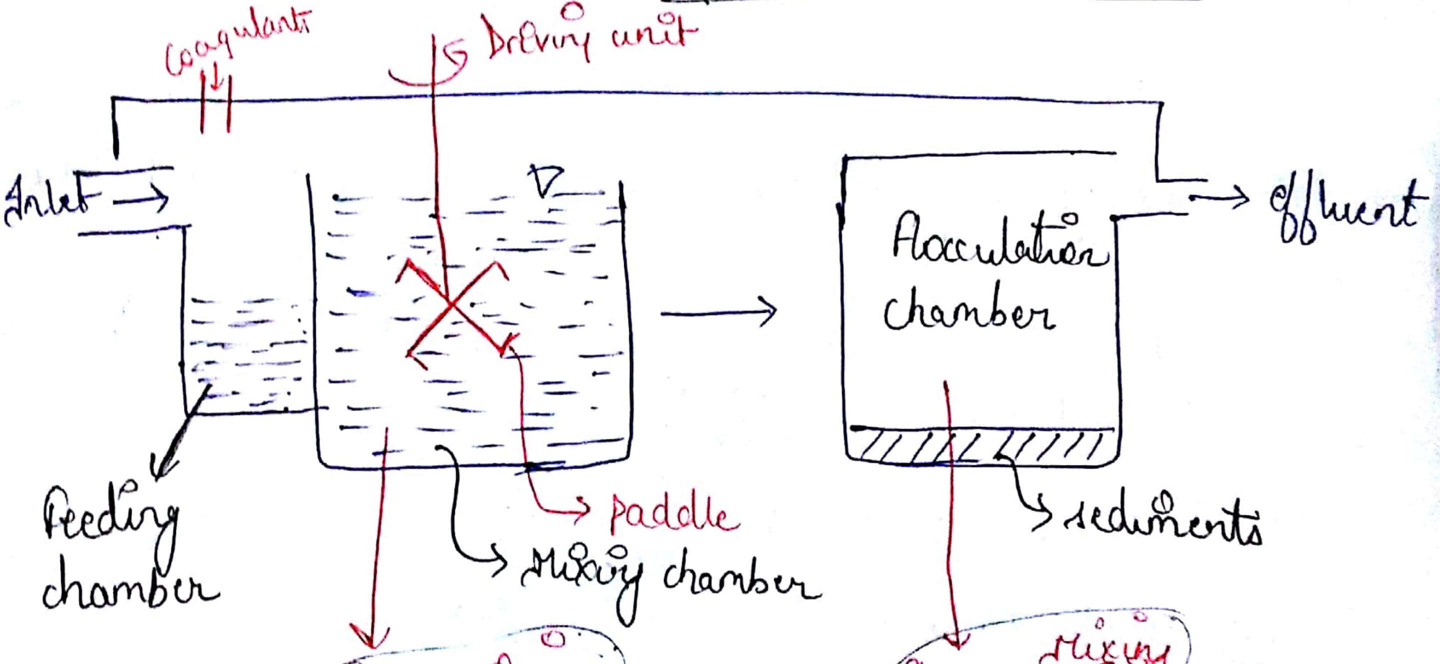
3) Low turbidity - High alkalinity

Due to low colloidal concentration the low dose of coagulant is less effective. Hence high coagulant should be use.

4) Low Turbidity - low alkalinity

High dose is required

Flow diagram for Coagulation & Sedimentation



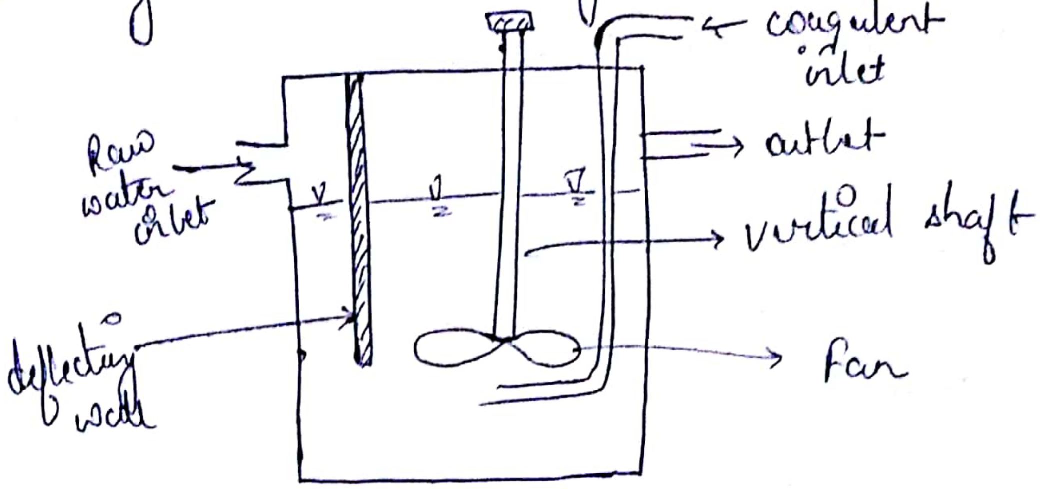
Fast mixing (ppt forms)

Slow ~~Motion~~ mixing ppt gets opportunity to combined with each other & are able to get attached with colloids.

Not imp

✓ Mixing

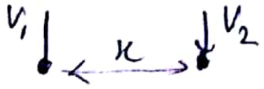
→ Mixing is done by mechanical mixer



→ The raw water & coagulants are mixed properly

→ The intensity of mixing depends upon temporal mean velocity gradient

→ Velocity Gradient is a Measure of relative velocity of two particles of fluid & distance b/w them.



velocity Gradient $\boxed{G = \dfrac{V_1 - V_2}{x}}$

→ Velocity Gradient indicates ~~two~~ that how velocity changes b/w points

→ Unit of G. is sec^{-1}

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

P = power in Watt

μ = dynamic viscosity Ns/m^2

V = Volume of tank in m^3

→ Propeller type impeller is used.
पार्टिकल पर धकेलने का काम करता है।

→ Detention Time = 30 - 60 sec

→ Deep circular or square tank is used

$$\boxed{\frac{H}{D \text{ or } B} = 1:1 \text{ to } 3:1}$$

H = height
D = ~~dia~~ diameter (circular)
B = sire (square)

→ G is kept 300 per sec or more

$$\boxed{\frac{\text{Impeller dia}}{\text{tank dia}} = 0.2 - 0.4}$$

→ Shaft speed is so kept such that the tangential velocity greater than 3m/s is Emparted at tip of the Blade.

Flocculation

→ It is basically slow mixing in which colloidal particles are brought into intimate contact in order to promote agglometer.

→ The rate of flocculation is depend on—

1.) Turbidity
2.) Type of coagulants & its dose
3.) Temporal mean velocity gradient

$$\boxed{\frac{G_{inlet}}{G_{outlet}} = 2}$$

V.Imp Obj

Data for sedimentation tank

→ overflow rate is normally adopted as (15,000 – 30,000 l/m²/day) for plain sedimentation & (30,000 - 40,000 L/m²/day) for sedimentation with coagulation.

→ Detention Time is (3-4 hrs) for plain sedimentation & (2-2.5hrs) for sedimentation with coagulation.

Q × Detention Time = Volume of tank

→ Horizontal flow velocity (V_f = 0.3 m/min (Range 0.15 to 0.9 m/min))

→ V_f × Detention time = length of tank

→ Width of tank is usually 10 - 12m

→ length = 4 × width

Scanned by CamScanner

→ * Sedimentation tank is designed for maximum daily flow

→ For tank without mechanical sludge removal additional depth is provided (i.e 0.8 – 1.2m) for storage of sludge ~~zone~~.
This zone is called sludge zone

→ Depth of tank is normally taken as 3m

Calculation of Velocity Gradient (G)

Power = Drag force × Relative velocity of impeller × fluid.

$= F_D \times V_r$

Relative velocity $V_r = V_p - V_w$

$$\boxed{F_D = \frac{C_D \rho_w A_p \cdot V_r^2}{2}}$$

C_D = coefficient of Drag
= ①.8 for paddle with flat plate

ρ_w = Density of water

A_p = area of plate

V_r = Relative velocity = $V_p - V_o$

$$V_r = V_p - KV_p$$

$$V_r = V_p(1-K)$$

where $K = 0.25$ (Normally)

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

where $P = F_D \times V_r$

$$P = \frac{C_D \rho_w A_p \cdot V_r^2}{2} \times V_r$$

$$\boxed{G = \sqrt{\frac{C_D \rho_w \cdot A_p V_r^3}{2\mu V}}}$$

Q A flocculation chamber 30m long 12m wide × 4.5m deep is to treat 75 MLD of water, it is equiped with 12m long 0.3m wide paddle supported parallel to & move by 4 horizontal shafts ~~with~~ ~~&~~ which rotate at a speed 2.5 rpm The center line of paddle is 1.8m from shaft & 2 paddles are mounted on each shaft one opposite to other if the mean velocity of water is 1/4 of the velocity of paddle. Find Power consumption, time of flocculation (Detention time) or/& value of G. Kinematic viscosity = 1.31×10^{-6} m²/sec

Filteration

⇒ It is the one of the most important process in the water purification / treatment process.

⇒ Filtration removes the suspended solids that do not get removed by sedimentation process.

⇒ In filteration process water is passed through a filter medium in order to remove suspended matter.

⇒ During filtration the turbidity & colloidal matter are removed.

⇒ The bacterial content of water is considerably reduced due to presence of ~~actually~~ active zoological layer. on the top of filtering medium.

⇒ James Simpson established first filter for the chelsea water company in 1829 london. (slow sand filter).

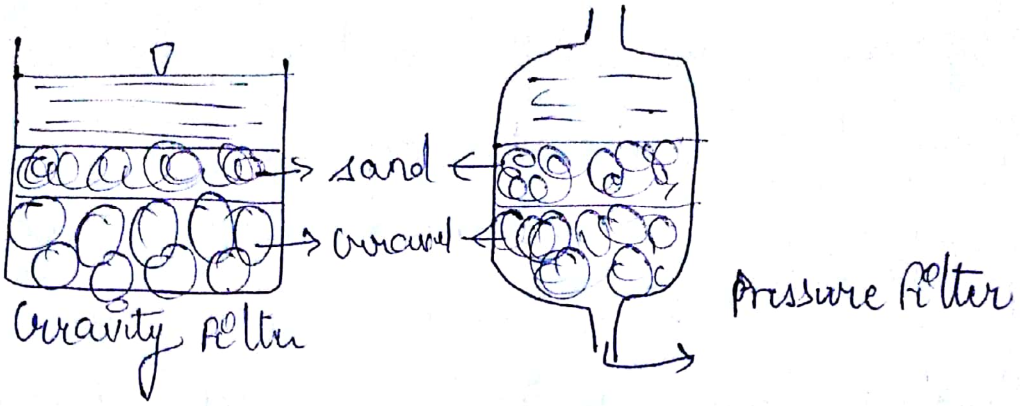
⇒ Rapid sand filter was introduced later in 1900 - 1910

⇒ Filters are classified as:—

1.) Gravity filter
2) Pressure filter

⇒ In gravity filter the head required to flow through sand is provided by the head of water over the sand medium.

⇒ The necessary head would be provided by pressure from the outside so the pressure filters are inside a closed container.



Theory of filtration

1) Mechanical straining
2) Sedimentation
3) Bio-logical action
4) Electrolytic charges.

1) Mechanical straining

Most of the particles are removed in upper layer arrested impurities including the coagulating floc forms a mat on top, which helps in straining.

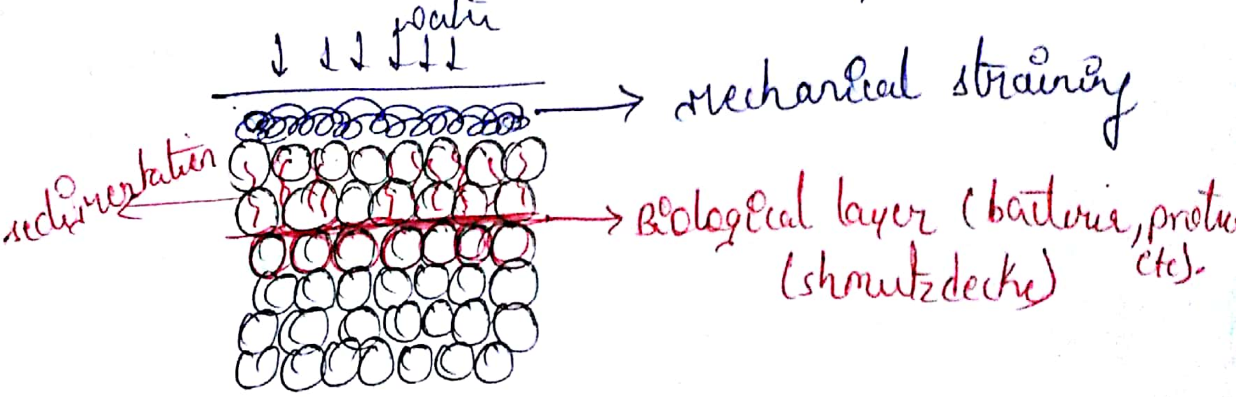
2) Sedimentation

Particles finer than voids are removed by sedimentation.

3) Biological action

when a filter is put & raw water is applied on a filter, during the first few days upper layer of sand is coated with reddish brow sticky deposit of partly decomposed organic matter along with Iron, Magnese, aluminium etc. After sometime a layer forms in upper portion of ~~sd~~ algae, bacteria, protozoa etc. This layer is called Shmutzdecke (dirty skin), which ~~at~~ acts as extremely fine mashed straining mat.

The organic impurities in water become food for different micro-organisms, these ~~back~~ bacterias not only decompose these organic matter but convert them also in harmless compound.



4.) Electrolytic charges

⇒ Sand grains in filter & impurities have opposite charges

⇒ Thus because of their interation chemical characteristi change & water become pure.

⇒ During washing process, neutralised matter is removal.

Filter media

The ideal filter medium should have a size & be of such material that it will provide satisfactory effluent, that retains max quantity of solids & readily ~~daky~~ cleaned with minimum amount of water.

Commonly used filter material are – 1) sand
2) anthracite
3) Garnet sand or ~~lignite~~, limenite
4) other locally available materials.

1) Sand

⇒ It is cheapest filter medium & is widely used.

⇒ Sand used for filter should be free from clay, silt, loam, suspended matter & other micro-organisms.

⇒ It should be fairly uniform, It shall be hard.

⇒ Soluble fraction in hydrochloric acid should not be more than 5%

⇒ Specific gravity shall be in the range of 2.55 – 2.65, wearing loss should not be more than 3%

⇒ Effective size should be 0.2mm – 0.3mm for slow sand filter & 0.45–0.7mm for rapid sand filter

⇒ Coefficient of uniformity $C_u = \frac{D_{60}}{D_{10}}$ = 3-5 for slow sand filter (poorly graded)

1.3 – 1.7 for rapid sand filter

2) Anthracite

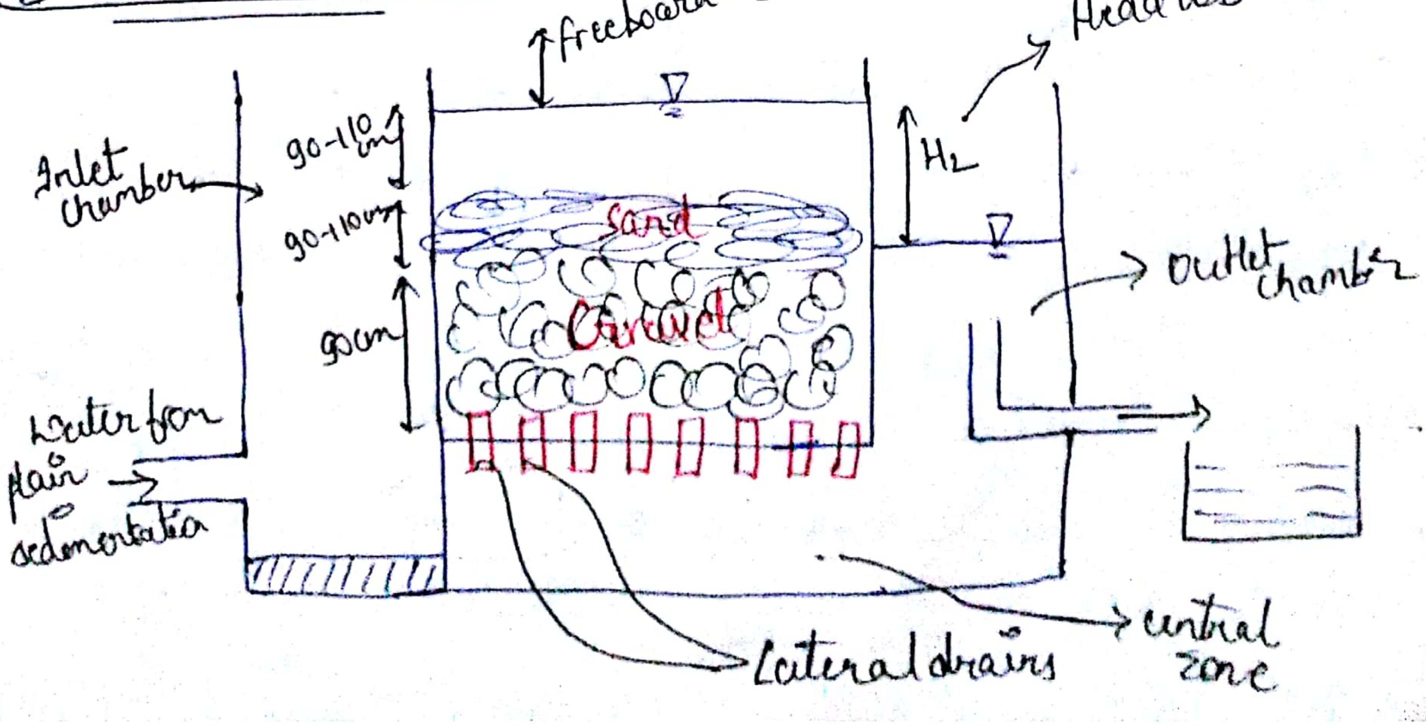
⇒ Crushed anthracite is successfully used as filter medium as a substitute of sand & may be used with conjuction with ~~sea~~ sand.

⇒ It is more costly.

3) Garnete sand / Ilmenite

It has high specific gravity (≃ 4.2) & is dense material because of its higher cost it is not used generally.

Slow Sand Filter



⇒ In slow sand filters gravel layer is provided only to support the sand layer.

⇒ Slow sand filter ~~at~~ utilizes effluent from plain sedimentation only.

⇒ Depth of tank is in the range of 2.5–3.5m overall depth.

⇒ Plan area required is 100–2000m² for per unit

⇒ Medium of filter used can be sand, anthracite, garnete sand etc.

⇒ The depth of sand layer is 90–110cm., depth of water over sand layer is same as that of depth of sand layer.

⇒ Depth of Gravel Medium is 60–90cm.

⇒ Effective size of sand particles (D_{10}) for filter medium is in the range of 0.2 – 0.3mm

⇒ coefficient of uniformity for filter medium is 5 as per Government of India manuals.

⇒ Top 15–30 cm sand is of finer variety in comparison to remaining sand (as to remove more finer particles)

⇒ Design of filter is approximately is done for 10–15yrs

⇒ In fresh filter head loss of 10–15cm is observed, which goes on increasing with operation of filter.

⇒ The cleaning of filter should be done when headloss becomes (0.7 – 0.8) times to depth of filter medium.

⇒ During cleaning top layer of sand is scraped & (1.5 – 3 cm) of sand is removed in each cleaning. Filter is washed with ~~two~~ brood water. Washed filter is then loaded with raw water but effluent is not used.

⇒ After (24 – 36 hrs) a film of arrested impurities is formed & then filter become ready for operation.

⇒ The frequency of cleaning is (1 – 3 months).

⇒ Rate of filtration is (100 – 200 l/hr/m2)

⇒ Efficiency of slow sand filter in bacterial removal is (97 – 98%)

⇒ If water is pre-chlorinated efficiency can go upto (99.5 – 99.9%)

⇒ This filter cannot be used if turbidity is greater than (> 50 ppm)

⇒ The effluent of slow sand filter contains less nutrients ~~(N & P)~~ (Nitrogen & Phosphorus) & enhance reduce possibility of after growth of organisms in distributory system. It is designed for (maximum ~~day~~ daily demand)

⇒ It is used in smaller plants in villages.

⇒ No. of beds of filter required depends on area –

upto 20m² area ⟶ 2 bed $(\overset{\text{operation}}{\overset{\uparrow}{1}} + \overset{\text{standby}}{\overset{\uparrow}{1}})$

upto 21-249m² —— 3 bed (2 + 1)

250 – 649m² ⟶ 4 bed (3 + 1)

650 – 1200 m² ⟶ 5 bed (4 + 1)

1200 onwards ⟶ 6 bed (5 + 1)

Q Design a slow sand filter for a population of 40,000 with an avg rate of water supply of 150ml/capita/day. Assume rate of filtration 150 l/m²/hr. Take L:B = 2:1

Sol

$$\text{Max daily demand} = \overset{1.8 \times \text{avg rcd}}{1.8 \times 150}$$

$$= 270 \text{ lpcd}$$

$$\text{Total water required} = 40,000 \times 270$$

$$= 10800000 \text{ lpd}$$

$$= \frac{10800000}{24} = 450000 \text{ lph}$$

$$\text{Area of slow sand filter} = \frac{\text{Demand}}{\text{Rate of filtration}}$$

$$= \frac{450000}{150} = 3000 \text{ m}^2$$

Area > 1200m²

Total unit required = 6

5 operational + 1 standby

Area of 1 operational unit = $\frac{3000}{5}$ = 600 m^2

Use L : B = 2 : 1

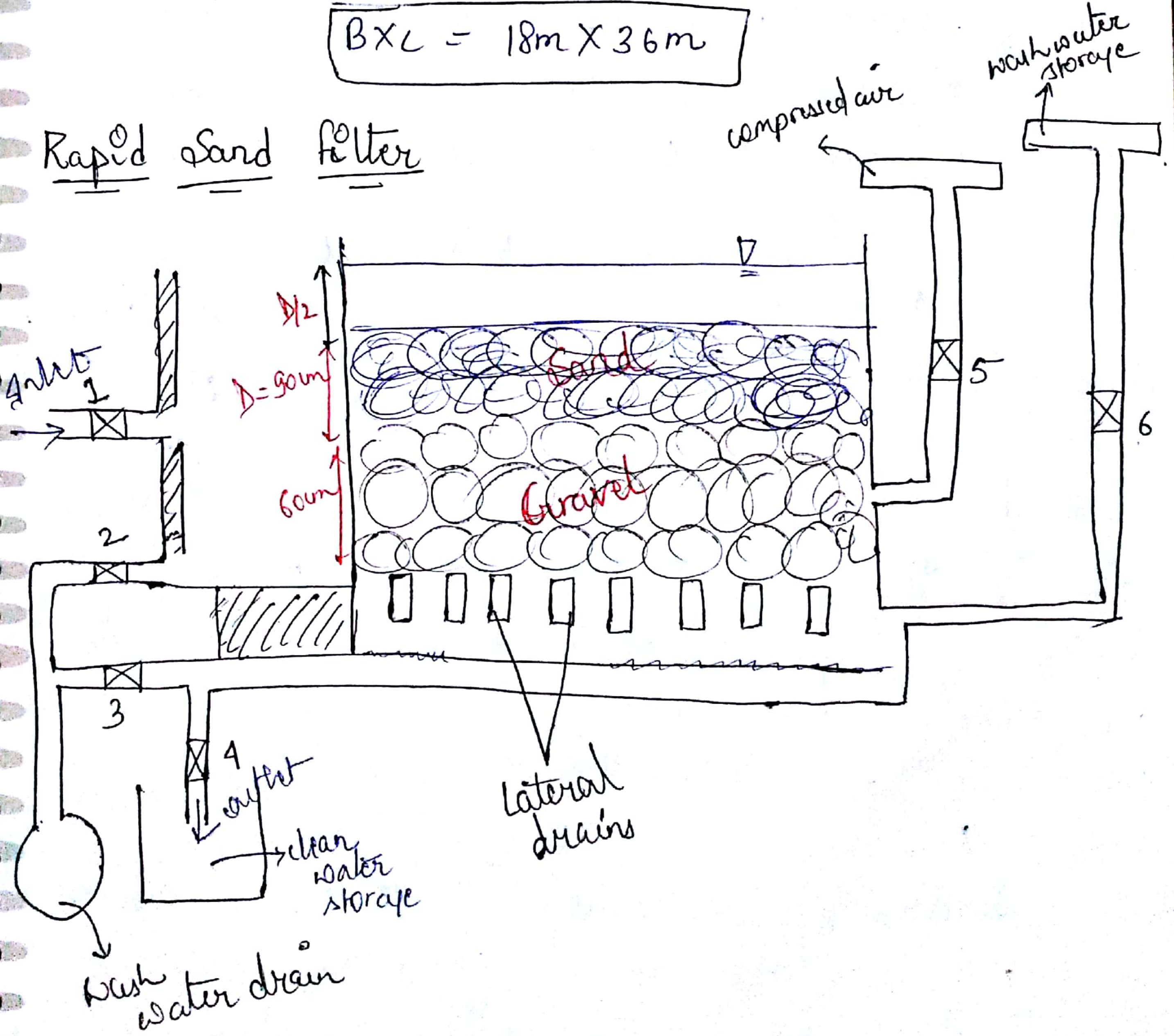
$2B^2 = 600$

B = 17.32 m

L = 34.64 m

$\boxed{B \times L = 18m \times 36m}$

Rapid Sand filter



⇒ The operation of Rapid sand filter is exactly same as that of slow sand filter

⇒ During operation of filter valve 1 & valve 4 are opened.

⇒ In case of rapid sand filter the size of particles of sand is more in comparison to slow sand filter. Hence the size of voids would be more comparitively so the impurities can penetrate deeper, in the bed (filter).

⇒ Hence in this case surface washing along is not sufficient & accompanied by backwashing.

⇒ In backwashing process valve 5, 6, 2 are opened & other remaining values are closed.

⇒ During this process compressed air & pressurised water is passed through a medium in upward direction, resulting in increase in porosity of filter medium thereby increasing opportunity of entrapped air. so the impurities are washed away with flowing pressurised water i.e finally ~~cora~~ collected in wash water basin/ trough.

⇒ After backwashing is completed valve 2, 5, 6 are closed only 1, 3 are opened by which filter is again loaded with raw water (from sedimentation tank) but this water is not used.

⇒ For further treatment (sufficient time is provided ~~by~~ for building up biological layer)
After building up ~~of~~ this layer valve 3 is closed & 4 is opened, Now the filter is return back to operational process & this filtered water is used for further treatment process.

⇒ The entire process of backwashing takes about 15 min.

⇒ The frequency of cleaning is 24-48 hrs

⇒ Amount of water required for backwashing is about 2% - 5% of total water filtered.

⇒ Rate of washing is 15-90 cm rise per minute (Normally taken as 45cm). The rate can be provided by a flow of 500 l/m²/min of bed area.

⇒ Area of tank is 10 - 1000 m²/unit

⇒ Min NO of unit is taken as 2 (operated) &
NO of units required as $n = 1.22\sqrt{Q}$ (only operational)
Q = plant capacity in MLD

⇒ Sand layer is 60 - 75cm deep & $D_{10} = 0.45 - 0.7$mm
Uniformity coefficient $C_u = \frac{D_{60}}{D_{10}} = 1.3 - 1.7$

⇒ Max head loss = 2.5m - 3m

⇒ Base material is Gravel & it is provided with 5-6 layers. Depth of gravel is 45cm - 60cm

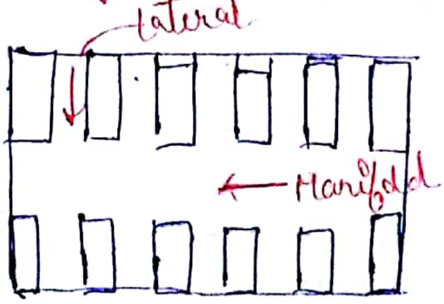
Gravel layer.
1 layer ⟶ 3-6mm
2 layer ⟶ 6-12mm
3 layer ⟶ 12-20mm
4 layer ⟶ 20-40mm

UNDER Drainage System For Rapid sand filter

$$\frac{\text{length of lateral}}{\text{Dia of lateral}} < 60$$

Rate of filteration ⇒ 3000-6000 l/m²/hr

Note The under drainage should be capable of passing wash water at high rates. We adopt Manifold & lateral system of under drainage.



Manifold is a pipe branching into several openings

~~In this cas~~

⇒ ~~In this ca~~

⇒ In this case two types of pipes & laterals are used —

(i) Perforated type
(ii) Strainer type

⇒ If dia of hole = 5mm, then spacing b/w holes = 8mm C/C

⇒ If dia of hole = 12mm, then spacing b/w holes = 20mm C/C

⇒ When perforated pipe is used the washing is called high velocity wash, In this case compressed air is not used.

⇒ If strainer is used washing is called slow velocity washing, in this case compressed air is used.

Size of Pipe

Total C/S area of perforations = 0.3% of filter area.

C/S area of ~~tot~~ each lateral = (2-4) times of C/S area of perforations

If hole diameter is 12mm then adopt 4 times or

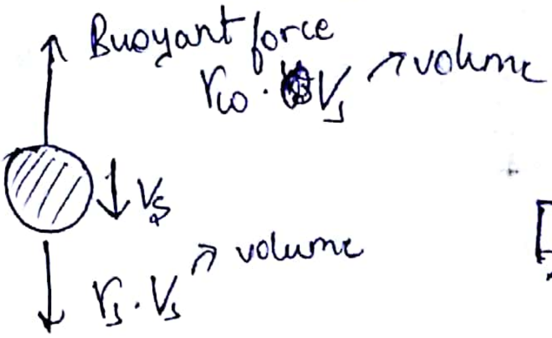
If hole diameter is 5mm then adopt 2 times

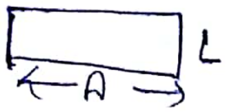
C/S area of Manifold = $\boxed{\text{2 times}}$ of C/S area of laterals

Max permissible velocity through manifold = $\boxed{1.8 - 2.4\ m/s}$

⇒ If depth of sand layer ~~is~~ is $\boxed{D}$ then top of wash water should be $\boxed{D/2}$ as shown in fig.

⇒ The rate of application of wash water should not be more than settling velocity of particle, the force created due to head loss must be equal to buoyant wt. of particle.





$V = A \times L$
↳ depth

Buoyant wt = $V_s(\gamma_s - \gamma_w)$

V_s = volume of solid
V = Volume of bed

$n = \frac{V_v}{V}$

$1-n = 1-\frac{V_v}{V}$

$$1-n = \frac{V - V_v}{V}$$

$$1-n = \frac{V_s}{V}$$

$$V_s = V(1-n)$$

By equating upward force & resultant downward wt

$$\gamma_w \cdot h \cdot A = V_s \cdot \gamma_{sub}$$

$$\gamma_w \cdot h \cdot A = V(1-n)(\gamma_s - \gamma_w)$$

$$h = \frac{V(1-n)(\gamma_s - \gamma_w)}{\gamma_w \cdot A}$$

$$h = \frac{\cancel{A} \times L(1-n)(\gamma_s - \gamma_w)}{\gamma_w \cdot \cancel{A}}$$

$$\boxed{h = \frac{L(1-n)(\gamma_s - \gamma_w)}{\gamma_w}}$$

For expanded length of bed.

$$\boxed{h = \frac{L_{ex}(1-n_{ex})(\gamma_s - \gamma_w)}{\gamma_w}}$$

L_{ex} = Depth of expanded bed

n_{ex} = porosity of expanded bed.

→ Porosity in the expanded condⁿ is function of backwash velocity & terminal flow velocity

Note Flow of water during filteration is laminar flow & in case of backwashing it is generally in Transition.

Hence $$\boxed{n_{ex} = \left(\frac{V_B}{V_t}\right)^{0.22}}$$

V_B = backwash velocity

V_t = Terminal settling velocity of particles

$$\boxed{V_t^2 = \frac{4}{3}\frac{(\rho_s-\rho_w)\,g\cdot d}{C_D \times \rho_w}}$$

where d = avg size of particle

C_D = Drag coeff. & it is function of Reynold's Number

laminar $$\boxed{C_D = \frac{24}{Re} \quad (Re < 1)}$$

laminar or turbulent. $$\boxed{C_D = \frac{24}{Re} + \frac{3}{\sqrt{Re}} + 0.34 \quad (Re > 1)}$$

Operational Trouble in Rapid sand filter

A. Air binding
B. Mud ball formation
C. Cracking of filter

A. Air binding

⇒ Initial head loss in filter is 15-30 cm & this headloss goes on increasing as more the impurities are entraped.

⇒ A stage comes when the frictional resistance by the medium exceeds the static head of the water over filter medium.

⇒ The bottom sand acts like a vaccum in this case, hence water gets suck in filter media without getting filter. Thus making bubbles ~~with~~ which stick to sand grains & this affects the operational of filter.

B. Mud Ball Formation

⇒ Mud from the atmosphere get deposited on the sand surface & during inadequate (impop washing, the mud may sink down in the bed this mud sticks to sand grains & other arrested impurities, therby forming a mudball.

⇒ This mudball may sink in gravel also, which interferes with the movement of water.

(c) Cracking of filter

⇒ Fine sand in top layer shrinks forming shrinkage cracks & these cracks are widen by pressure applied.

⇒ Thus impurities can penetrate deeper into the filter & reduces efficiency of filter.

Q Design approximate dimension of a set of two rapid gravity filters for treating water required for a population of 50,000 the rate of supply be 180 lpcd. The filters are rotated to work 5000 l/h/m². Assume max daily demand to be 1.8 times of avg demand.

Sol^n

Max daily demand = 1.8 × 180

= 324 lpcd

Water required for 50000 population in 1 day = 324 × 50000

= 162×10^5 LD

= $\frac{162 \times 10^5}{24}$ L

= 675000 L

अगर No of tank given नहीं है तो $n = 1.22\sqrt{Q}$ से निकालते

Total Area = $\frac{675000\,L}{5000\,L/m^2} = 135\,m^2$

~~L × B = 135~~

Area of 1 tank $= \frac{135}{2} = 67.5$

Taking $L : B = 2 : 1$

~~$B^2 = 67.5$~~

$L \times B = 67.5$

$2B^2 = 67.5$

$B = 5.80 m$

$L = 2 \times 5.80 = 11.61 m$

Q A rapid, sand filter is to be provided in a water treatment plant to process the water for town with a population of 2,75000, The water demand 200 lpcd the rate of filteration 15 $m^3/m^2/h$. Allow 5% of filtered water for storage to meet backwash requiremen Each backwashing period is of 30min determine No. of filters required allowing 1 as standby unit. The available surface area of configuration of filter unit is 10m × 4m Also include the upflow velocity & headloss to expand the bed to 0.66m. The porosity of bed is 0.5 specific gravity of particle 2.5. The avg particle size is 0.6mm Drag coefficient is 25.02. Kinematic viscosity $0.10136 \times 10^{-5} m^2/s$. The flow is assumed to be transitional flow.

Sol^n

$P = 2,75000$

Water demand = 200 lpcd

Rate of filteration = $15\,m^3/m^2/h$

Backwashing period = 30 min

water req for backwashing = 5% of filter water

No. of filter = operational + 1 standby

filter bed size = $(10 \times 4)\,m^2$

Depth of expanded Bed $L_{ex} = 0.66\,m$ $\nu = 0.10136 \times 10^{-5}\,m^2/s$

$n_e = 0.5$ $h_L = ?$

$G_s = 2.5$

$D_{10} = 0.6\,mm$ upflow velocity $V_B = ?$

$C_D = 25.02$

Max daily demand = 1.8×200
$= 360$ lpcd

Water required for 2,75000 population
$= 99 \times 10^6$ ~~MLD~~ LD
$= 99$ MLD

Adding water required for as backwashing is done $= 99 \times \frac{5}{100}$
$= 4.95$ MLD

Then Total water rq $= 99 + 4.95$
$= 103.95$ MLD

~~Total area = $\frac{103.95 \text{ MLD}}{15 \times 24}$~~

Now time for only filteration w/o backwashing = 24 hrs − 30

= 23.5 hr

$$\text{Total area} = \frac{103.95 \text{ MLD}}{15000 \times 23.5}$$

$= 294.89 \text{ m}^2$

Bed area = 10 × 4 = 40 m^2

~~For~~ No. of unit required in bed = $\frac{294.89}{40}$

$= 7.37 \simeq 8$

Operational = 8 + 1 standby

= 9

Now head loss (Not expanded) $= h = \frac{L(1-n)(\overset{\rho_s}{\gamma_s} - \overset{\rho}{\gamma_w})}{\gamma_w}$

L = 0.6 m for sand

$$= \frac{0.6(1-0.5)(2.65-1)}{1}$$

$h = 0.45 \text{ m}$

Now for n_{ex} is not given

so;

$V_s = V(1-n)$

$\left\{ \begin{array}{l} V_s' = V_s'(1-n_{ex}') \\ \text{final} \\ = A \times L_{ex}(1-n_{ex}) \end{array} \right.$

~~$A \times L(1-n) = A \times L_{ex}(1-n_{ex})$~~

$\cancel{A} \times L_{ex}(1-n_{ex}) = L \times \cancel{A}(1-n)$

As wt of solid/vol. of solid is equal in critical condn & expanded condn
V(1-n) = V'(1-n')

$$0.66(1-n_{ex}) = 0.6(1-0.5)$$

$$\boxed{n_{ex} = 0.54}$$ Porosity of expanded bed

Now head loss ~~Aex~~ for expanded $= \dfrac{L_{ex}(1-n_{ex})(\gamma_s-\gamma_w)}{\gamma_w}$

$$= \frac{0.66(1-0.54)(2.5-1)}{1}$$

$$h_L = 0.45\,m$$

$$V_t^2 = \frac{\frac{4}{3}(\rho_s-\rho_w)g\cdot d}{C_D \times \rho_w}$$

$$\cancel{= \frac{\frac{4}{3}(2.5-1)\times 9.81\times \frac{0.6}{10}}{25.02\times 1}}$$

$$V_t^2 = \frac{\frac{4}{3}(2500-1000)\times 9.81\times 0.6\times 10^{-3}}{25.02\times 1000}$$

$$\boxed{V_t = 0.022\,m/s}$$

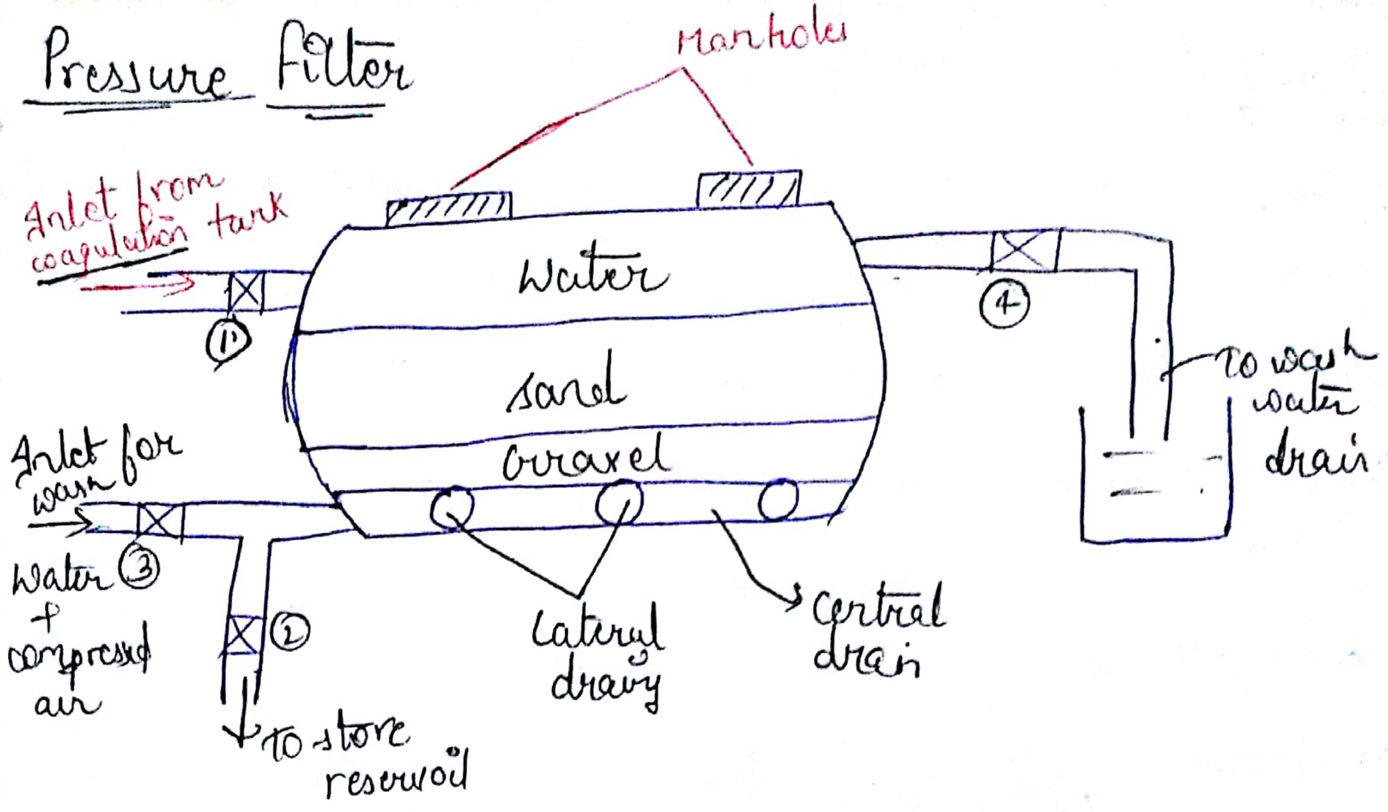
$$n_{ex} = \left(\frac{V_B}{V_t}\right)^{0.22}$$

$$0.54 = \left(\frac{V_B}{0.022}\right)^{0.22}$$

$$(0.54)^{\frac{1}{0.22}} = \frac{V_B}{0.022}$$

$$\boxed{V_B = 1.37\times 10^{-3}\,m/s}$$

Pressure Filter



⇒ The unit is like a rapid sand filter with a difference that complete unit is inside a closed chamber.

⇒ Flow of water through sand is not under gravity it occurs under pressure

⇒ Diameter of tank is 1.5m - 3m

⇒ Height of tank is 3.5m — 8m

⇒ It is operated like a rapid gravity filter

⇒ Rate of filteration 6000 l/m²/h – 15000 l/m²/h

⇒ It is used for clarifying water at industrial plants & for swimming pools.

Disinfection

⇒ It may be defined as the process of destruction or inactivation of harmful micro-organisms in water either by physical process or chemical process.

⇒ Physical process includes – heating, uv radiation etc & chemical process includes – addition of disinfectants such as chemicals.

⇒ Disinfection sterilize (कीटाणु रहित) the water. In sterilization process all organisms are killed.

⇒ Disinfection is min essential treatment required for any drinking water & it is the final process in the chain of water purification process.

Methods of Disinfection

- → Physical
  - → Boiling
  - → UV rays
- → chemical
  - → oxidising agents (Cl_2, Br, I, O_3, $KMnO_4$ etc)
  - → Metal ions (Ag, Cu)
  - → Alkalis & acids (Detrimental to bacteria)

$pH > 11$
$pH < 3$ } Bacteria can't survive

Out of various M/d chlorination is most ~~comm~~ commonly used.

Minor Methods

(1) Treatment with excess lime

⇒ Excess lime kills bacteria but excess lime has to be removed before supply, for removal of excess lime recarbonation process is used.

Note → In recarbonation process CO_2 bubbles are introduced to lower the pH value. ~~It~~

⇒ It is not used Now a days.

(2) Treatment with Ozone

$$O_3 \longrightarrow O_2 + O$$

↳ Nascent oxygen

⇒ Nascent oxygen is a powerful oxidising agent & it removes organic matter as well as bacteria.

⇒ 2-3ppm ozone is require to maintain a residue of 0.1ppm.

⇒ Residual ozone is measured by orthotolidine test.

⇒ Ozone is unstable hence Nothing remains in water by the time it reaches the distribution system.

⇒ Ozone removes colour, taste & odour also & gives pleasant taste to water.

⇒ It does not ensure safety against future - ~~recon~~ recontamination.

⇒ It is less efficient than chlorine in killing of bacteria.

3) Treatment with ~~B~~ 'F' & 'Br'

⇒ They are available in the form of pills.

4) Treatment with $KMnO_4$

⇒ It is popular disinfectant for wells water supply in village which are contaminated with less amount of bacterias.

⇒ Besides killing bacteria it also helps in oxidising the taste ~~prod~~ producing organic matter. Hence it is sometimes added to filter & chlorinated water.

⇒ $KMnO_4$ is mixed in water in a bucket & added to well. Pink colour forms bcoz of this. If pink colour disappears it indicates presence of bacteria, hence more $KMnO_4$ is added until the pink colour stands. The well is not used for next 48 hrs.

⇒ Dose is Normally - 1-2 mg/l & contact period is 4-6 hrs

⇒ It removes about 98% bacterias & 100% bacteria causing cholera. (हैजा)

Major Methods

1) Chlorination

$$Cl_2 + H_2O \xrightleftharpoons{pH>5} HOCl + HCl$$

↓
Hypochlorous acid

$$HOCl \xrightleftharpoons{pH>8} OCl^- + H^+$$

↓
Hypochlorite ion.

Note At $pH<5$ chlorine does not react with water & remains as free chlorine.

($HOCl$, OCl^-, Cl_2) are combinely called free available ~~&~~ chlorine
↓ Most dangerous

⇒ Out of these 3 forms $HOCl$ is more destructive

⇒ It is 80% ~~the~~ more effective than hypochlorite ion hence pH of water should be maintain near by 7 or slightly less than 7

⇒ Chlorine reacts with ammonia immediately & forms chlora amine.

$$Cl_2 + NH_3 \xrightleftharpoons{pH>7.5} NH_2Cl + HCl$$

Monochloroamine

$$NH_2Cl + Cl_2 \xrightleftharpoons{pH-5\text{ to }6.5} NHCl_2 + HCl$$

dichloroamine

$$NHCl_2 + Cl_2 \xrightleftharpoons{pH < 4.4} \underline{NCl_3} + HCl$$

Nitrogen trichloride

⇒ ~~Flora~~ Chloro-amines are combined form of chlorine & it is less effective than free chlorine (about 25 times lesser). But they are stable & remains for greater time in water

⇒ In the usual chlorine treatment in which pH is slightly less than 7, dichloraamine predominates.

⇒ Doses of chlorine should be sufficient so as to leave a residue of 0.2mg/l after 10min of constant period.

⇒ These disinfectant kill those enzymes which are ~~essn~~ essential for metabolism.

⇒ The residual chlorine is tested by DPD test (Diethyl paraphenylene-diamine)

⇒ Forms in which chlorine is added-

a) as free clorine (Gas or liquid)

b) Hypochlorites (Bleaching powder)

c) chloramines (Ammonia + chlorine)

d) chlorine dioxide (ClO_2

a) Free chlorine (Gas or liquid)

⇒ Chlorine is generally added in liquid form to disinfection of water

⇒ Chlorine is highly temperature ~~susc~~ susceptable
If temp $< 10^\circ C$, chlorine will get frozen.

⇒ Liquid cylinder is kept as $32-48^\circ C$. अत does not form any sludge.

Note 1) steel cylinder burns in di-chlorine at a temp. greater than $92^\circ C$ hence high. temp. should be avoided.

2) chlorine forms explosive mixture with carbon monoxide.

⇒ Free chlorine can be stored w/o getting deterorated.
(नष्ट होना)

⇒ chlorine dose can be easily measured in the liquid form. Hence underloading (underdose) & overloading are less frequent.

⇒ Free chlorine is powerful disinfectant & remains for a long time in water so provide safeguard against future recontamination of water.

b) Hypochlorites or Bleaching powder

$$Ca(OCl_2)_2 \rightleftharpoons Ca + 2OCl^-$$

$$OCl^- \underset{pH > 8}{\overset{pH < 7}{\rightleftharpoons}} HOCl$$

⇒ Hypochlorites are generally not used. Now a days because they increase pH values, because they have lime content.

⇒ ~~Hypoll~~ Hypochlorites contain very low amount of chlorine it is used for swimming pools only.

c) Use of chloraamines

⇒ Chlora·amines are weaker disinfectants hence high dose is used.

⇒ chloramines are stable & remain in water for long period, hence they provide greater safeguard against future ~~reco~~ pollution.

⇒ They are weaker than free chlorine but do not impart bad taste, if left at residue.

⇒ when phenol is present in water, chloraamines are mostly used because chlorine with phenol gives bad taste, but chloraamine with phenol does not give any taste.

d) Chlorine dioxide

⇒ It is highly effective (about 2.5 times more effective than free chlorine).

⇒ It may also be used when phenol is present & it oxidises organic matter also, pH range is 8-10.

⇒ Normal dose 0.5-1.5 mg/l

Types of chlorination

1) Plain chlorination

⇒ Only chlorination, & no other treatment is given to water. Plain chlorination is the application of chlorine to plain or raw water as it enters the distribution system.

⇒ It removes bacteria, organic matter & color.

⇒ It is used to clean water if turbidity is b/w 20-30 mg/l.

⇒ Dose is given 0.5 mg/l

2) Pre-chlorination

⇒ It is application of water before treatment especially before filteration.

⇒ Doses required are such that 0.1-0.5 mg/l comes to filter.

⇒ Normal dose is 5-10mg/l

⇒ Pre-chlorination should always be followed by post-chlorination.

⇒ Pre-chlorination has following advantages :-

a) It reduces the quantity of coagulants required

b) It reduces bacterial load on filter

c) It helps in maintaining long runs of filter.

d) It controls algae growth in basin & filter

e) It eliminates taste & odour.

3) Post chlorination

⇒ It is application of chlorine after the treatment of water.

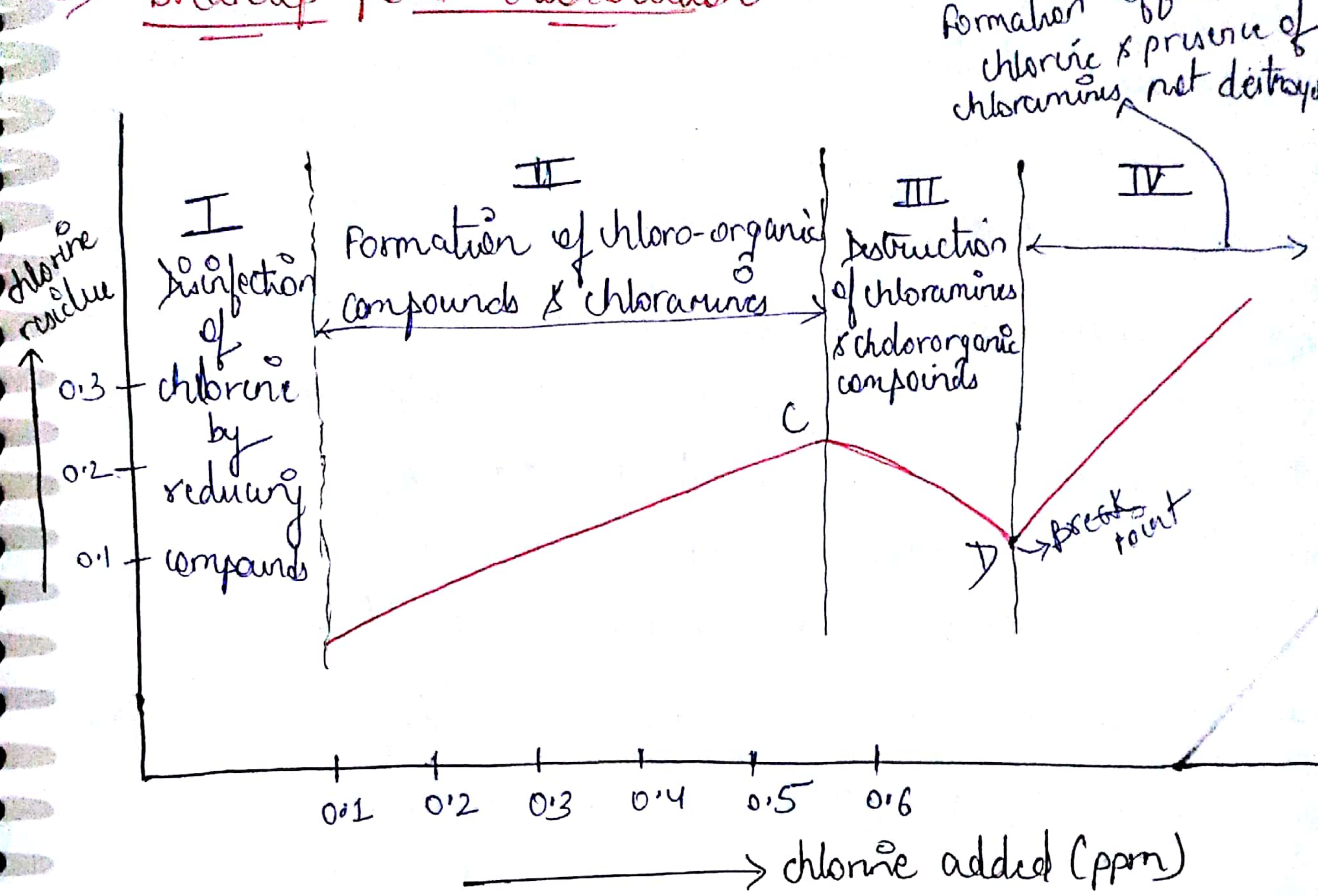
⇒ Dose should be such that 0.1 - 0.2 mg/l should be left after contact period of 20 min.

⇒ It is useful for protection against contamination from cross connections.

4) Double or multiple chlorination

⇒ Pre-chlorination & Post chlorination combinedly called double chlorination.

5) Breakup point chlorination



⇒ During disinfection process amount of residual chlorine is less in the begning (Stage I), during which Iron, Nitrite etc are oxidise.

⇒ In stage II chloramines & combined residual chlorines forms, combined residual chlorine will increase as ~~dg~~ demand for disinfection is satisfied.

⇒ Actually residual of combined chlorine is tested by **DPD Test**, which measures both combine & free chlorine.

⇒ Amount of residual should be slightly less than applied chlorine because some bacteria killing takes place.

⇒ At point C bad smell comes out, it is due to the fact that decomposition or oxidation of organic matter starts at point C, hence chlorine residual decreases.

⇒ In stage III chlorine breaks down into Nitrogen compounds, chloro-organic compounds are also destructed & at point D that bad smells dissappears. This implies that oxidation of organic matter is complete at point D.

⇒ Any further chlorine addition appears as free chlorine. Thus point D is called Break point.

⇒ In general chlorine is added beyond break point to ensure a residual of 0.2 – 0.3 mg/l of chlorine. It helps to prevent against future recontamination.

Note The difference b/w chlorine added & chlorine residue is called chlorine demand.

6) Super-chlorination

When excess chlorine (5-15 mg/l) is added in water & residue is 1 - 2mg/l beyond break point is called super chlorination.

7) Dechlorination

when chlorine residue is high the excess chlorine is removed by dechlorinating agents (sodium thiosulphate, activated carbon, sodium bisulphate, sodium sulphate, potassium permang sulphur dioxide), sodium disulphide etc).

Testing of chlorine residue

(i) orthotolidene test
(ii) Starch iodine test
(iii) DPD test.

(i) Orthotolidene test

Orthotolidene is colorless organic liquid that is oxidised by chlorine into a yellow ~~to~~ color compound ~~(~~ (Hologuinone)

orthotolidene + chlorine → Hologuinone (Yellow)

⇒ 10ml of chlorinated water sample is with 0.1 ml of orthtolidene

⇒ ~~the~~ color form is noted after 5min.

⇒ Color will be yellow (Free chlorine + Combined chlorine)

⇒ By ~~combin~~ comparing this color with color of non concⁿ of chlorine amount of residual is obtained.

Scanned by CamScanner

Note - The itg. of yellow color (intensity) will be directly proportional to amount of residue.

⇒ If free & combined chlorine are found seperately color form in 5min & 5 sec are noted down.

Color formed in 5 sec $\longrightarrow R_1$

Color " " 5 min $\longrightarrow R_2$

$R_1 \longrightarrow$ Yellow (Free chlorine)

$R_2 \longrightarrow$ Yellow (Free + combined chlorine)

Note → If the water is highly alkaline Blue ~~tinge~~color is formed instead of yellow. In that case quantity of orthotolidene is doubled.

⇒ In orthotolidenetest presence of iron, mangnese, nitrites etc will give erronious (गलत) result. For such water ~~ortholi~~ orthotolidene arsenite test is done.

⇒ In this case sodium arsenite is added into chlorinated water this will dechlorinate the sample.

⇒ To this dechlorinated sample orthotolidine soln is added & color form is noted down (say R_1 → Mn, Iron, nitrites etc)

⇒ Another ~~water~~ sample of chlorinated water is added with orthotolidine & colour form after 5sec & 5min are noted down say R_2 & R_3 respectively.

R_1 → Magnese, Iron, Nitrite etc

R_2 → Freechlorine $+R_1$

R_3 → Free + combined chlorine $+R_1$

⇒ This test is not perform. Nowadays it has found to cause cancer.

(ii) Starch Iodide Test

⇒ Starch iodide test is also called Iodometric m/d. It is used when ~~pro~~ magnese, nitrite etc are present which makes orthotolidine test unsuitable.

⇒ This test is more presise than orthotolidine test particularly when chlorine residue is more than 1ppm.

1L water + 10ml (KI) + 5ml starch ⟶ Blue colour is formed.

⇒ To remove this blue color 0.001N sodium thiosulphate is added & the quantity of sodium thiosulphate required to remove this color will be directly proportional to amount of chlorine residue.

Cl_2 in ppm = 0.3546 × ml of sodium thiosulphate

(iii) DPD Test

⇒ It is also a color ~~mated~~ matching technique developed by BDH (British Drug house)

Factors affecting efficiency of chlorination

(i) Turbidity

⇒ More is the turbidity lesser will be the efficiency of chlorination.

⇒ The effect of turbidity in water is to make it difficult to obtain residual chlorine, also the penetration of chlorine hence it disturbs destruction of bacterias.

(ii) Presence of Metallic Compounds (Fe^{2+}, Mn^{2+})

⇒ The presence of metallic compounds such as iron & magnese in soln in the water utilizes more amount of chlorine, to get oxidize, hence ~~@~~ its is essential to remove iron & ~~Mg~~ Mn before chlorination to increase bacterial removal efficiency of chlorine.

(iii) Amonia

⇒ chlorine reacts with ammonia & forms chlora-amine which is less effective.

(iv) pH

⇒ chlorination is pH dependent process if pH<5 chlorine will not react with water & remains as free chlorine

⇒ At pH nearby 7 Hypochlorous acid (HOCl) forms which is most destructive

⇒ If pH>8 ~~OC~~ Hypochloride ion (OCl⁻) forms which is less destructive then HOCl.

(v) Temperature

⇒ As temp decreases efficiency of chlorine reduced

⇒ 28°– 32°C is eff. range of temp. for better efficient chlorination.

①

Water softening

Water is said to be hard when it contains relatively large amount of bicarbonate, carbonate, sulphates, chlorides etc of Ca & Mg.

Removal of Temporary Hardness

Temporary hardness of Carbonate & Bicarbonate is removed by simple ~~b~~ boiling & by adding lime.

(i) By boiling

$$Ca(HCO_3)_2 \xrightarrow{\Delta} CaCO_3 \downarrow + H_2O + CO_2 \uparrow$$

$$Mg(HCO_3)_2 \xrightarrow{\Delta} MgCO_3 \downarrow + H_2O + CO_2 \uparrow$$

The boiling does not remove permanent hardness

(ii) By adding lime

$$MgCO_3 + \underset{\text{hydrated lime}}{Ca(OH)_2} \longrightarrow Mg(OH)_2 \downarrow + CaCO_3 \downarrow$$

$$Mg(HCO_3)_2 + Ca(OH)_2 \rightarrow Ca(HCO_3)_2 + Mg(OH)_2$$

$$Ca(HCO_3)_2 + Ca(OH)_2 \rightarrow 2CaCO_3 \downarrow + 2H_2O$$

1 mole of $MgCO_3$ requires 1 mole of hydrated ~~lime~~ lime.

Removal of permanent hardness

(i) Lime-soda process
(ii) Base exchange (Zeolite process)
(iii) Demineralization

(i) Lime soda process

Addition of lime reduces only the bicarbonate hardness & by adding of soda / soda ash (Na_2CO_3) both temporary & permanent hardness can be remove.

$$Ca(HCO_3)_2 + Ca(OH)_2 \rightarrow 2CaCO_3 \downarrow + 2H_2O$$

$$Mg(HCO_3)_2 + Ca(OH)_2 \longrightarrow Ca(HCO_3) + Mg(OH)_2 \downarrow$$

$$Ca(HCO_3) + Ca(OH)_2 \rightarrow 2CaCO_3 + 2H_2O$$

$$Mg\left[\begin{array}{l} 2Cl^- \\ SO_4^{2-} \\ 2NO_3^- \end{array}\right. + Ca(OH)_2 \longrightarrow Mg(OH)_2\downarrow + Ca^{2+}\left[\begin{array}{l} 2Cl^- \\ SO_4^{2-} \\ 2NO_3^- \end{array}\right.$$

$$CaCl_2 + Na_2CO_3 \longrightarrow CaCO_3\downarrow + 2NaCl$$

$$CaSO_4 + Na_2CO_3 \rightarrow CaCO_3\downarrow + Na_2SO_4$$

$$Ca(NO_3)_2 + Na_2CO_3 \rightarrow CaCO_3\downarrow + 2NaNO_3$$

also remove CO_2

$$CO_2 + Ca(OH)_2 \longrightarrow CaCO_3 + H_2O$$

⇒ lime removes entire carbonate hardness.

⇒ ~~Permanent~~ lime reacts with Non-carbonate (permanent Hardness & converts it to Non-carbonate Hardness of calcium.

⇒ Non-carbonate hardness of calcium is finally removed by soda ash.

⇒ lime also helps in removing carbon dioxide from the system, if conc^n of CO_2 is more it will consume more lime. So lime available to remove hardness is less. Thus if CO_2 conc^n is more than 10mg/l we remove CO_2 first by aeration before adding lime.

Note: Optimum pH for $CaCO_3$ ppt by lime addition is 9 – 9.5. & for $Mg(OH)_2$ ppt the pH requires is 11.

Complete ~~rero~~ removal of hardness cannot be acheive by chemical precipitation.

⇒ Under Normal condition in treatment plant 40 mg/l of $CaCO_3$ & 10 mg/l of $Mg(OH)_2$ remains in soften water.

⇒ These remains will ppt. slowly & get accumulated inside, hence it is necessary to make soluble & this process is done by <u>re-carbonation</u>.

वापस Hardness में change कर दिया

परत जमा जाएगी जो choking कर देती है।

$$CaCO_3 + CO_2 + H_2O \longrightarrow \underset{\text{soluble}}{Ca(HCO_3)_2}$$

को Heat करके remove कर दिया जाएगा

$$Mg(OH)_2 + CO_2 \longrightarrow MgCO_3 + H_2O$$

$$MgCO_3 + CO_2 + H_2O \longrightarrow Mg(HCO_3)_2 \cancel{+ H_2O}$$

⇒ This process can also be perform by acids

$$CaCO_3 + H_2SO_4 \longrightarrow 2Ca(HCO_3) + SO_4^{2-}$$

$$Mg(OH)_2 + H_2SO_4 \rightarrow \underline{MgSO_4} + H_2O$$

acid is permanent hardness पैदा हो जाती है जो रहती है।

⇒ when lime soda is added with alum less quantity of coagulant is acquired.

⇒ lime soda process reduces the corrosion of pipe due to increase in alkalinity. It also helps in removing smaller quantity of iron & magnese but it form ~~[illegible]~~ huge quantity of sludg

⇒ 1mg/l of Mg. Hardness (as ~~[illegible]~~ $CaCO_3$) remove, will produces ~~0.58g~~ 0.58mg/l of $Mg(OH)_2$.

पहले करा हुआ है (~~[illegible]~~ gram eq.

$$\underbrace{CaCO_3}_{100} \longrightarrow \underset{58}{\underline{Mg(OH)_2}}$$

⇒ 1mg/l of added of lime (as $CaCO_3$) will produce 1mg/l of $CaCO_3$

⇒ 1mg/l of Calcium hardness (as $CaCO_3$) remove, will produce 1mg/l of $CaCO_3$

⇒ Dry sludge of ~~remove~~ produces mg/l = lime added 1mg/l + Calcium removed + 0.58 of $Mg(OH)_2$

⇒ lime soda remove hardness to about 50mg/l (as $CaCO_3$)

⇒ 50-200 mg/l hardness can be left for consumer.

⇒ Any hardness above 200 mg/l requires water softening.

(ii) Base exchange process / Zeolite Process / Cation exchange process

⑥

⇒ The lime soda m/d is essentially a ~~pre~~ precipitation m/d of water softening

⇒ In the ~~zeo~~ zeolite process no such chemicals are added to water.

⇒ In this process water is passed through a bed of special material called zeolite which has ~~pro~~ ability to remove Ca & Mg from the water.

⇒ Zeolite is natural or synthetic cation or base exchange ~~A~~ hydrated silicates of sodium & aluminium which is known as Green sand.

$$\boxed{\text{Zeolite} \rightarrow Na_2O \cdot Al_2O_3 \cdot xSiO_3\, y \cdot H_2O} \text{ (Green sand)}$$

$$2SiO_2 \cdot Al_2O_3\, Na_2O + Ca(HCO_3)_2 \longrightarrow 2SiO_2 \cdot Al_2O_3 \cdot CaO + 2NaHCO_3$$

⇒ Thus water will have zero hardness. No sludge is form.

⇒ No problem of incrustation (choking) of pipe as in lime soda process.

(iii) Demineralisation Process

⇒ This process removes minerals in water, the complete removal of water is done ~~B~~ by first passing through cation exchange resins &

through anion exchange resins.

Minor Treatment

(i) Activated Carbon

⇒ Due to adsorption property it removes taste, color & odour.

⇒ It removes phenol type impurity.

⇒ It is added before or after coagulation but before filteration.

⇒ In this M/d a ~~per~~ part of activated carbon is mixed in mixing tank & remaining is mixed before it enters into filter.

⇒ Usual dose 5-20mg/l.

⇒ when used with coagulants it aids (help) the coagulation.

⇒ It reduces chlorine demand of water.

⇒ It also remove organic mater & its overdose is not ~~hard~~ harmful.

(ii) Treatment with Copper sulphate ($CuSO_4 \cdot 7H_2O$)

⇒ When added in reservoirs it controls the growth of algaes. It also helps in removing taste, color & odour.

⇒ Usual dose is 0.5-0.75mg/l

(iii) Deflouridation

⇒ ~~(a) Adsorption by activated carbon.~~

~~(b) Adsoption by activated alumina~~ (prashanti Techniqu

a) Adsorption of activated aluminium or activated carbon (Prashanti Technique)

→ In this m/d raw water containing high 'conc^n' of flouride is passed through insoluble granular bed of activated alumina or activated carbon.
So the flouride particles are adsobed by these beds.

⇒ Activated alumina is an ~~ex~~ excellent medium in removing flouride than any other medium.

→ This adsorption is best carried out in slightly acidic medium (pH → 5-7)

b) Nalgonda Technique

⇒ Mainly used for rural areas where ground water containing excess flouride.

→ This technique uses aluminium salt, for removing flourides.

⇒ The raw water ~~is~~ is first mixed by lime or soda.

→ Alum sol^n is then added & water is stirred slowly for about 10 min & allow to settle for about 1 hr, the ppt sludge is removed.

$$\boxed{\text{Water} + \text{lime} / Na_2CO_3} + \text{Alum} \longrightarrow \text{sludge}$$

c) Ion exchange adsoption method

⇒ This process is almost similar to is that use for removing hardness as zeolite process.

d) Reverse Osmosis

⇒ In this m/d the water is passed through a semi-permeable membrane which permits the flow of clean water only & block the flow of salts including flourides.

(iv) Desalination

⇒ It is done by reverse osmosis process.

⇒ It is done by electro-dialysis.

Q An analysis of a surface water sample gave following units:-

Ca ⟶ 70 mg/l, Mg/l → 50 mg/l Na → 10 mg/l

Bicarbonate → 300 mg/l, sulphate - 135 mg/l, chloride - 7 mg/l

a) Calc. No of mili equivalent / litre of each substance

b) Total hardness, carbonate & Non-carbonate hardness & alkalinity in the form $CaCO_3$.

Q Calc. the amount of hydrated lime & soda for treating 50000 l of water per day if water contains following impurities.

$CaCO_3$ → 280 ppm, $MgCl_2$ → 138 ppm, $Mg(HCO_3)_2$ → 110 ppm

$MgSO_4$ - 80 ppm, $NaCl$ → 35 ppm, Fe_2O_3 → 55 ppm

$CaCO_3$ → 110 ppm, SiO_2 → 40 ppm.

Atomic wt $Ca - 40$, $Mg = 24$, $S = 32$, $Cl = 35.5$

⑩ $O = 16$, $Na = 23$, $Fe = 56$, $Si = 26$

Sol^n Gram equivalent / milieq. litre

$Ca^{+2} = \frac{\text{wt. in g}}{\text{eq. wt}} = \frac{70}{40/2} = 3.5$

$Mg^{+2} = \frac{50}{24/2} = 4.16$, $Na^{+} = \frac{10}{23/1} = 0.43$

$HCO_3^{-} = \frac{300}{(1+12+48)/1} = 4.91$, $Cl^{-} = \frac{7}{35.5/1} = 0.197$, $SO_4^{2-} = \frac{135}{(32+64)/2}$

$= 2.81$

Temporary ~~Total~~ hardness $= [Ca^{2+} + Mg^{2+}] \times 50$

$= [3.5 + 4.16] \times 50$

Temph. H $= 383$

Alkalinity $= 4.91 \times 50 = 245.5$

P. H $= 2.81 \times 50 = 140.5$

Total Hardness $= 523.5$

CH-4

Distribution System

Methods of Distribution

⇒ Water is forced in the distributions system by following ways—

1) Gravitational system
2) Direct Pumping
3) Combined system.

1) Gravitational System

⇒ In this system water from higher level source is supplied by simple action of gravity without pumping.

⇒ This system works well where lakes are located at the top of hill.

2) Direct Pumping

⇒ In this system treated water instead of pumping to the service or distribution reservoir, is pumped directly to distribution system.

⇒ Since water demand vary pumps are required to be run, such that sufficient water demand is made

⇒ Due to variable speed pump do not work with their max. efficiency so this system is uneconomical.

3) Combined system

⇒ In this system of water supply pumping & gravitational system are combined.

⇒ The treated water is stored in elevated reservoir & fed to the distribution system by the acc^n of gravity.

⇒ Pumping works at constant & convenient schedule & pressure can be maintain uniform for the supply.

System of Supply

1) Continuous supply
2) Intermittent supply

~~1) Continuous Supply~~

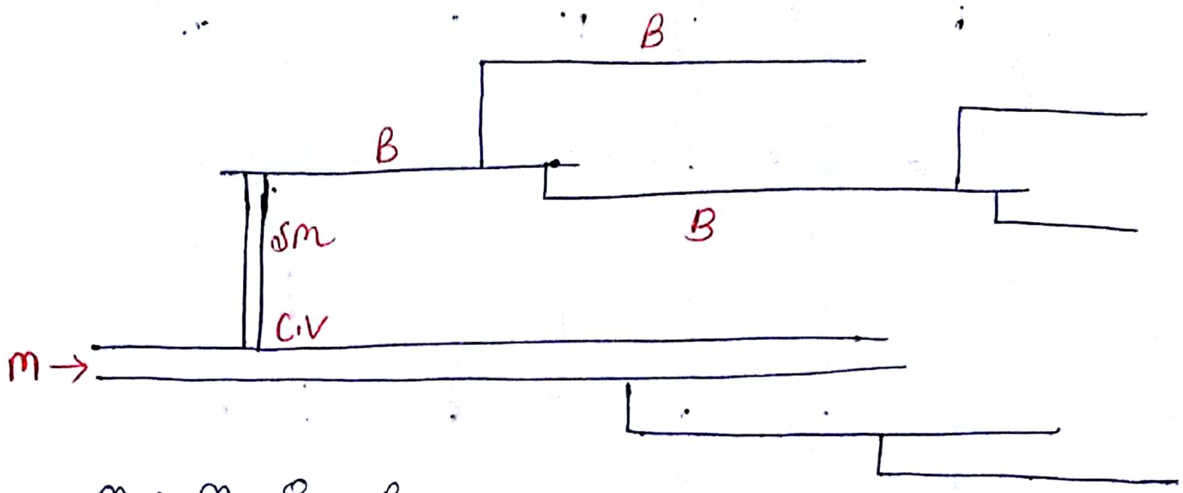
⇒ In continuous supply water is supplied continuously to the consumer.

⇒ In intermittent supply water is supplied mostly at peak hours.

⇒ Or if the ~~storat~~ storage of water is there, then whole distribution area is divided into different zones & water is supplied to each zone at different fix timings.

Layout of Distribution system

1) Dead End System / Tree system



M → Main pipe
SM → Submain pipe
CV → cutoff valve
B → Branch pipes / Laterals

⇒ The system has 1 main pipe, from which a no. of submain pipe & branch pipes are seperated.
↳ lateral

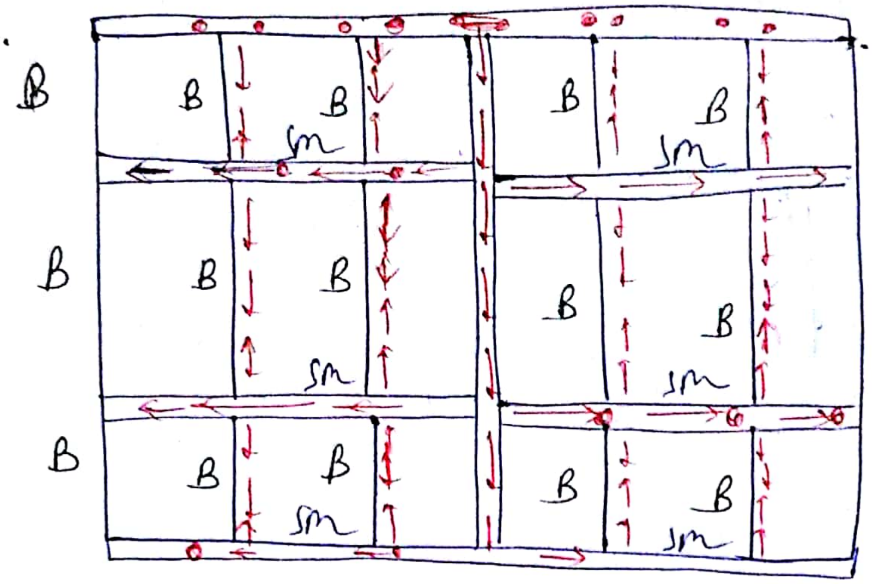
⇒ From laterals house connections are given to different houses.

obj
⇒ This type of distribution system is followed for old cities.

⇒ System is easy to design & is cheap & simple. but there are some drawbacks,

⇒ water can reach at a particular point by one root hence if some fault creep in water supply disturbed in that area bcoz water conveyance is unidirection only.

2) Grid system / Reticular / Interlaced system



M = Main Pipe
SM = Submains pipe
B = Branch
● = Scour valves

⇒ The system is also known as reticular system

⇒ In this system one main pipe runs through center & branches & laterals run in grid pattern, which are interconnected.

⇒ Since mains & laterals are interconnected so dead end is eliminated & water reaching at different location from more than 1 root.

⇒ Since pipeline get water from different direction design is quite difficult

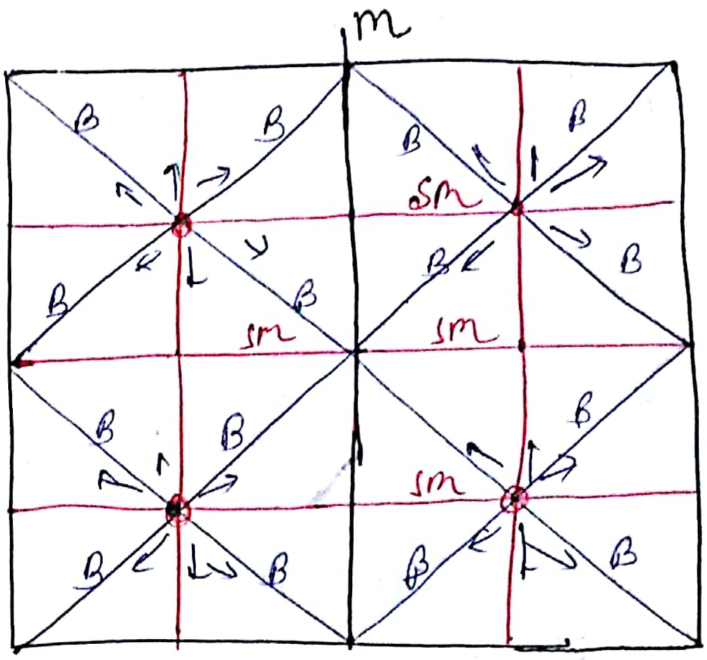
⇒ Size of pipes are larger & ~~no of~~ more No' of valves are required. The system is most suitable for planned cities where roads & streets are in Grid pattern.

Disadvantages

1) This system requires more length of pipeline & more No. of valves

2) Design is difficult & costlier

3) Ring system / Circular system



B = Branch or laterals

● = distributi reservoir

sm = submain pipe

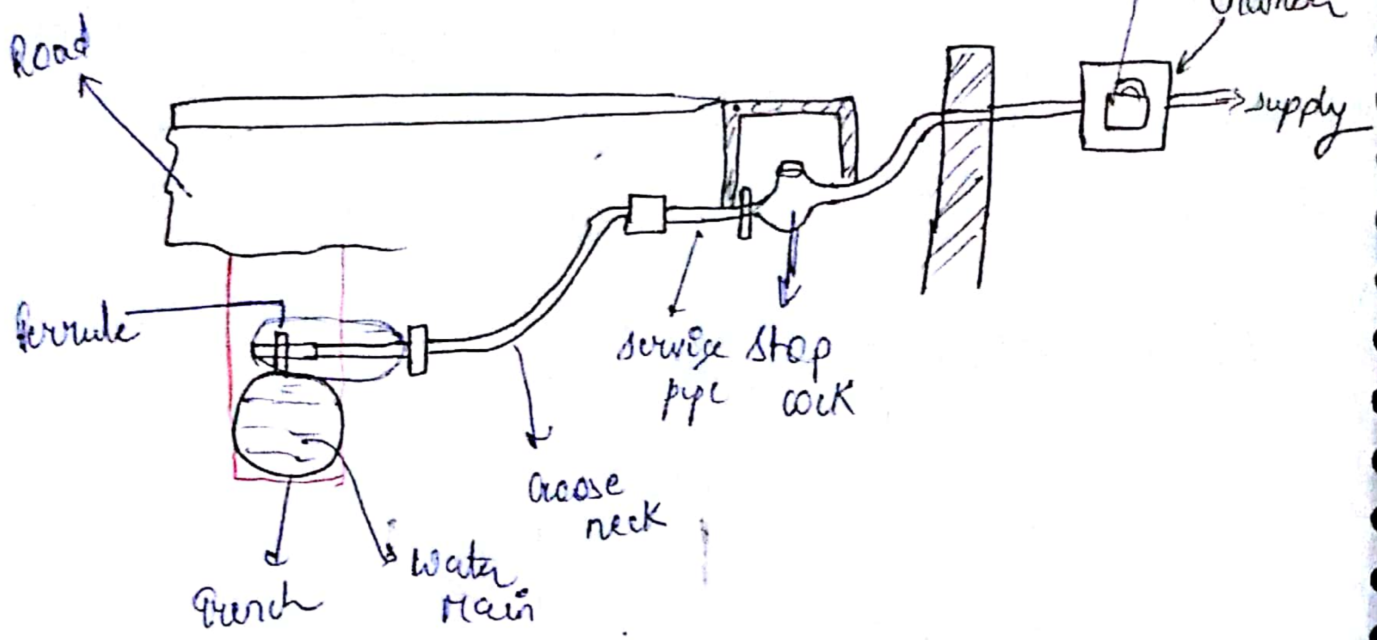
M = Main

⇒ The system consist of main pipe all around syste

4) Radial system

⇒ A very big area is divided into several zones & at center of each zone, a distribution reservoir is kept.

Appurtenances in Plumbing system



(i) Ferrule

⇒ It is right angle ~~Step~~ sleeved made of non-ferrous metal mostly of brass or gun metal.

⇒ It is joint to the opening drill in the water main with the plug.

⇒ Its size usually varies with 10-~~15~~50 mm dia

(ii) Goose Neck

⇒ It is a small curve flexible pipe for making connection b/w ferrule & service pipe.

⇒ They are generally about 75 cm length & made up of flexible material.

⇒ It provides ~~eath~~ ease in connecting service pipe with ferrule.

(iii) Service pipe

⇒ It is galvanised iron pipe of Nominal size (less than 50mm).

⇒ It is laid below ground level making a trench.

⇒ It is connected to main pipe through goose ~~and~~ neck & ferrule.

(iv) Stop cock

⇒ It is provided before the watermeter, if watermeter is provided.

(v) Water meter

⇒ Water meter are connected through stop cock to measure the quantity of water.

Part - 2

-Nitish Sir

(Mayank Singh)

Sewage

- Ch-5 - Waste water charac.
- Ch-6 - Bio-chemical reatⁿ
- Ch-7 - Disposal of sewage effluent
- Ch-8 - Design of sewer system
- Ch-9 - Sewage Treatment

Ch-5 Waste water characteristics

⇒ Waste water are usually classified as — Industrial waste water & Municipal waste water (Domestic).

⇒ Industrial waste water with characteristics ~~expt~~ compatible to municipal waste water is often discharge to the municipal sewers.

⇒ Many Industrial waste water requires pre-treatment to remove non-compatible substances prior to discharge into municipal sewer.

⇒ Water collected in municipal waste water system contains a wide range of ~~c~~ contaminants.

⇒ commonly found contaminants —

Contaminant	Source	Environmental significance
1) Suspended solids	Domestic use, Industrial wastes	Causes sludge deposition & anaerobic reatⁿ in environment.
2) Biodegradable organics	Domestic use, industrial waste	Cause biological degradation

3> Pathogenes	Domestic waste	Transmit communicable diseases
4) Nutrients	Domestic & industrial water	Cause Eutrophication

Physical Characterstics

1) Turbidity

Waste water is Normally turbid containing waste from bath, Kitchen, peices of paper, grease, vegetable debris etc.

2.) Colour

⇒ The color of waste water can be detected by naked eyes & it refers to the age of waste-water.

⇒ Imp (or obj) Fresh waste water is of usually Grey or light brown color. However as organic compound are broken down by bacteria the dissolved oxygen in waste water was reduce to zero & colour of waste water is converts to blackish. This cond^n of waste water is called septic or stable

⇒ Some industrial waste water can also impart color to sewage.

3) Odour

Odour in waste water is caused by dissolved gases produced by decomposition of organic matters.

4) Temperature

The avg. temperature of sewage in India is $\boxed{20^\circ C.}$ which is near about the ideal temp for Biological activities.

Chemical characteristics

1) Total solids, suspended solids, settalable solids.
2) pH value
3) chloride content
4) Nitrogen content
5) Presence of fat, grease & oil.
6) Sulphids, sulphates & H_2S gas
7) Dissolved oxygen
8) Chemical Oxygen demand (COD)
9) Bio-logical Oxygen demand (BOD)
10) Theoretical Oxygen demand (ThOD)
11) Total organic carbon (TOC)

1) Total solids, suspended solids, settelable solids

⇒ Suspended solids are those which remain floating in water

⇒ Dissolved solids are those which are dissolve in water.

⇒ Colloidal solids are those which are finely divided solids remain either in suspension or in solution.

⇒ Settlable solids are that portion of solid matter which settles out if the waste water is kept undisturbed for about 2hrs.

⇒ The solids in waste water may be organic solids as well as inorganic solids (Eg- inorganic matter consist of sand, gravel, debris, chloride, ~~se~~ sulphate etc & organic matter ~~consist of fats & oils for from kit~~ may be — (i) carbohydrates such as cellulose, fibres, sugar etc.

(ii) fats & oils from kitchen, garages, shops etc.

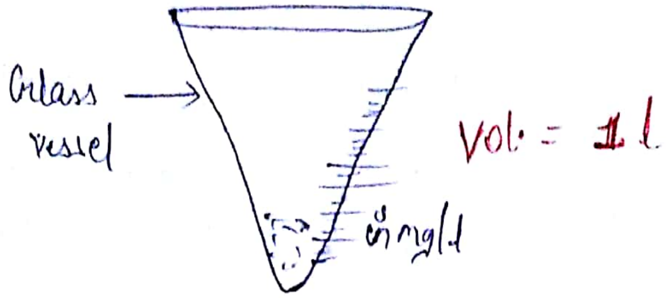
(iii) Nitrogeneous compounds like urea, protiens, fatty acids etc.)

⇒ The amount of various kind of solids may be determined as —

a) Total amount of solids can be determined by evaporating a known vol. of waste water sample & weighing the residue left. The mass of residue. The mass of residue left divided by vol. of sample is total solids in mg/l.

b) The quantity of settelable solids can be determine by using Imhoff cone. Waste water is allowed to stand in cone for about 2hrs, & quantity of solids settle down.

at the bottom directly read out which gives approx. amount of settlable solids.



2) pH value

⇒ The determination of pH value is very important as it gives an idea about certain treatments which depends upon pH value.

⇒ Fresh waste water is generally alkaline but as time passes it becomes acidic bcoz of bacterial action in anaerobic condition or Nitrification process.

⇒ The pH value can be measured by potentiometer which measure electropotential exerted by hydrogen ions & thus indicating the conc^{n} of hydrogen ions.

3) chloride content

⇒ These are derived from Kitchen waste, human ~~weases~~ feases, urinary waste.

⇒ The normal chloride content of wastewater is 120mg/l. However large amount of chloride may enter from ice-cream plant, meat salting etc.

⇒ The chloride content can be measured by ~~tx~~ titrating the sample by standard silver Nitrate ($AgNO_3$) sol^n using potassium chromate (K_2CrO_4) as indicator.

4) Nitrogen Content

⇒ Presence of Nitrogen in wastewater indicates presence of organic matter

⇒ Nitrogen is found in following forms –

Imp

a) Free ammonia or ammonia Nitrogen (indicates recent pollution)

b) Organic ammonia (indicates quantity of ammonia before decomposition has started)

c) Nitrites (indicates partial decomposition of organic matter)

d) Nitrate (indicates complete decomposition of organic mater)

5) Presence of Oil, Fat & Grease

⇒ Fats & acids are compounds of alcohol & glycerols with fatty acids. Such matter form scum on the top of sedimentation tank & clog the voids of filter medium. Therefore they interferes with Normal treatment methods hence they are to be detected & removed.

⇒ The amounts of fats & grease in waste water sample can be determined by evaporating it & then mixing the residual solids with Ether, the solⁿ is then poured off & evaporated leaving behind the fats & greases as residue.

6.) Sulphides , sulphates & H_2S Gas

⇒ Sulphides & sulphates are formed due to decomposition of various sulphur containg substances, present in waste water.

⇒ This decomposition also leads to evolution of H_2S gas causing bad smell & odour.

⇒ Besides this causing corrosion in concrete sewer pipes.

⇒ The aerobic & faccultative bacteria oxidises sulphur & its compound present in waste water to initially form sulphides & ultimately break it down into sulphate which is stable & unobjectionable end product.

$$S \xrightarrow[\text{faccultative}]{\text{aerobic}} \text{sulphide} \longrightarrow \underset{\text{stable}}{\boxed{\text{Sulphate}}}$$

⇒ The initial decomposition is associated with formation of H_2S.

7.) Dissolved oxygen (DO)

⇒ Dissolved oxygen is required for respiration of living things.

⇒ The dissolved oxygen in fresh waste water depends on temp. If the temp. of sewage is more dissolved oxygen will be less.

⇒ Max. quantity of Dissolved oxygen that can remain in water at a particular temperature is called Saturation Dissolved Oxygen

⇒ At Normal temp. saturation DO is taken as 9.2 mg/l or ppm. If DO is less than 4ppm fishes may not survive.

⇒ The dissolved oxygen contain waste water is ~~derto~~ determined by winKler's m/d

8.) Chemical Oxygen demand (COD)

⇒ COD Test is used to measure the total content of organic matter of waste water both biodegradable or Non-biodegradable.

⇒ The oxygen equivalent of organic matter that can be oxidised is ~~de~~ measured by using a strong chemical oxidising agent in acidic medium.

⇒ Potassium dichromate ($K_2Cr_2O_7$) is used as oxidising agent so sometimes this test is also called Dichromate oxygen demand test

9) Total organic Carbon (TOC)

⇒ It is another m/d of expressing the total organic matter in the form of carbon.

10) Theoritical Oxygen demand (ThOD)

⇒ If the chemical formula & quantity of all organic matter present in sewage is known the exact amount of oxygen required to oxidise this can be calculated. called Theoritical Oxygen demand.

11) Biochemical Oxygen Demand (BOD)

⇒ Biochemical oxygen demand is used as a measure of the quantity of oxygen required, to oxidation of Biodegradable ~~oxygen~~ organic matter present in water sample (waste water) by aerobic bacterias.

⇒ Oxygen demand of wastewater is exerted by –

(i) Carbonaceous organic materials

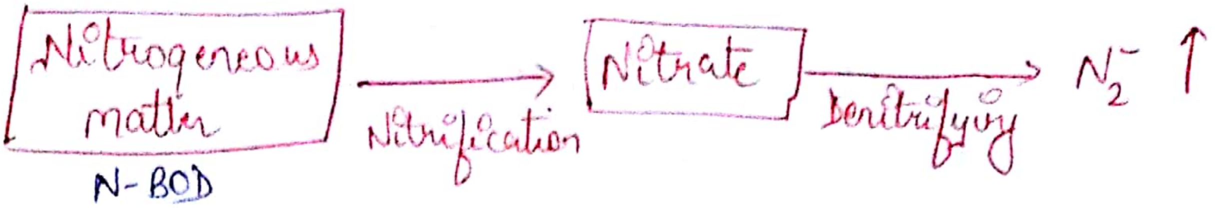
(ii) oxidisable Nitrogen

(iii) chemical reducing compounds.

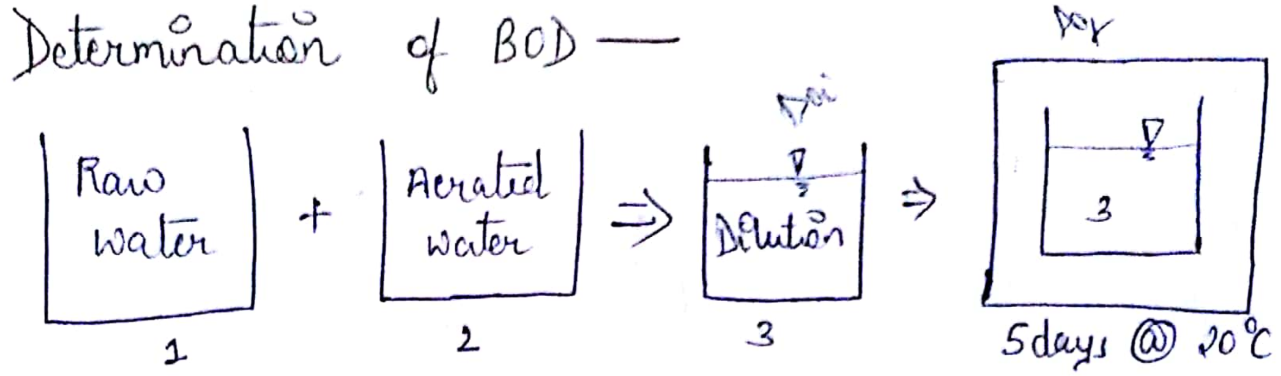
Note – 1) For domestic waste water about all oxygen demand is for carbonaceous matter.

2) When Nitrogeneous is also to be removed in treatment process Nitrogeneous demand is also found out this is called N-BOD (Oxygen require for conversion of Nitrogenous matter to Nitrate)

Removal of Nitrogen from system is achieved by first oxidising the Nitrogeneous organic matter to Nitrate (called Nitrification) & then denitrifying the Nitrate to ~~reo~~ release Nitrogen gass which goes out of sewage.



Determination of BOD —



⇒ BOD can be determined by diluting a known value of sample of waste water with a known of volume of pure water & then calculating the dissolved oxygen of this dilluted sample.

⇒ Dillution sample is then incubate for five days at 20°c the dissolved oxygen of dilluted sample after incubation is calculated again the diff. b/w initial dissolve oxygen & final dissolve oxygen will indicate the amount of oxygen consumed by dilluted sewage.

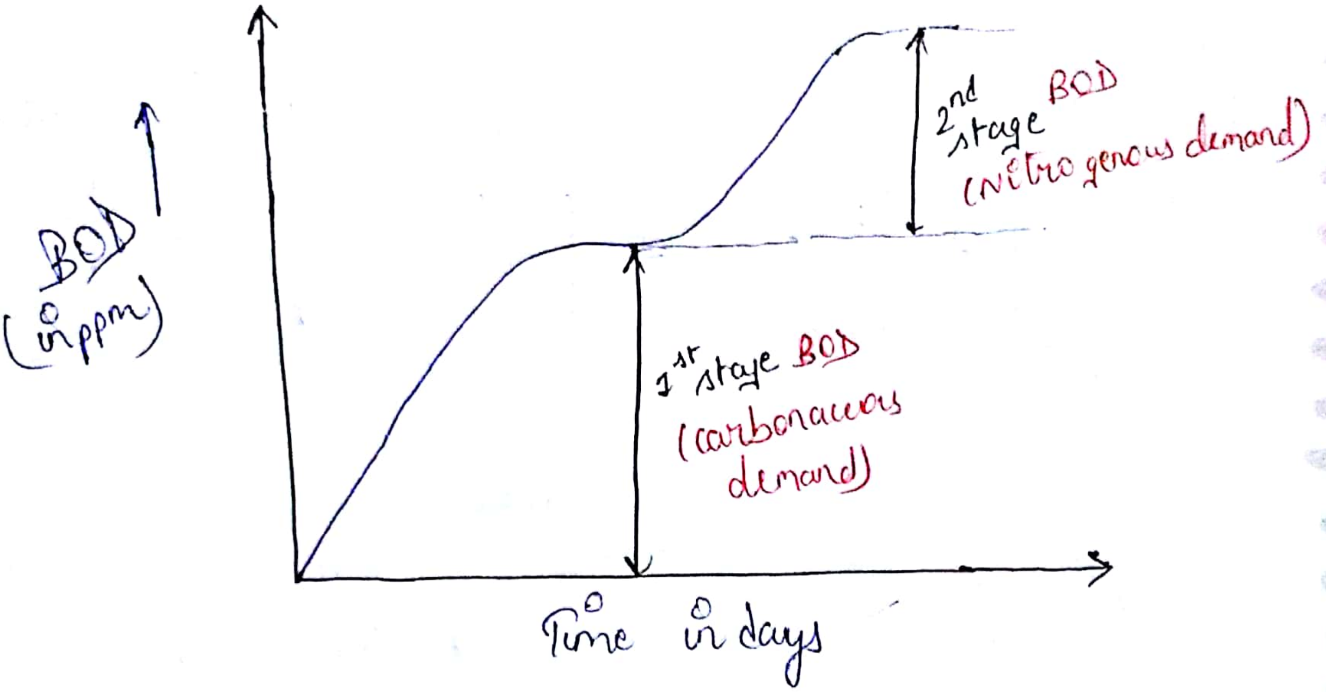
$$\boxed{BOD_{5,20^\circ} \text{ of Diluted sample} = DO_i - DO_f}$$

$$\boxed{BOD_5 \text{ of Raw water} = (DO_i - DO_f) \times \text{dilution factor}}$$

dilution factor = D.F = $\boxed{\text{Vol. of } 3 / \text{vol. of } 1}$

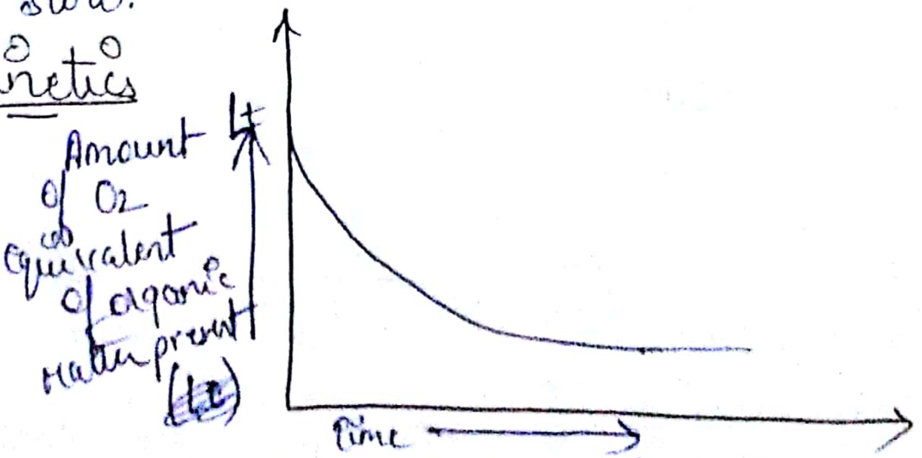
⇒ First demand occurs due to oxidation of organic matter & is called carboneous demand or first stage demand & later demand occurs due to biological oxidation of ~~demand~~ ammonia & is called Nitrogeneous demand or second stage demand.

⇒ However the term BOD usually means first stage BOD or carbonaceous demand.



⇒ Nitrogeneous demand starts only after 5-8 days because the ~~re pr~~ reproduction rate of Nitrification bacteria is slow.

Reaction Kinetics



L_t = amt of organic matter present at time (t)

$$\frac{dL_t}{dt} = -K L_t$$

$$\frac{dL_t}{L_t} = -K\,dt$$

$$\int_{L_0}^{L_t} \frac{dL_t}{L_t} = -\int_0^t K\,d_t$$

$$\log L_t - \log L_0 = -Kt$$

$$\log \frac{L_t}{L_0} = -Kt$$

$$\boxed{L_t = L_0 \cdot e^{-Kt}}$$

$$2.303 \log_{10}\left(\frac{L_t}{L_0}\right) = -Kt$$

$$\log_{10}\left(\frac{L_t}{L_0}\right) = \frac{-Kt}{2.303}$$

$$\log_{10}\left(\frac{L_t}{L_0}\right) = -K_D \cdot t$$

$$\boxed{L_t = L_0\, 10^{-K_D \cdot t}}$$

K_D = Deoxygenation Constant which is temp dependent

K = Reaction const.

$$\left\{ K_D = \frac{K}{2.303} \right\}$$

$BOD = L_0 - L_t$ → ऊपर वाली value डाल दो L_t की

$$BOD = L_0 - L_0 \cdot 10^{-K_D \cdot t}$$

$$\boxed{BOD = L_0\left(1 - 10^{-K_D t}\right)}$$ at any time

for BOD_5, $t = 5$

for K_D

$$\boxed{K_{D(T^\circ C)} = K_{D_{20^\circ C}}\left[1.047\right]^{T-20}}$$ → ~~Vet~~ Vanthoffs - arrhenius eqn

unit of K_D = day^{-1}

1) In general value of $\boxed{K_D}$ varies b/w $\boxed{0.05-0.2}$ for municipal sewage per day

2) For tap water $\boxed{K_D = 0.01 - 0.05}$ day^{-1}

3) Surface water $\boxed{K_D = 0.05 - 0.1}$ per day or day^{-1}

4) For untreated sewage $\boxed{K_D = 0.1 - 0.15}$ per day

5) For treated sewage $\boxed{K_D = 0.05 - 0.1}$ per day

But let $K_D = 0.1$

$$BOD_5 = L_0\left[1 - 10^{-K_D \times 5}\right]$$

$$BOD_5 = L_0\left[1 - 10^{-0.1 \times 5}\right]$$

or $\boxed{BOD_5 = 0.68 L_0}$ L_0 = ultimate BOD

Q Calculate 1 day 37°C BOD for sewage sampe whose 5 day 20°C BOD is 100mg/l. Assume K_D at 20°C is 0.1 per day

Soln

$$[K_D]_{T^\circ C} = K_{D\,20^\circ C}\left[\cancel{1.047}\ 1.047\right]^{T-20}$$

$$[K_D]_{37^\circ C} = 0.1\left[1.047\right]^{37-20}$$

$$(K_D)_{37^\circ C} = 0.21 \text{ day}^{-1}$$

$$BOD_5 = L_0\left[1 - 10^{-K_D \cdot t}\right]$$

$$100 = L_0\left[1 - 10^{-0.1 \times 5}\right]$$

$$BOD_u = L_0 = 147.05$$

$$BOD_1 = L_0 \left[1 - 10^{-K_D t} \right]$$

$$BOD_1 = 147.05 \left[1 - 10^{-0.21 \times 1} \right]$$

$$BOD_{1\ 37^\circ C} = 56.37 \text{ mg/l}$$

Q Following observations were made on 3% dilution of waste water, dissolved oxygen of aerated water used for dilution is 3mg/l D.O of dilluted sample after 5days of incubation is 0.8mg/l D.O of original sample is 0.6mg/l Calc. 5 day BOD & ultimate BOD take $K_D = 0.1 \text{ day}^{-1}$.

Sol^n D.O of aerated water = 3mg/l

D.O of dilluted sample after 5days = 0.8mg/l

D.O of raw ~~wa~~ water = 0.6mg/l

$D.f = \frac{100}{3}$ ~~$D.f = \frac{3}{100} = 0.03$~~

$$DO_1 = \frac{A \times DO_1 + B \times DO_2}{A+B}$$

$$BOD_5 = \underbrace{(DO_i - DO_f)}_{(3)\ \text{not}} \times D_f$$

$$DO_i \text{ of dilution} = \frac{3 \times 0.6 + 97 \times 3}{3+97} = 2.928$$

$$BOD_5 = (2.928 - 0.8) \times \frac{100}{3} = 70.9$$

$$BOD_5 = BOD_u \left[1 - 10^{-K_D t} \right]$$

$$70.9 = BOD_u \left[1 - 10^{-0.1 \times 5} \right]$$

$$\boxed{BOD_u = 103.8 \text{ mg/l}}$$

Population Equivalent

Avg. standard BOD of domestic sewage is 80 g/day percapita ↳ औसत

The No. of person which produces amt. of BOD at the rate of 80g per person per day equal to that produce by industrial sewage is called ~~popul~~ population equivalent of industrial sewage.

Eg → Industrial sewage produces BOD = 80000 g/day (say)

$$\text{Population equivalent} = \frac{80000}{80\cancel{000}}$$

$$= 1000 \text{ g/day}$$

Ch-6 Bio-chemical Reactions in treatment of wastewater

Aerobic Reactions & Anaerobic Reactions

⇒ The aerobic & anaerobic react^n are two basic form of ~~stab~~ biological stabilization, ~~of~~ react^n whose occurence is dependent upon the availability of Oxygen.

⇒ Aerobic reaction takes place in the presence of ~~oxyg~~ free oxygen & produce reasonably stable organic end product, with relatively low energy contents.

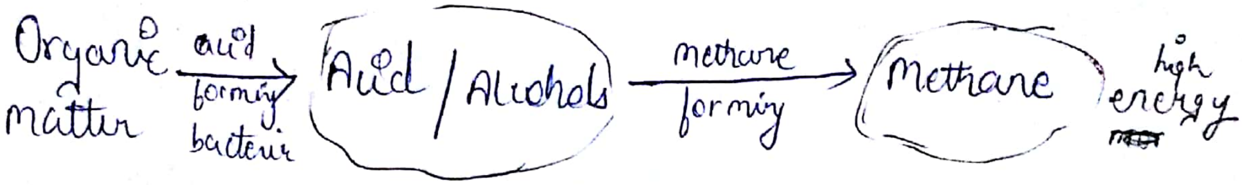
A ~~A~~ considerable portion of the organic matter is synthesized ([illegible]) to new microbial cells.

⇒ The react^n provides high degree of stabilization although the synthesized micro-organism result in formation of large vol. of sludge which require further treatment.

⇒ Anaerobic reaction take place only in absence of oxygen, these react^n are more complex because they occur in two stages carried out by different type of bacterias — acid forming bacteria
Methane forming bacteria

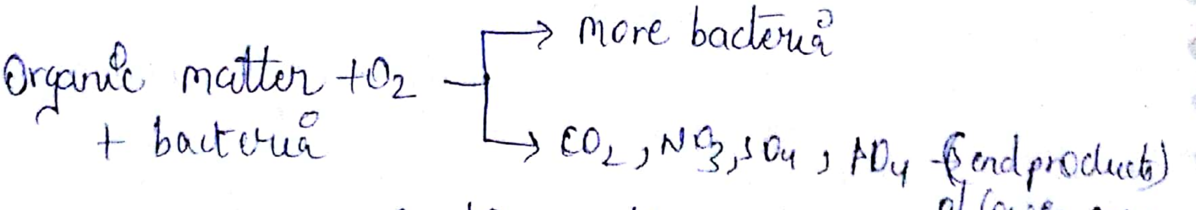
⇒ Acid forming bacteria initially convert complex organic matter into organic acid/alcohols. At this point methane forming bacterias convert acids & alcohols into methane & other end products such as Hydrogen sulphide (H_2S).

⇒ The end product of Anaerobic react^n still contain considerable amount of energy.

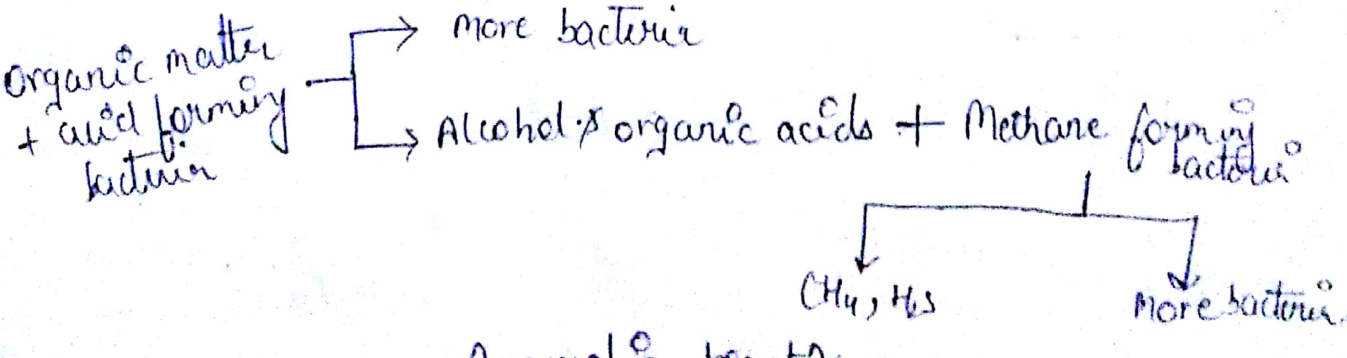


⇒ Because of lower release of energy in anaerobic react^n the synthesis of New microbial cell is very much less than in aerobic ~~bacteria~~ react^n.

⇒ It means there is less sludge from anaerobic stabilization of a waste water, then in aerobic stabilization in same water.



Aerobic react^n



Anaerobic react^n

Biological Growth

⇒ In biological treatment process the material to be stabilized provides basic nutritional & energy requirements for its conversion into end products & formation of New microbial cell.

⇒ Catabolic reactⁿ are those in which food is broken to release energy.

⇒ Reaction which provides material for synthesis of new microbial cells is anabolic.

⇒ In the absence of organic matter micro-organism can exist for some time bcoz of the existence of auto-oxidation or endogeneous respiration.

⇒ In endogeneous respiration which takes place continuously in biological system & cells die & release organic matter & Nutrients back into system where they can be reused.

Parameters	Aerobic-oxidation	Anaerobic-oxidation
Oxygen requirement	Abundant	Nil
Area of application	Commonly dilute liquids, waste & solid waste composting	commonly sludges
Energy release (per gram mole glucose)	484-674 K.cal	26Kcal
Stages required for complete process	1	2

Decomposition & Products	CO_2 ; H_2O, NO_3	CH_4, H_2S, NH_3
BOD of effluent	less (less than 60mg/l)	High (up to 5000mg/l)

Various Types of Bacterias

Autotrophs - They derive both energy & material from inorganic material.

Hetrotrophs - They drive both energy & material from organic material

Phototrophs - Utilises sunlight as energy source & inorganic material for material source.

Aerobic bacteria - presence of O_2

Anaerobic bacteria - absence of O_2

Faccultative bacteria - presence & absence of O_2

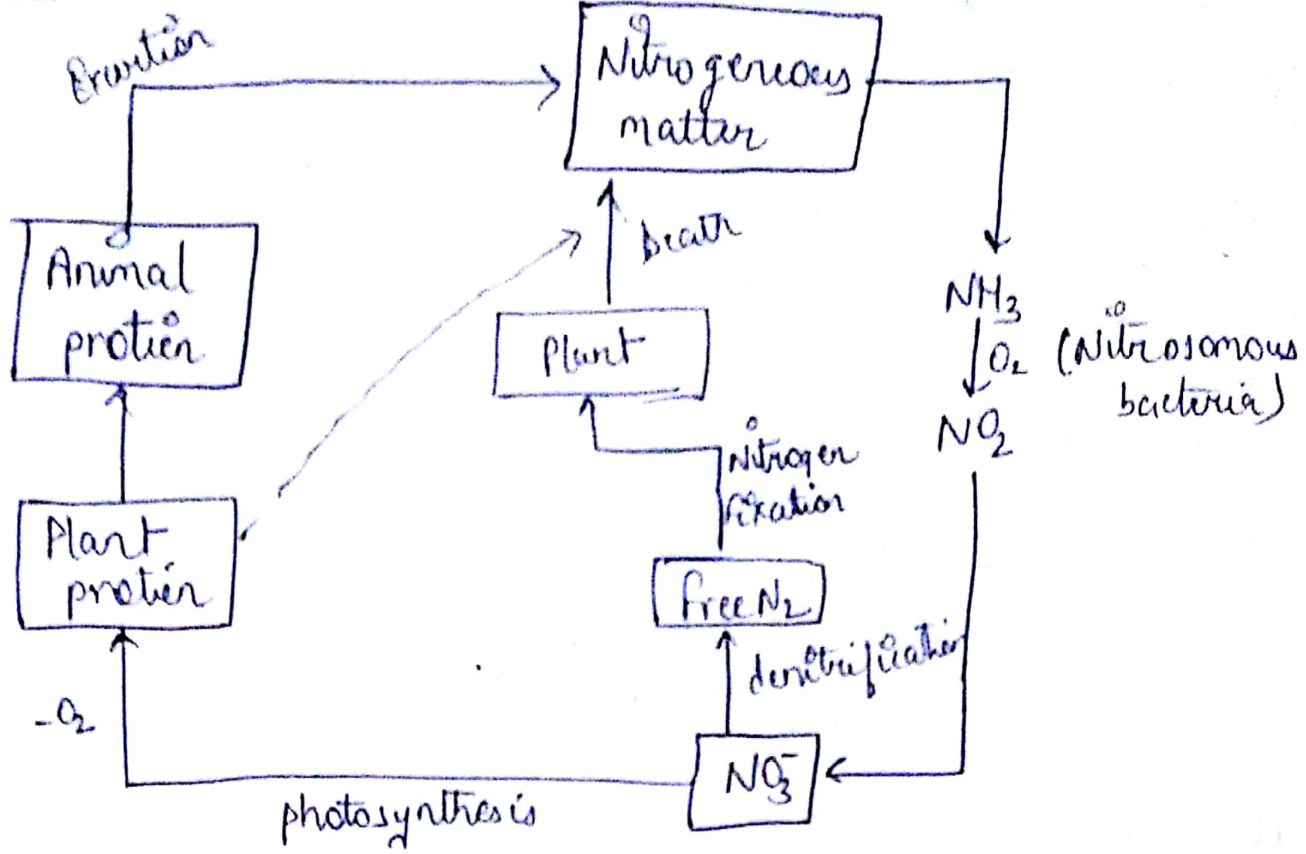
Various Natural Cycles

1) Nitrogen cycle
2) Sulphur cycle
3) Carbon cycle

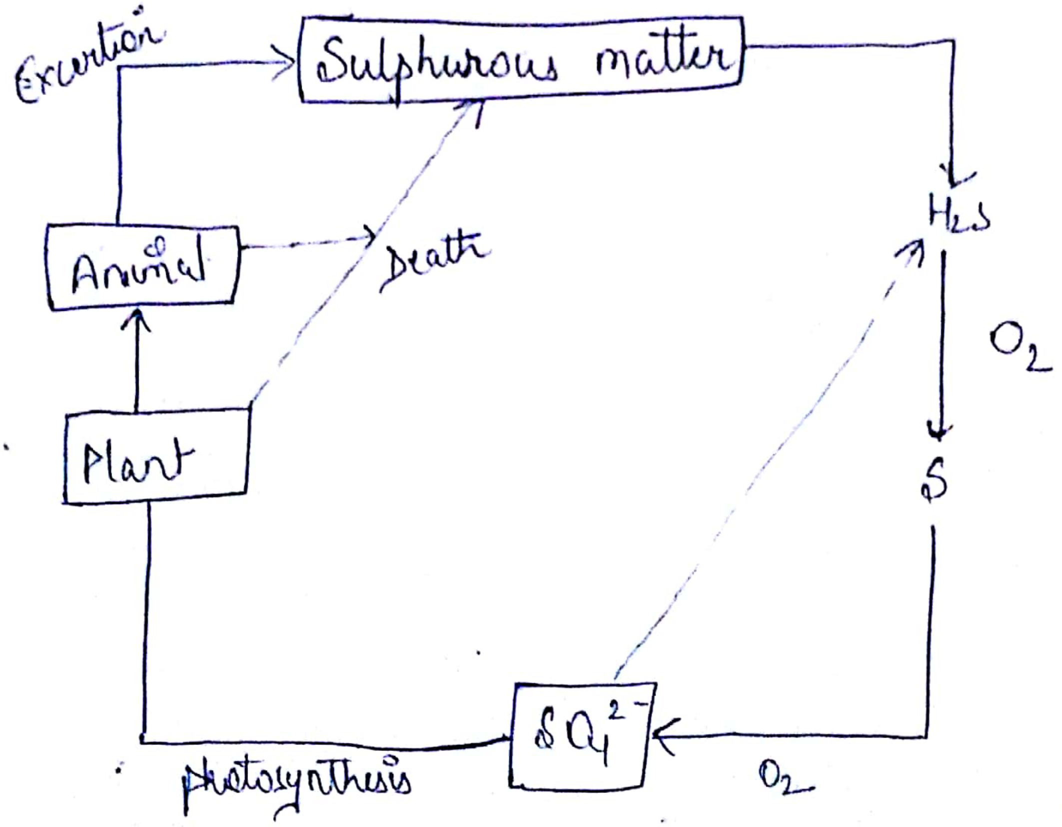
} Aerobic cycle

- Anaerobic cycle

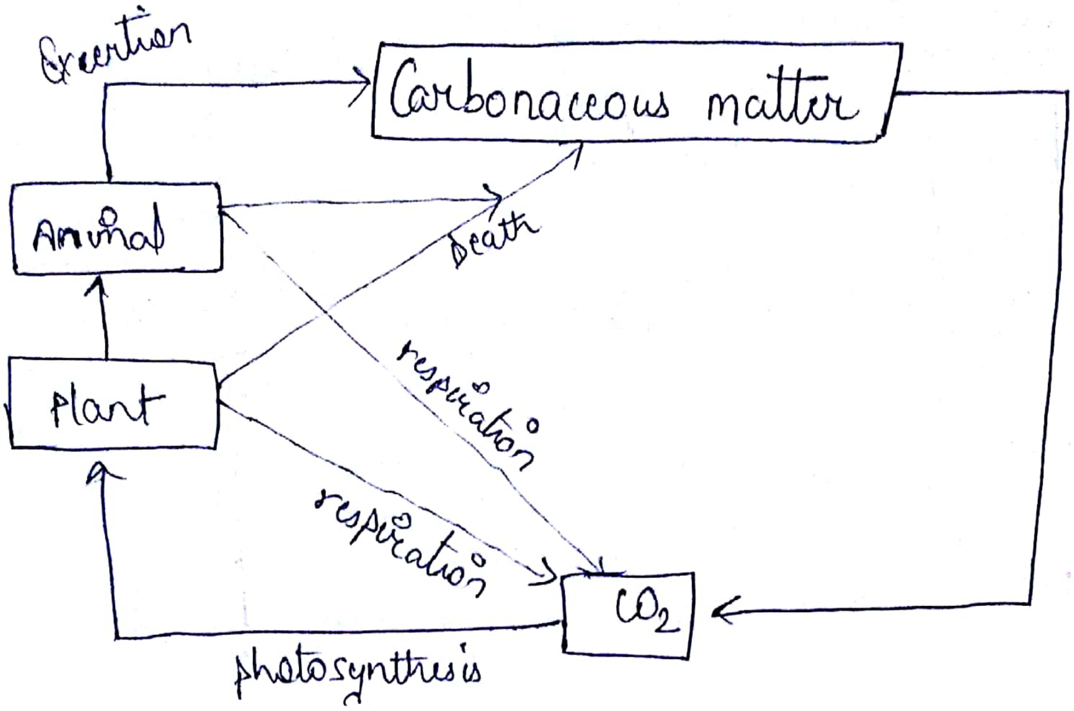
1) Nitrogen Cycle



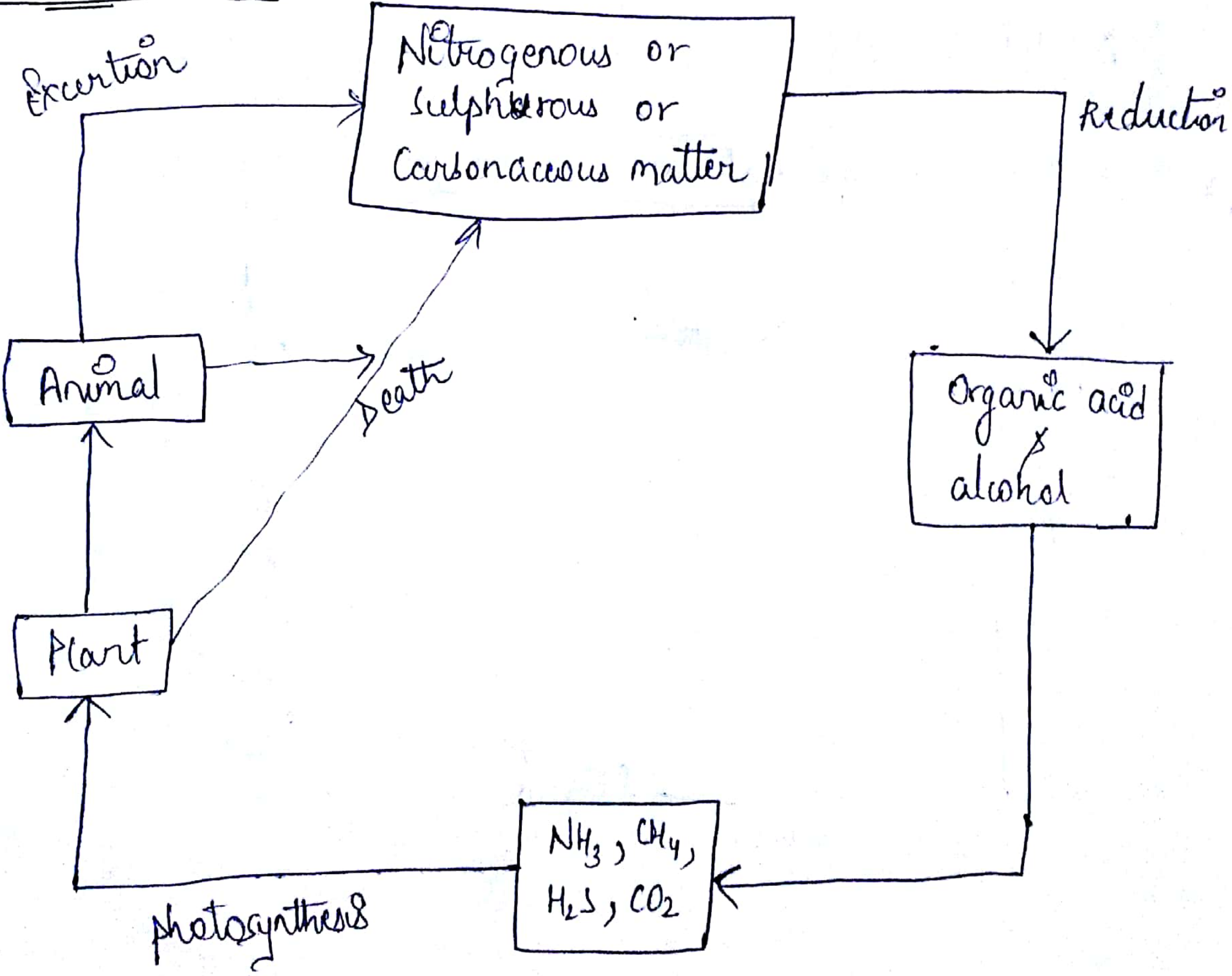
2) Sulphur Cycle



3) Carbon Cycle



Anaerobic Cycle



Ch-7 Disposal of sewage effluent

⇒ There are two general m/ds of disposing of sewage effluent –

1) Dilution i.e disposal in water
2) Disposal on land.

⇒ Disposal by dilution is most common m/d, it is a process where by the treated sewage or the effluent from the sewage treatment plant is discharge into a river stream or large water body such as lake or sea.

⇒ Standard of dilution for discharge of waste water into rivers –

Dilution Factor	Standards of purification required
Above 500	No Treatment is required sewage can be directly discharged
B/w 300 - 500	Primary Treatment such as plain sedimentation is given to sewage & the effluent should not contain suspended solid more than 150ppm
B/w 150 - 300	Treatment such as screening, sedimentation & some chemical precipitation is required such sewage effluent should not contain suspended solids more than 60ppm
less than 150	Complete thorough (proper) treatment should be given sewage should not contain ss more than 30ppm & BOD more than 20 ppm

⇒ These parameters are given as per royal comission report on sewage.

Bureau Indian Standard (BIS) standards for dispisal of sewage

Imp

Parameters	Domestic sewage of discharge into surface water source	Industrial waste	
		Surface water	Public sewer
BOD_5	20mg/l	30mg/l	500mg/l
pH	—	5.5 — 9	5.5-9
Suspended solids	30mg/l	100mg/l	600mg/l
Phenolic compounds	—	1mg/l	2mg/l

Mechanism of Self purification

⇒ When a sewage is disposed in a river self purification ~~by~~ by natural agents takes place

⇒ The self purification occurs by various mechanism they are —

1) Dilution & dispersion:
2) Sedimentation
3) Sunlight
4) Biological oxidation
5) Reduction

Scanned by CamScanner

1) Dillution & Dispersion

⇒ Dillution only reduces potential nuisance (जो नुकसान करे) due to sewage, it is not a self purification process.

$$\boxed{C_{mix} = \frac{C_S Q_S + C_R Q_R}{Q_S + Q_R}}$$

किसी में
भी निकाल
सकते हैं
चाहे, DO, BOD

C_S = conc. of material in sewage
C_R = conc. of material in river
Q_S = Discharge of sewage
Q_R = Discharge of river

2) Sedimentation

⇒ The suspended solids in sewage (organic & inorganic) will settle down on the bed of river.

⇒ Primarily settled organic solid at bottom are stablised by bacterias.

3) Sunlight

⇒ Due to sunlight, in the process of photosynthesis oxygen is released which helps in oxidation of organic matter. Thereby forming a stable product.

4) Biological Oxidation

⇒ Oxidation of organic matter occurs ~~due~~ due to oxygen mixed in river water.

5) Reduction

(breaking down of any substance by water)

⇒ By hydrolysis of organic matter settles down at bottom either chemically or biologically, these organic matter are stablized, process called reduction

Factors affecting self purification

1) **Temperature** – Inc. in temp leads to decrease in dissolved oxygen, hence the oxygen gets quickly depleted & anaerobic condition may set up.

2) **Turbulence** – It leads to increase ~~oxygen~~ mixing of oxygen in water.

3) **Amount & type of organic matter** – Some compound can be easily oxidized & some will take time thereby purification may be slow or fast depending upon the type & amt. of organic matter.

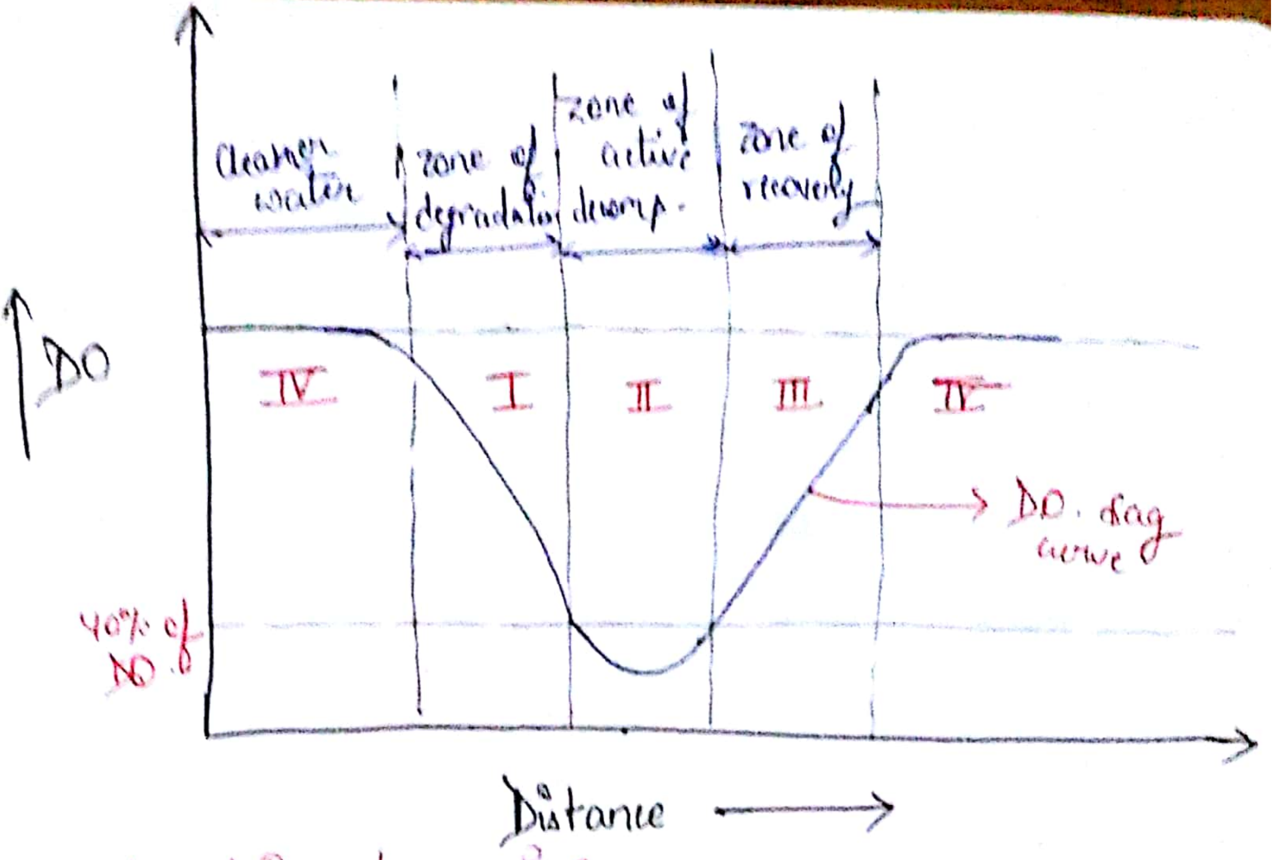
4) **Hydrography of River stream** – Higher velocity & large surface area leads to greater turbulence & dillution of sewage, this will help in purification.

5) **Rate of reaeration** – More is the rate of reaeration faster will be purification.

Zone of Pollution in river stream

a) Zone of degradation

⇒ It is found upto certain distance downstream of the point at which sewage is discharge into the river. In this zone algaes die but fishes may survive, water becomes dark & turbid. DO reduces upto 40% of saturation.



b) Zone of active decomposition

⇒ It is zone of heavy pollution, water is darker than zone of degradation, dissolved oxygen may fall to zero, fishes will disappear.

⇒ At upper ends anaerobic bacteria will replace aerobic bacteria, hence anaerobic condition setup in zone of active decomposition, thus gases like CH_4, H_2S, CO_2 will be evolved (formed).

⇒ At the end of this zone DO concentration will reach upto 40% of the saturation DO.

c) Zone of recovery

⇒ BOD falls down organic matter will produce NO_3 sulphate etc.

⇒ DO contents rises above 40% of saturation value.

d) zone of clearer water

⇒ DO. will rise upto saturation value but pathogens may remain.

Oxygen Deficit

⇒ When Bio-degradable organics are discharged into river stream containg DO.

⇒ Micro-organism begin the metabolic process to convert the organics ~~matter~~ along with DO to new cells & oxidise waste product.

⇒ The rate at which DO is used will depend on the quantity & quality (type) of organics

⇒ The DO that is used from the stream must be bring back otherwise anaerobic condition will bring up.

⇒ Two mechanism are known to contribute oxygen –

a) Reaeration

b) Photosynthesis

⇒ At a particular temperature there is a max. value of dissolved oxygen that remain fixed in water saturation DO.

$$\boxed{\text{Oxygen Deficit} = [\text{saturation DO} - \text{actual DO}]}$$

~~$D = C_s - C$~~

⇒ D.O deficit is the driving force for reaeration ~~Later~~ Greater ~~will~~ is deficiter, greater will be Rate of Reaeration.

Eg:- $D.O = 0.5 mg/l$, $Q = 2 m^3/s$ calc. Oxygen deficit

$D.O = 8$, $Q = 1500 m^3/s$

$DO_s = 10$

Soln

$$D.O \text{ of mix} = \frac{0.5 \times 2 + 1500 \times 8}{1502} = 7.9$$

Oxygen deficit $= 10 - 7.9 = 2.1$

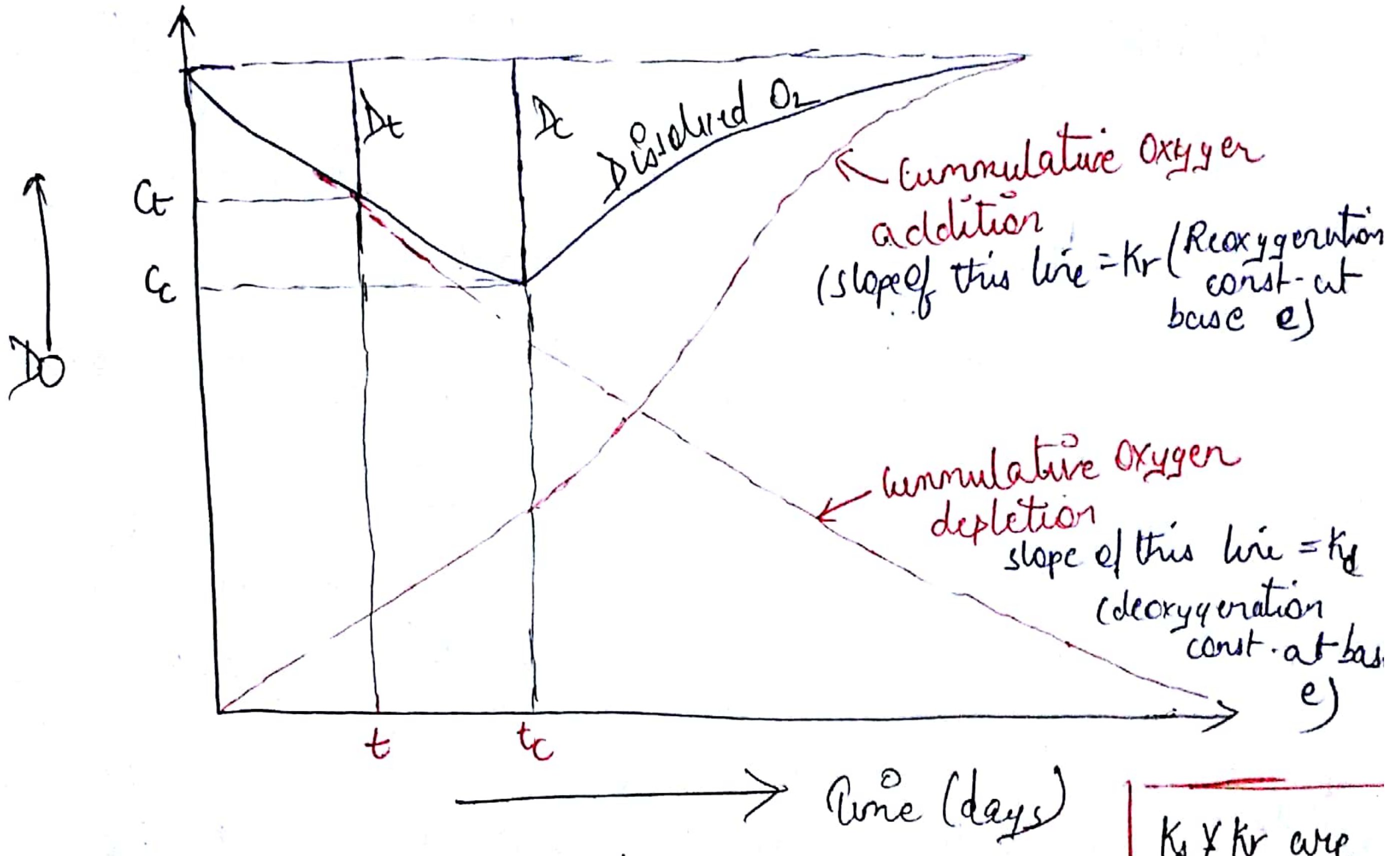


fig:- characteristics of oxygen sag curve

Oxygen deficit

$$\boxed{D = \frac{K_d L_o}{K_r - K_d}\left[e^{-K_d t} - e^{-K_r \cdot t}\right] + D_o e^{-K_r t}}$$

$$\boxed{D = \frac{K_D \cdot L_o}{K_R - K_D}\left[10^{-K_D t} - 10^{-K_R \cdot t}\right] + D_o e^{-K_R t}}$$

→ Strecher Phelps eqn

K_d & K_r are at base e
K_D & K_R are at base 10

Critical time at which D.O is minimum —

$$\boxed{t_c = \frac{1}{K_R-K_D}\times\log_{10}\left[\left\{\frac{K_D L_o-(K_R-K_D)\cdot D_o}{K_D\cdot L_o}\right\}\frac{K_R}{K_D}\right]}\ \text{days}$$

K_R = Re-oxygenation constant
K_D = De " constant
L_o = ultimate BOD
D_o = Initially $(D.O_s - D.O)$ of mix

Critical Oxygen defect i.e Max oxygen defect

$$\boxed{D_c = \frac{K_D L_o}{K_R}10^{-K_D t_c}}$$ If t_c given

$$\boxed{\left(\frac{L_o}{D_c\cdot f}\right)^{f-1} = f\left[1-(f-1)\frac{D_o}{L_o}\right]}$$ If t_c not given

where $f = \frac{K_R}{K_D}$

f = self purification const / coeff. of purification

Q 125 m³/s of a sewage is discharge in a river which is fully saturated with Oxygen & flow at a rate of 1600 m³/s with a velocity 0.12 m/s If the 5 day BOD of sewage 300 mg/l. Find out where the critical dissolve oxygen will occur in river assume coefficient of purification of river is 4, $K_D = 0.11$/day, BOD ulti = 1.5 times BOD_5 are $DO_{sat.} = 9.2$ mg/l

→ Mix की ले रहे है)

Sol^n $Q_s = 125 m^3/s$, $DO_{sew} \overset{\text{Not given}}{\equiv} 0$, $BOD_5 = 300 mg/l$ → sewage

$Q_R = 1600 m^3/s$, $DO = 9.2$, $BOD_5 = 0$ → river

(वहां sab है रही है तो 0 mg/l)

$K_D = 0.11$ per day

$f = 4 \Rightarrow \frac{K_R}{K_D} = K_R = 0.44$

Note - Assuming temp of river is 20°C, saturation DO of river = 9.2 mg/l.

DO of sewer = 0

$t_c = ?$, $D_c = ?$

$$D_o = [C_s - C_{mix}]$$

$$\text{DO of mix } \cancel{(D_o)} \; C_{mix} = \frac{Q_s \cdot DO_s + Q_R \times DO_R}{Q_s + Q_R}$$

$$= \frac{125 \times 0 + 1600 \times 9.2}{1600 + 125}$$

$$\cancel{D_o} \; C_{mix} = 8.53 mg/l$$

$$D_o = 9.2 - 8.53 = 0.67 mg/l$$

$$\underset{\text{ultimate BOD of mix}}{L_o} = 1.5 \times BOD_5 \, mix$$

$$BOD_5 \, mix = \frac{Q_s \times BOD_5 + Q_R \times BOD_R}{Q_s + Q_R}$$

$$= \frac{300 \times 125 + 1600 \times 0}{1600 + 125} = 21.74 mg/l$$

$$L_o = 1.5 \times 21.74 = 32.60 mg/l$$

$$t_c = \frac{1}{K_R - K_D} \log_{10}\left[\left\{\frac{K_D \cdot L_o - (K_R - K_D) D_o}{K_D \cdot L_o}\right\} \frac{K_R}{K_D}\right]$$

$$= \frac{1}{0.44 - 0.11} \log_{10}\left[\left\{\frac{0.11 \times 32.60 - (0.44 - 0.11) 0.67}{0.11 \times 32.6}\right\} \frac{0.44}{0.11}\right] \Rightarrow 1.74 d$$

Case $D_c = \frac{K_D L_0}{K_R} 10^{-K_D t_c}$

$$D_c = \frac{0.11 \times 32.60}{0.44} 10^{-0.11 \times 1.74}$$

$$\boxed{D_c = 5.24 \text{ mg/l}}$$

Alternate

$$\left[\frac{L_0}{D_c \cdot f}\right]^{f-1} = f\left[1 - (f-1)\frac{D_0}{L_0}\right]$$

$$\left[\frac{32.6}{D_c \times 4}\right]^{4-1} = 4\left[1 - (4-1)\frac{\overset{0.82}{\cancel{D_o}}}{32.60}\right]$$

$$\boxed{D_c = 5.24}$$

velocity $= 0.12$ m/s

$D = v \times t$

$= 0.12 \times 1.74 \times 86400 = 18040.32$ m

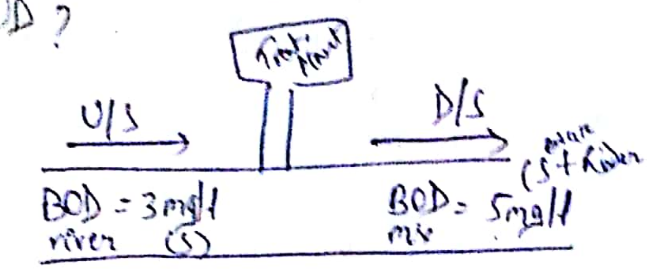
$= 18.04$ Km

Q The treated domestic sewage of a town is to be discharged in a natural stream. Calc the percentage of purification required in the treatment plant with following data.

(i) Population = 50,000
(ii) BOD contribution per capita = 007 kg/day
(iii) BOD of upstream = 3 mg/l
(iv) permissible max BOD of stream downstream = 5 mg/l
(v) Dry weather flow of sewage = 140 lpcd
(vi) Min flow of stream = $0.13 \text{ m}^3/\text{s}$

What is amount of % for treatment in terms of BOD?

Sol



$Q_R = 0.13 m^3/s$

$BOD_{Riv} = 3mg/l$ $\qquad$ $1m^3 = 1000 l$

$BOD_{mix} = 5mg/l$

Discharge of sewage $Q_S = 50000 \times 140 \frac{l}{d}$

$= \frac{50000 \times 140 \times 10^{-3} m^3}{86400}$

$Q_s = 0.081 m^3/s$

per. BOD of mix $= 5mg/l$

$$BOD_{mix} = \frac{Q_s BOD_s + Q_R \times BOD_R}{Q_s + Q_R}$$

$$5 = \frac{0.081 \times BOD_s + 0.13 \times 3}{0.081 + 0.13}$$

$$\boxed{BOD_{sew} = 8.2 mg/l}$$

Initial BOD of sewage $= 0.07 Kg/day \times 50,000$

$= 3500 Kg/day$

$\frac{3500 Kg/day \times 10^6}{140 \times 50000 \frac{l}{day}}$ (for mg) $= ~~500 Kg/l~~$ $\times 10^6$ $~~= 500~~$

$= 500 mg/l$

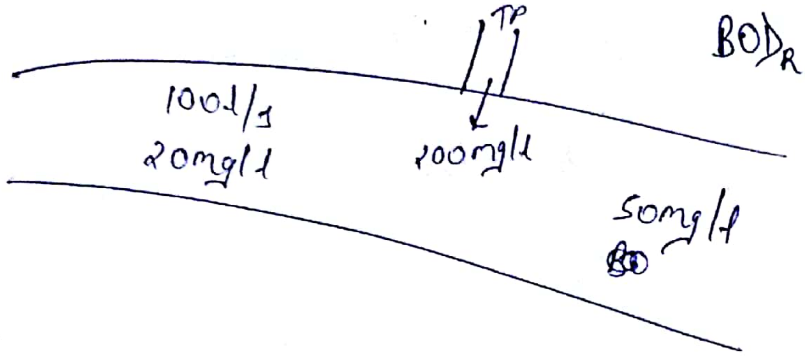
$$\% \text{ purification} = \frac{\overset{Int.}{500} - \overset{Final}{8.2}}{\underset{Ini}{500}} \times 100 = \boxed{98.36\%}$$

Q A river before its entry into a ~~town~~ town has a discharge of 100 l/s & has 20mg/l as conc. of a conservative parameter. The ~~too~~ towns wastewater outfalls having 200mg/l conc. of same conservative parameter raised the river conc. of 50mg/l after a complete mix of river water determine the dilution ratio.

Sol

$Q_R = 100 l/s = 100 \times 10^{-3} = 0.1 m^3/s$

$BOD_R = 200 mg/l$



conservative parameter
= जैसे की D.O या BOD

$$BOD^{o}_{mix} = \frac{Q_R \times BOD_R + Q_s \times BOD_s}{Q_R + Q_s}$$

$$50 = \frac{\overset{100 l/s}{\cancel{0.1 m^3/s}} \times 20 mg/l + Q_s \times 200}{0.1 + Q_s}$$

~~So~~ $Q_s = 20 l/s$

$$D.F = \frac{Q_R + Q_s}{Q_s} = \frac{100 + 20}{20}$$

$$\boxed{D.F = 6}$$

Q A large stream has reoxygenation const of 0.4/day at a velocity of 0.85m/s & at the point at which an organic pollutant is discharge It is saturated with oxygen at 10mg/l ($D_0 = 0$) below the outfall the ultimate demand of oxygen is found to be 20mg/l & the deoxygenation const 0.2/day. What is DO at 48.3Km downstream.

Ans

$K_R = 0.4/\text{day}$ $DO_{satu} = 10\,mg/l$, $L_0 = 20\,mg/l$

$K_D = 0.2/\text{day}$

Distance = 48.3Km

velocity = 0.85 m/s

$$\text{Time} = \frac{48.3 \times 1000}{0.85\ \frac{m}{s}}$$

Time = 56823.52 s

$$\text{Time (in days)} = \frac{56823.52}{86400} = 0.65\ \text{days}$$

Oxygen deficit

$$D = \frac{K_D L_0}{K_R - K_D}\left[10^{-K_D t} - 10^{-K_R t}\right] + D_{\cancel{0}} e^{-K_a t}$$

$$= \frac{0.2 \times 20}{0.4 - 0.2}\left[10^{-0.2 \times 0.65} - 10^{-0.4 \times 0.65}\right] + 0$$

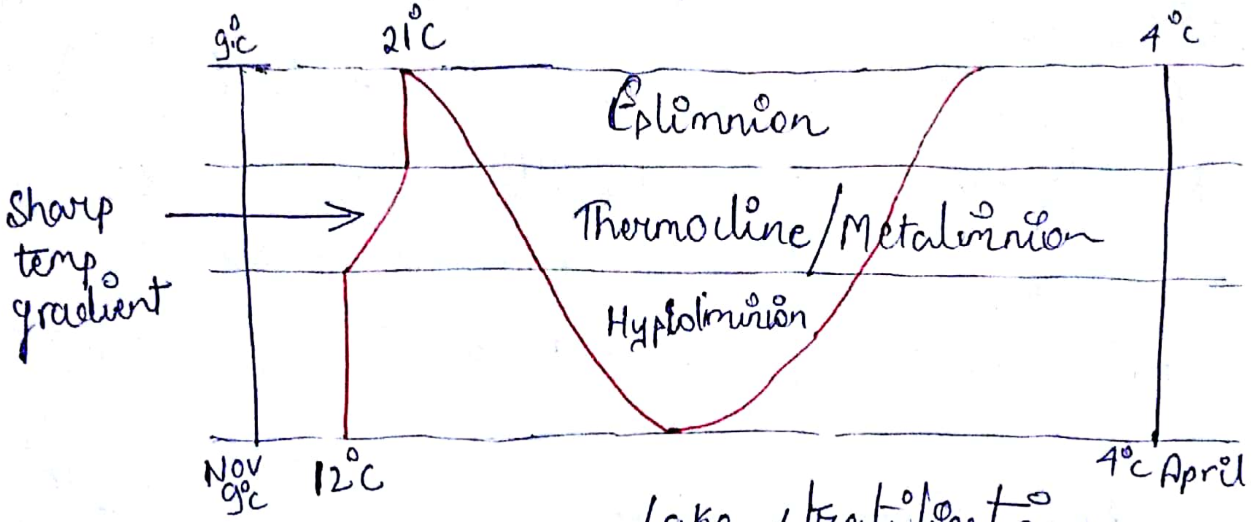
$D = 3.83\ mg/l$

Oxygen deficit = Saturation DO − actual DO

Actual DO = ~~3.83~~ 10 − 3.83

$$\boxed{DO = 6.16\ mg/l}$$

Disposal of waste water in lake & management of waste water



Lake stratification

⇒ The fresh water reaches their max density at 4°C with density declining as water ~~f~~ moves towards freezing point or grows warmer.

⇒ During warmer season water divides into upper layer of warm water is known as Epilimnion.

⇒ In Epilmnion zone water is well mixed & sufficient oxygen is available.

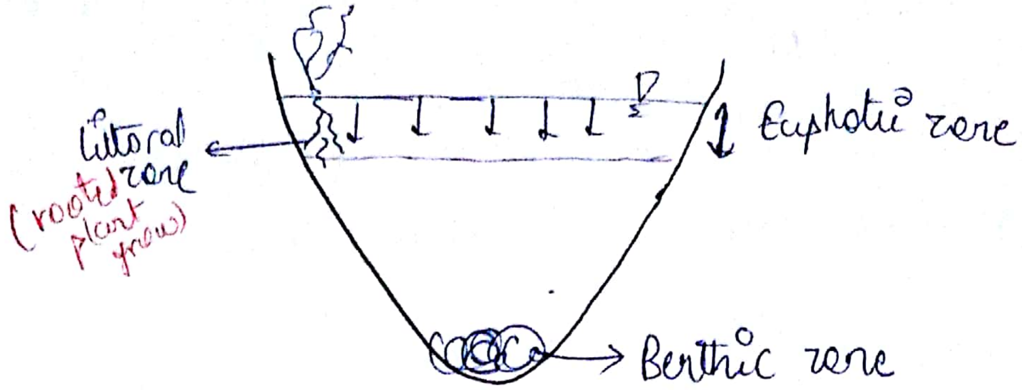
⇒ The lower layer is cool & unmixed, is known as Hypolimnion, this zone have deficiency of oxygen

⇒ These 2 layers are seperated by Thermocline/metalminion

⇒ This is a region of sharp temp. gradient.

Biological zones in lake

1) Euphotic zone
2) Littoral zone
3) Benthic zone



1) Euphotic zone

⇒ The upper layer of lake through which sunlight can penetrate is called euphotic zone.

⇒ The depth of euphotic zone is decrease by turbidity

2) Littoral zone

⇒ The zone where rooted plant grow is called littoral zone. This zone never extend beyond euphotic zone.

3) Benthic zone

⇒ The bottom sediments in lake that contains bacteria called benthic zone

Productivity of lake

⇒ Productivity is measure of ability ~~of~~ to support of food chain.

- ⇒ It is measure of algal growth
- ⇒ Higher algal growth will lead to decrease quality of water

⇒ Depending upon the increasing order of productivity lakes can be classified as —

a) Oligotrophic lakes
b) Mesotropic lakes
c) Eutrophic lake
d) Senescent lakes

Increased productivity ↓

1) Oligotropic lakes

⇒ These have a low level of productivity due to limited supply of nutrients.

~~2) Mesotropic lakes~~

Notes → In these types of lake Hypolimnion layer will be aerobic.

2) Mesotropic lakes

⇒ It supports medium growth of algaes, hyplimnion layer is aerobic but there will be depletion of oxygen.

3) Eutrophic lakes

⇒ It has fairly high productive lakes which support considerable growth of algaes. Euphotic zone will partially extend to epilimnion.

⇒ Hypolmnion will be Anaerobic.

4) Senescent lakes

⇒ It represent very old lake get ~~infes~~ infested with algaes has ~~also~~ almost become marshy.

Eutrophication of lakes

⇒ It is a natural process under which lakes get infested with algae.

⇒ This productivity is due to entry of nutrients (like Carbon, phosphorus, Nitrogen etc)

Disposal of wastewater into sea

Acc. to BIS standards —

BOD — 100 mg/l
COD — 250 mg/l
pH — 5.5-9
SS — 100 mg/l
Fluoride — 15 mg/l
Oil & grease — 20 mg/l

⇒ Sewage will be disposed during low tides only

Disposal on land

BIS standard of waste water effluent to be disposed on irrigated land.

1) BOD at 20°C → 500 mg/l
2) TDS → 2100 mg/l
3) Oil & grease → 30 mg/l
4) chlorides → 600 mg/l
5) Sulphates → 1000 mg/l
6) % of sodium w.r.t total conc. of sodium, calcium, magnesium & potassium → 60%

i.e $\frac{\text{sodium}}{\text{sodium} + Ca + Mg + K}$

Ch-8 Design of Sewer System

⇒ The hydraulic design of sewer & drains which means finding out their ~~sections &~~ ~~grad~~ & ~~gradients or the same time~~ mean size, gradient carried out as that of water supply lines but there are two differences.—

1.) The water supply pipes carry pure water without carrying any solid particle but the sewage may contain solid particles & suspended particles also, as a result of flow due to settling of these solid particles in sewage velocity of flow reduces. Thus ultimately clogging of sewer may occur. In order to avoid such clogging / ~~silting~~ silting it is necessary that the pipe dimension laid at such a gradient as to generate self cleansing velocity at different possible discharges (कम से कम Discharge पर भी Particle नहीं रुकना चाहिए). On the other hand sewer material should be capable of resisting wear & tear due to these solid particles.

2.) The water supply pipe carry water under pressure enhance they may carried up & down hills whereas in case of sewer system only gravity conduits are used & they must be laid at certain downward gradient (from household to treatment plant)

Laying of sewer

All the sewer pipes are generally laid starting from their outfall end towards their starting end. The laying of sewer consist of following steps ——

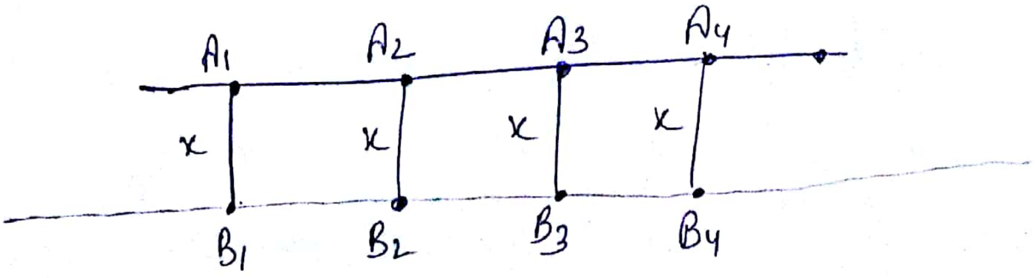
1) Marking the alignment
2) Excavation of trench
3) Bracing the trench
4) Dewatering the trench
5) laying and joints of pipe
6) Testing the leakage.

1) Marking the alignment

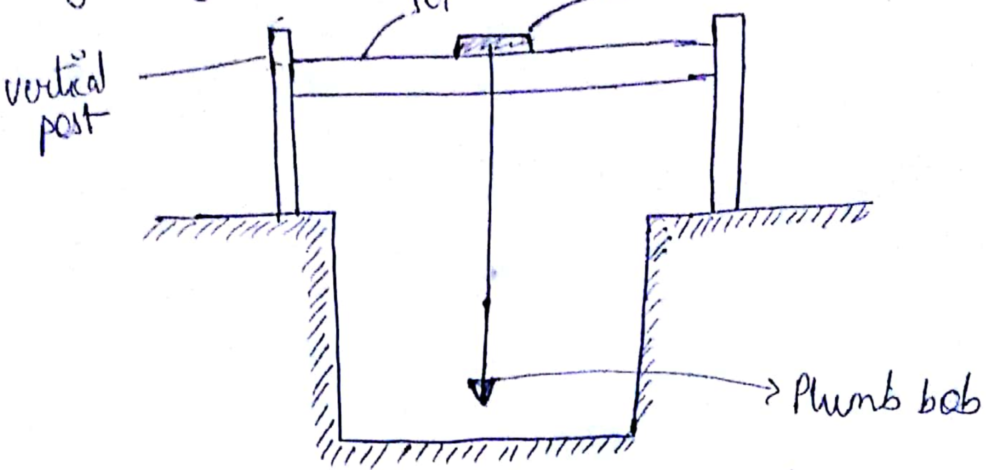
a) By reference line

In this m/d a reference line is mark along a side of road by theodolite & invar tape then the points ~~A~~ like A_1, A_2, A_3, A_4 on reference line are mark the center line through first refrence point (B_1) is marked with help of Peg, then distance is measured $A_1 B_1 = x$ then distance

~~$A_1 A_2 = B$~~. $A_2B_2 = A_3B_3 = A_4B_4 = x$



b) By sight rail

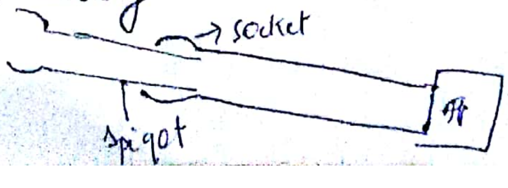


⇒ In this m/d 2 vertical post are driven at a suitable distance apart then by ranging through a theodolite, the center line is mark with Nail on a sight rail which is fixed on a vertical post the sight rail should be fixed in such a way that it upper edge is just coincide with line of sight

⇒ The center of line of, sewer is transfered to ground by plumb bob.

2) Excavation of Trench
3) Bracing of trench
4) Dewatering the trench
5) laying & Joint of pipes

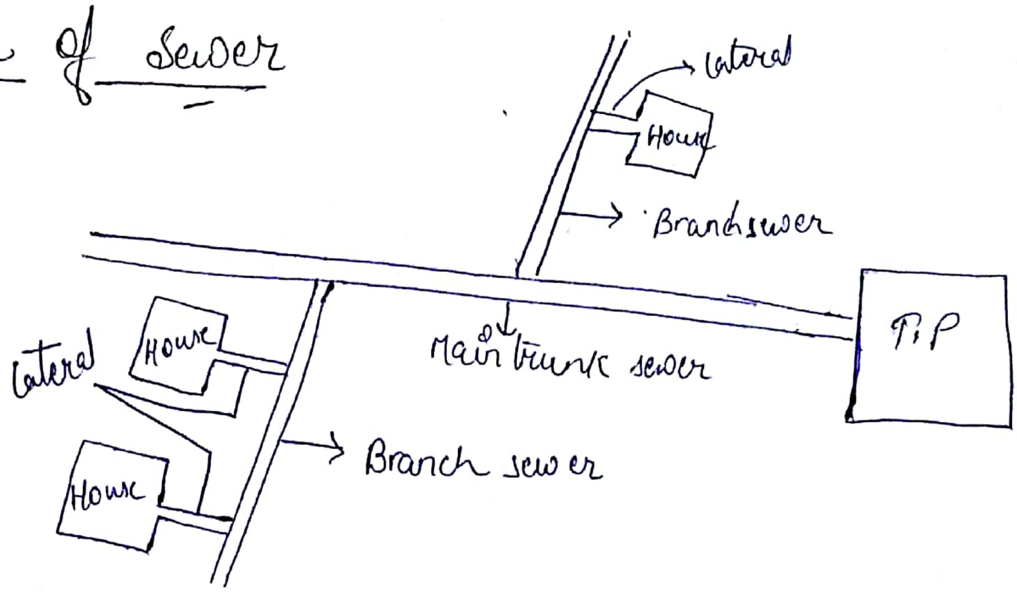
⇒ After the preparation of sewer bed the operation of Joint is performed in which sewer pipe lanes are laid from lowest point with their socketed end facing upstream & spigot of each pipe be easily inverted in the socket.



6) Testing of leakage

a) By water Test
b) Air Test

Design of Sewer



⇒ Sewer should be designed so as to carry entire sewage effectively & efficiently from house to the point of disposal

⇒ Sewer are design not to flow under gravity because reserve space in sewer safeguards against flow fluctuations, they should be adequate in size & a proper gradient.

⇒ Sewers are of 2 type –

1) Sanitary sewer
2) Storm sewer

1) Sanitary sewer

⇒ In sanitary sewer system lateral system collects discharge from houses & carry then to branch sewer than branch sewer carries it to main sewer & main sewer transport it to treatment plant, manholes are provided at intersection of sewer lines & at regular interval for regular inspect & cleaning.

2) Storm Sewer

⇒ Storm sewer carries surface runoff during rainfall.

Combined system

⇒ This system consist of a single sewer line of large diameter through which sewage & storm water are collected & carried to treatment plant.

⇒ Self cleansing velocity is easily acheived.

~~⇒ Assumpti~~

⇒ As the single sewer lines serves double function so it is comparitively economical.

Assumptions in sewer design

1) The flow of waste water in sewer is steady & uniform.

2) Design of sewer is based on peak flow discharge.

Flow Formula

As per manning

$$\boxed{V = \frac{1}{n} R^{2/3} S^{1/2}}$$

V = velocity of flow

R = Hydraulic Radius = A/P

S = Slope

Design Data

Pipe size	Design condition
D < 0.4m	1/2 full at max discharge
0.4 ≤ D ≤ 0.9m	2/3 full at max discharge
D > 0.9m	$\frac{3}{4}$ full at max discharge

Peak Factor / Peaking Factor

$$= \frac{\text{max. discharge}}{\text{Avg. discharge}}$$

Population	Peaking factor
upto 20,000	3
20,000 - 50,000	2.5
50,000 - 750000	2.25
> 750000	2

Min flow $= \frac{1}{3}$ to $\frac{1}{2}$ of avg flow

⇒ Sewer should be design to carry peak discharge & should be check to ensure at min discharge. (i.e at min. discharge velocity of flow should not be less than self cleansing velocity)

⇒ Self cleansing velocity is min velocity at which no solids gets deposited on the bed of sewer. ~~It is~~

⇒ Self cleansing velocity is given by –

Shield Formula

for rote

$$\boxed{V = \frac{1}{n} R^{1/6} \left[K_s (G_s - 1) d_p \right]^{1/2}}$$

G_s = sp gravity of solid particles
d_p = particle size
R = Hydraulic radius
n = Manning's coefficient
K_s = dimensionless constant its value is about $\boxed{0.04 - 0.8}$

Max velocity

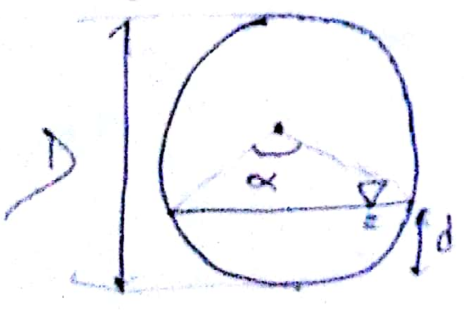
To avoid erosion of pipe max. velocity should be limited as —

Concrete sewer → 2.5 – 3 m/s
Cast iron sewer → 3.5 – 4.5 m/s
Stone ware sewer → 3–4 m/s

Note Slope of sewer should be designed for min. permissible velocity at min. flow

Partial flow characteristics of circular sewer

Q4



$$\boxed{\frac{d}{D} = \frac{1}{2}\left(1 - \cos\alpha/2\right)}$$

d = depth of flow
D = Dia. of pipe

$$\boxed{\frac{a}{A} = \frac{\alpha}{360} - \frac{\sin\alpha}{2\pi}}$$

a = area of flow under partial flow condition

A = area of sewer

$$\boxed{\frac{p}{P} = \frac{\alpha}{360}}$$

p = wetted perimeter

P = πD

$$\boxed{\frac{r}{R} = 1 - \frac{360\sin\alpha}{2\pi\alpha} = \frac{a/A}{p/P}}$$

r = Hydraulic radius under partial flow

R = Hydraulic radius of full flow $\frac{\frac{\pi}{4}D^2}{\pi D} = D/4$

$$\boxed{\frac{v}{V} = \frac{1/n \, r^{2/3} s^{1/2}}{1/N \, R^{2/3} S^{1/2}}} \qquad \boxed{\frac{v}{V} = \frac{N}{n}\left(\frac{r}{R}\right)^{2/3}}$$

→ If bed slope is same

$$\boxed{\frac{q}{Q} = \frac{a \times v}{A \times V}} \qquad \boxed{RS = rs}$$

when slope is different

Points ~~Not~~ to be Noted

1) If manning's coefficient is assumed to be constant then

$$\boxed{\frac{v}{V} = \left(\frac{r}{R}\right)^{2/3}}$$

Imp

2) For constant 'n' velocity of flow is max when $\boxed{d/D = 0.81}$ & this Vmax is $\boxed{12.5\%}$ greater than the velocity at full flow.

Imp

3) For constant 'n' discharge is max when $\boxed{\frac{d}{D} = 0.95}$ & this discharge is $\boxed{7\%}$ more than that at full flow

Equal degree of self cleansing

When two sewers are kept on different slopes than under this condition for equal degree of self cleansing —

$$\boxed{\frac{v}{V} = \frac{N}{n}\left(\frac{r}{R}\right)^{1/6}}$$

Q. Design a sewer to serve a population of 36,000, daily water supply per capita is equal to 135 L of which 80% goes into sewer slope is 1 in 625 & sewer would be design to carry 4 times the avg discharge under design condition. n = 0.013

Assume $\frac{d}{D} = \frac{3}{4}$

Soln

$P = 36,000$

$= 135$ lpcd

~~Max daily = 8 36,000 × 135~~ $\frac{L}{d}$

Total sewage = 36000 × 135 = 4860000 l/d

= ~~36,000 × 135 × 1.8~~

$q = 80\% = \frac{\cancel{380}38.8}{86400} \times 10^5 \times 10^{-3} = 0.045\ m^3/s$

= ~~8748000~~

peak discharge = $4 \times 0.045 = 0.18\ m^3/s = q$

$$\frac{d}{D} = \frac{1}{2}(1 - \cos\alpha/2)$$

$$\frac{3}{4} = \frac{1}{2}(1 - \cos\alpha/2)$$

$$1 - \cos\alpha/2 = \frac{3}{2}$$

$$\alpha = 240^\circ$$

$$\frac{a}{A} = \frac{\alpha}{360} - \frac{\sin\alpha}{2\pi}$$

$$\frac{a}{A} = \frac{240}{360^\circ} - \frac{\sin 240^\circ}{2\pi} = \boxed{0.804}$$

$$\frac{v}{V} = \frac{1/n\ r^{2/3}\ s^{1/2}}{1/N\ R^{2/3}\ s^{1/2}} \qquad \left\{ N = n = 0.012 \right\}$$

$$\frac{v}{V} = \left(\frac{r}{R}\right)^{2/3} \times \frac{s^{1/2}}{s^{1/2}}$$

$$\text{for } \frac{r}{R} = 1 - \frac{360 \sin 240^\circ}{2\pi\ 240} = 1.20$$

$$\frac{v}{V} = (1.2)^{2/3} = \boxed{1.12}$$

$$\frac{q}{Q} = \frac{a}{A} \times \frac{v}{V} \Rightarrow Q = \frac{0.18}{1.12 \times 0.804} = \boxed{0.19}\ m^3/s$$

$Q = V.A$

$Q = \frac{1}{n} R^{2/3} S^{1/2} . \frac{\pi D^2}{4}$

$0.19 = \frac{1}{0.012} \left(\frac{D}{4}\right)^{2/3} \left(\frac{1}{625}\right)^{1/2} \frac{\pi D^2}{4}$

$0.19 = \frac{1}{0.012} \times 0.40\, D^{2/3} \times \frac{1}{25} \times \frac{3.14}{4} D^2$

0.18 ~~0.045~~ $= D^{2.66}$

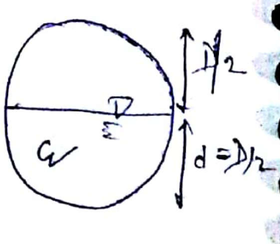
$\boxed{D = 0.54\ m}$

Q Determine size of circular sewer for a discharge of 500 l/sec running 1/2 full assume bed slope $S = 0.0001$ & $n = 0.015$.

Soln

$q\ Q = 500\ l/s = 500 \times 10^{-3}\ m^3/s$

for 1/2 full $R = \frac{\frac{\pi/4\, D^2}{2}}{\frac{\pi D}{2}} = \frac{D}{4}$



$$q = v \times A$$

$$q = \frac{1}{n} s^{1/2} r^{2/3} \cdot \frac{\pi}{8} \left(D\right)^2$$

$$0.5 = \frac{1}{0.015} (0.0001)^{1/2} \left(\frac{D}{4}\right)^{2/3} \frac{\pi}{8} (D)^2$$

$$\boxed{D = 1.8m}$$

Alternate

$$\alpha = 180^\circ$$

$$\frac{q}{Q} = \left(\frac{\alpha}{360^\circ} - \frac{\sin\alpha}{2\pi}\right)\left(1 - \frac{360 \sin^\circ \alpha}{2\pi\alpha}\right)^{2/3}$$

at $\frac{q}{Q} = \frac{1}{2}$

$$Q = 1 m^3/s$$

$$Q = Av$$

$$Q = \frac{1}{n} R^{2/3} S^{1/2} \times \frac{\pi}{4} D^2$$

$$1 = \frac{1}{n} \left(\frac{D}{4}\right)^{2/3} \times (0.0001)^{1/2} \times \frac{\pi}{4} D^2$$

$$\boxed{D = 1.8m}$$

Q Design a sewer of Diameter 300mm flowing at $\frac{d}{D} = 0.3$ on a grade ensuring a degree of self cleansing equivalent to that obtain in a sewer of diameter 300mm in which at full flow velocity is 0.9m/s. Find required grade (s) velocity of flow & discharge.

– $n = 0.013$

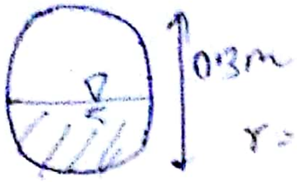
Sol^n

Assume $n = 0.013$

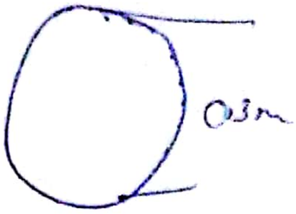
$\frac{d}{D} = 0.3$

$D = 300mm$

$\frac{d}{D} = \frac{1}{2}(1 - \cos\alpha/2)$



$0.3 = \frac{1}{2}(1 - \cos\alpha/2)$



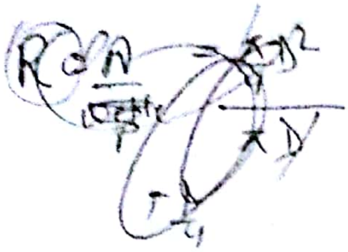
$\alpha = \cancel{227}^\circ\ 132$

$q \quad v \quad s$

V full flow

Q S

$\frac{r}{R} = 1 - \frac{360 \sin\alpha}{2\pi\alpha}$



$\frac{r}{R} = 1 - \frac{360 \sin \cancel{227}\ 132}{2\pi \cancel{227}\ 132}$

$\frac{r}{R} = \cancel{1.18}\ \boxed{0.67}$

For good self cleansing velocity

$\frac{v}{V} = \frac{N}{n} \cancel{\times 1.18}(0.67)^{1/6}$

$\frac{v}{V} = 0.935$

$v = 0.935 \times 0.9 = \boxed{0.84m/s}$

$R = \frac{D}{4} = \frac{0.3}{4} = 0.075m$

$V = \frac{1}{n} R^{2/3} S^{1/2}$

$0.9 = \frac{1}{0.013}(0.075)^{2/3} S^{1/2}$

$S = \cancel{0e}\ 4.328 \times 10^{-3} = \boxed{\frac{1}{231}}$

~~$v = \frac{1}{n}$~~ $\boxed{\gamma s = RS}$ → when s is different.

$$s = \frac{R}{\gamma} \times S$$

$$= \frac{1}{0.67} \times 0.00432$$

$$\boxed{s = 0.00632}$$

$$\frac{q}{Q} = \frac{a \times v}{A \times V}$$

$$Q = A \times V = \frac{\pi}{4} D^2 \times 0.9$$

$$= \frac{\pi}{4} (0.3)^2 \times 0.9$$

$$Q = 0.06 \, m^3/s$$

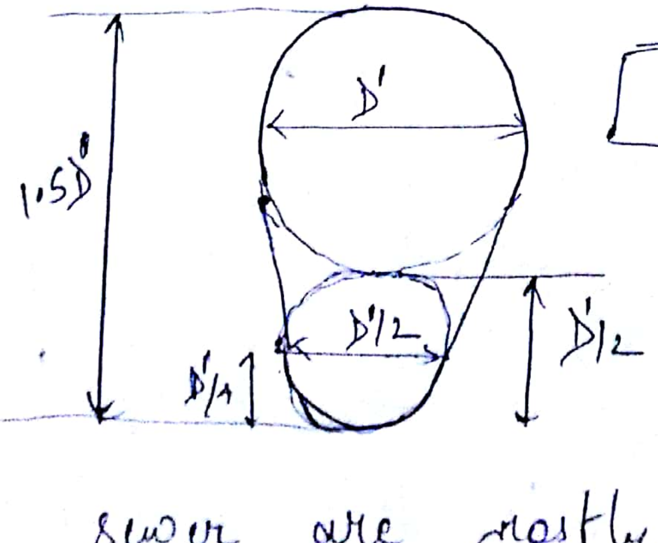
$$\frac{q}{0.063} = 0.25 \left(\frac{0.86}{0.9} \right)$$

$$\frac{a}{A} = \frac{\alpha}{360} - \frac{\sin \alpha}{2\pi}$$

$$\boxed{q = 0.015 \, m^3/s}$$

$$\frac{a}{A} = 0.25$$

Egg shaped sewer



$\boxed{D' = 0.84 D}$ → circular sewer

⇒ circular sewer are mostly used for separate sewage system but the advantage of circular sewer is obtained only when sewer is running half full

⇒ when the depth goes below half depth velocity reduces considerably.

⇒ Thus to take advantage of circular sewer 2 such circular sewers are assumed to combined together into a form of egg shape in which smaller ~~circul~~ circular section will be effective during dry weather & full circular section is effective during max flow.

⇒ Egg shape sewer of an equivalent section whose top diameter $\boxed{D' = 0.84D}$ where D = dia of circular section of same c/s area.

Storm water Drainage

⇒ Design of storm sewer requires estimation of peak runoff

⇒ Peak runoff ~~is~~ will depend on intensity of rainfall.

⇒ Intensity of rainfall depends on recurrence (frequency) interval & duration of rainfall.

Time of Concentration

⇒ In the starting the entire catchment will not be contributing the runoff at the monitoring point. The time after which the entire catchment start contributing the runoff is called time of concentration.

$$\boxed{\text{Time of conc.} = \text{Overland flow time} + \text{Channel flow time}}$$

Rational Formula

$$\boxed{Q = CiA}$$

Q = peak rate of runoff

C = runoff coefficient = $\frac{\text{runoff}}{\text{Rainfall}}$

i = intensity of rainfall

A = catchment area

⇒ Runoff coefficient is also called impermeability factor

⇒ When severall different type of surface comprise the catchment then runoff coefficient —

$$\boxed{C = \frac{C_1A_1 + C_2A_2 + C_3A_3 + --- C_nA_n}{\Sigma A}}$$

⇒ It is necessary to multiply the point rainfall intensity by a factor in order to account the non-uniformity of rainfall over the catchment basin.

∴ Design rainfall intensity = (Point rainfall intensity) × (Areal dispersion factor)

$$\boxed{i = i_0\left(\frac{2}{1+T_c}\right)}$$

i_0 = Rainfall intensity ~~*~~ ~~Aeration factor~~

i = critical rainfall intensity

T_c = Time of conc. (hr)

Q A combined sewer of circular section is laid to serve a particular area. Calculate size of sewer for following Data —

Area = 120hect, Population = 1,00000, max velocity of flow = 3m/s, Time of entry = 10min

Time of channel flow = 20min, per capita water supply = 250lpcd, aerial/aeration factor K = 0.45, hourly maximum rainfall of area = 5cm, Peak factor of sewage = 3

sol^n

Water supply = 250lpcd

Population = 1,00,000

Assume 80% of water supply goes into sewer = 0.8× 250 × 10,00,00 $\frac{L}{d}$ ⇒ $\frac{250 \times 10^5 \times 10^{-3}}{86400}$

peak $q = 0.28\ m^3/s \times 3 \times 0.8 = 0.69\ m^3/s$

t_c = Time of entry + Time of channel flow

= 10 + 20 = 30min = 0.5hr

$$i = i_0 \left(\frac{2}{1+t_c}\right) = 5\left[\frac{2}{1+0.5}\right] = 6.66\ cm/h$$

$= 0.06\ m/h$

~~$i = 6.66 \times \overset{(K)}{0.45} = 3\ cm$~~

peak storm discharge = Intensity × area × aeration factor

$$= \frac{0.06}{3600} \times 120 \times 10^4 \times 0.45$$

$Q_R = 10\ m^3/s$

$Q = 10 + 0.69 = 10.69\ m^3/s$

$$Q = v \times A$$

$$A = 3 \times \frac{\pi}{4} (D)^2$$

$$10.69 = 3 \times \frac{\pi}{4} \times D^2$$

$$\boxed{D = 2.13 m}$$

Sewer Appurtenances

1) Manholes -

⇒ A manhole is an opening constructed on alignment (straight line) of a sewer for facilitating a person access to a sewer for the purpose of inspection, testing, cleaning & removal of obstruction from sewer line.

⇒ Manhole should be built at every change of alignment, gradient or diameter, at the head of all sewers & branches & every junction of ~~to~~ two or more sewers.

⇒ The spacing of manholes above 90-150m may be allowed on the straight reach, for sewer of diameter 0.9-1.5 m

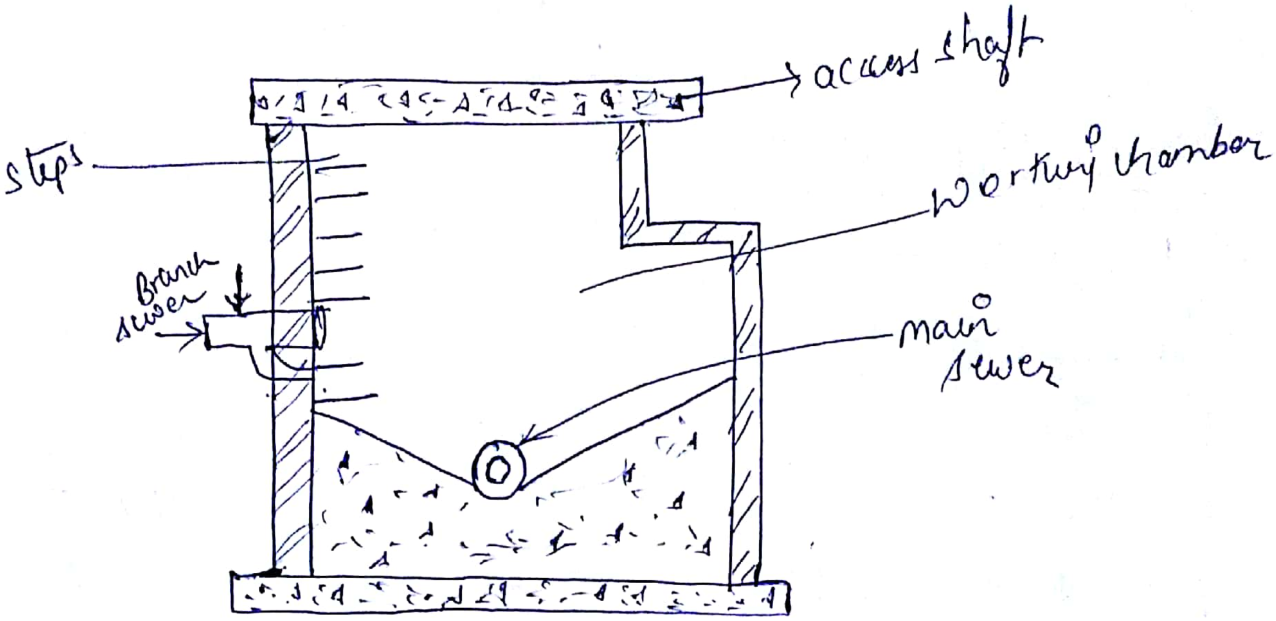
⇒ Spacing of manholes at 150-200m may be allowed on straight reach for sewer having diameter 1.5-2m

→ chamber [illegible] of manholes

⇒ The depth is less than 1m is considered for normal manholes & may be greater than 1.5m for deep man holes,

Types of Manhole

1) Straight through manholes
2) Junction manholes
3) Drop manholes



Drop manhole

Components of Manhole

a) Access shaft
b) Working chamber

a) Access shaft

⇒ It may be rectangular or circular (min size → 90×100 cm & 90 cm dia) serves as purpose of removing, preventing the debris & allow passes of worker.

b) Working chamber

⇒ It provides sufficient space for workers for cleaning of the sewer.

2) Lamp hole

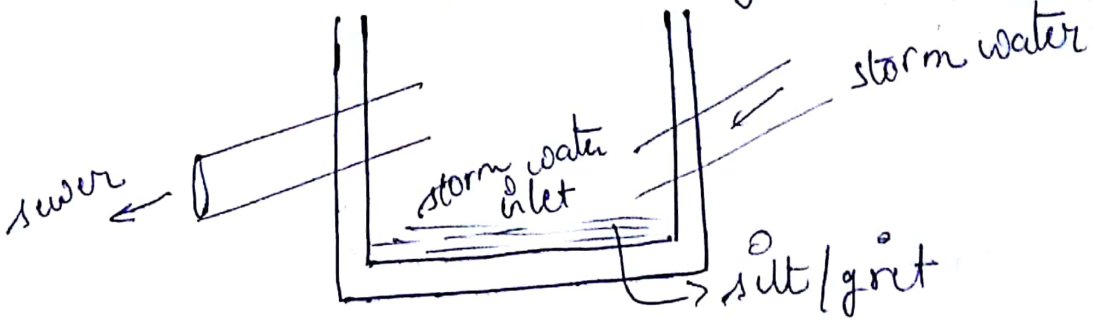
⇒ A hole or opening reaches provided in sewer for lowering a lamp inside.

3) Grease & oil trap

⇒ The traps or chamber which are generally constructed on a sewer line for removing of grease & oil from sewage.

4) Catch basin

⇒ A catch basin is rectangularchamber constructed along the sewer line to allow storm water to the sewer by eliminating the silt, grit etc



Ch-9 Sewage treatment

⇒ Waste water can be defined any water or liquid that contains impurities & pollutants, in form of solid, liquid, gas or their combination. In such a concentration that is harmful to dispose.

Treatment methods

Usually physical, chemical & ~~&~~ biological treatment (is used) the means are applied for treatment of swage of wastewater based on these treatment methods may be classified into two processes —

1) Unit operation
2) Unit process

1) Unit operation

⇒ The mean of treatment in which applications of physical forces are predominant are known as unit operation.

Eg - Screening, mixing, sedimentation etc.

2) Unit process

The type of treatment in which removal of any impurity or contaminants is achieved either ~~eg~~ chemically or biologically is called unit process.

Based on the type of agent unit processes are of two types-

a) Suspended Growth process

Ex - activated sludge process, aerated lagoon, oxidation pond etc.

b) Attach Growth process

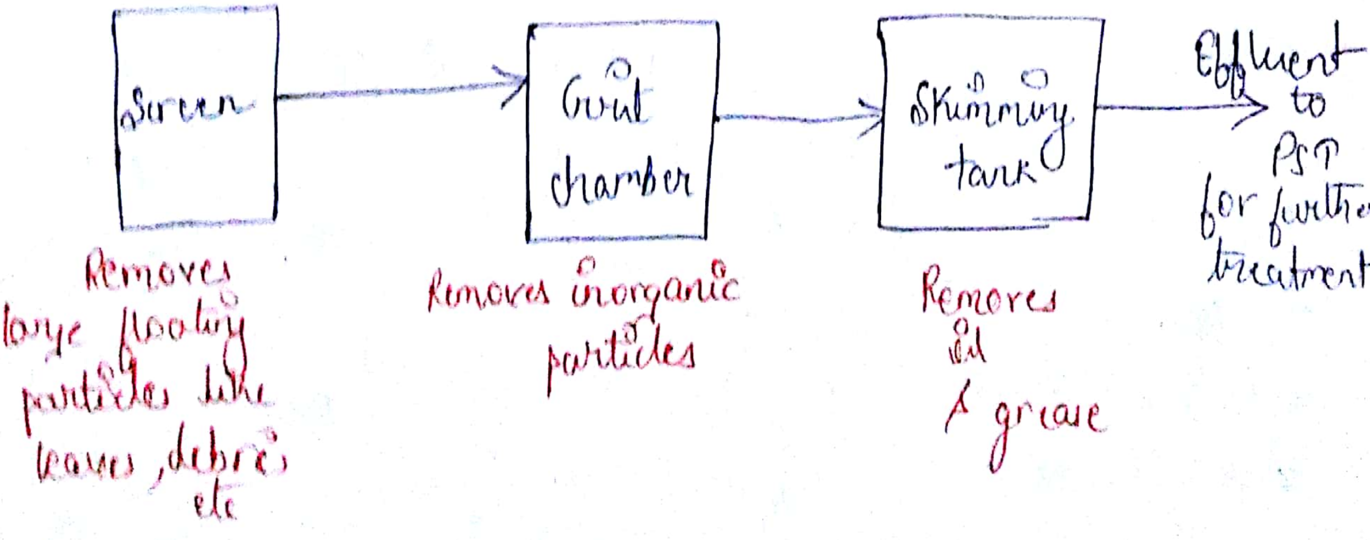
Ex - Trickling filter, Rotating Biological Contractors, Biotowers etc.

The type of combination ~~used~~ of ~~a~~ treatment process used for treatment of water & ~~for~~ called treatment system.

1) Preliminary Treatment system

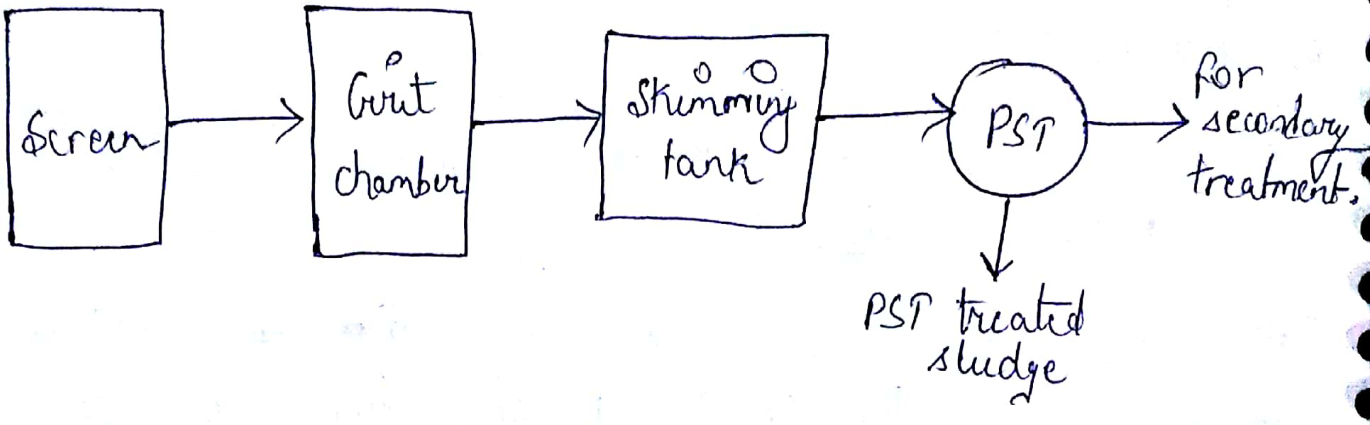
⇒ This system is mainly selected to remove floating materials & large inorganic matters of waste water.

⇒ The preliminary treatment system consist of screening, Grit chamber & skimming tank (to remove oil & grease).



2) Primary Treatment System

⇒ It includes all the ~~units~~ treatment units of preliminary treatment system & plain sedimentation tank (PST).



⇒ The purpose of primary treatment is to remove suspended solids from the influent ~~series~~ sewage.

⇒ It will not remove soluble or ~~organic~~ colloidal matter.

⇒ large debris are removed by screens.

⇒ Inorganic particles are removed by grit chamber

⇒ organic suspended solids are removed by Primary sedimentation tank.

⇒ Disposal of inorganic matter is ~~&~~ convenient bcoz it does not de-compose but disposal of organic matter is difficult bcoz of decomposition hence it is necessary or desirable that inorganic or organic matter are removal in different chamber.

imp ⇒ Primary treatment removes about 60-70% of suspended solids & about 30% of BOD.

3) Secondary Treatment System

⇒ After primary treatment wastewater is further treated for removal of colloidal & soluble organic matter it is called Secondary Treatment.

⇒ Secondary treatment system consist of biological conversion of dissolve & colloidal organic matter into biomass that can be removed by secondary sedimentation.

Note Biomass – Total quantity or weight of organics in a given volume or ~~of~~ area.

⇒ contact b/w microorganism (bacteria) & organic matter is optimised by suspending the biomass in the ~~contact b/w micro-organisms~~ waste water or passing the wastewater ~~of~~ over a film of biomass attached to a solid surface.

⇒ Most common suspended biomass is activated sludge process.

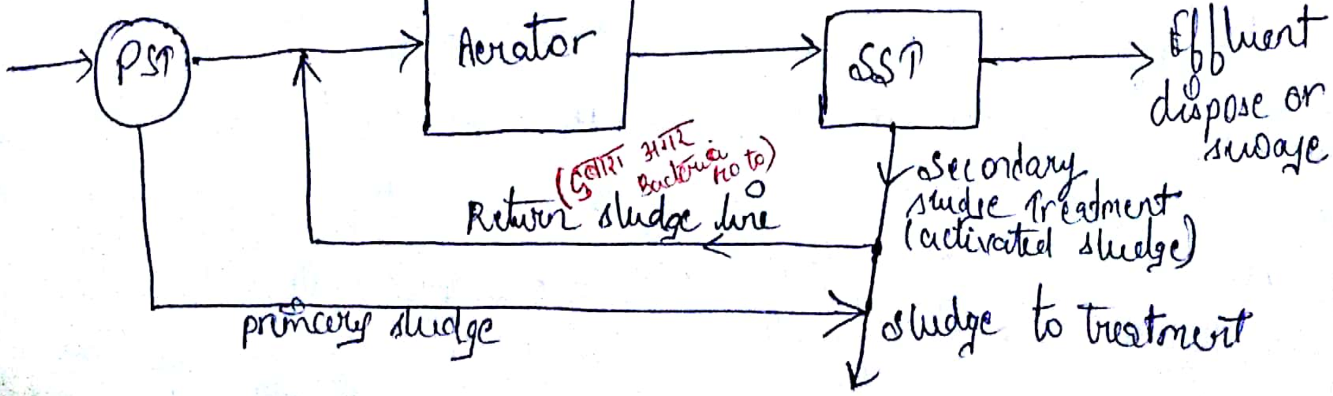


Fig:- Secondary Treatment system with activated sludge process.

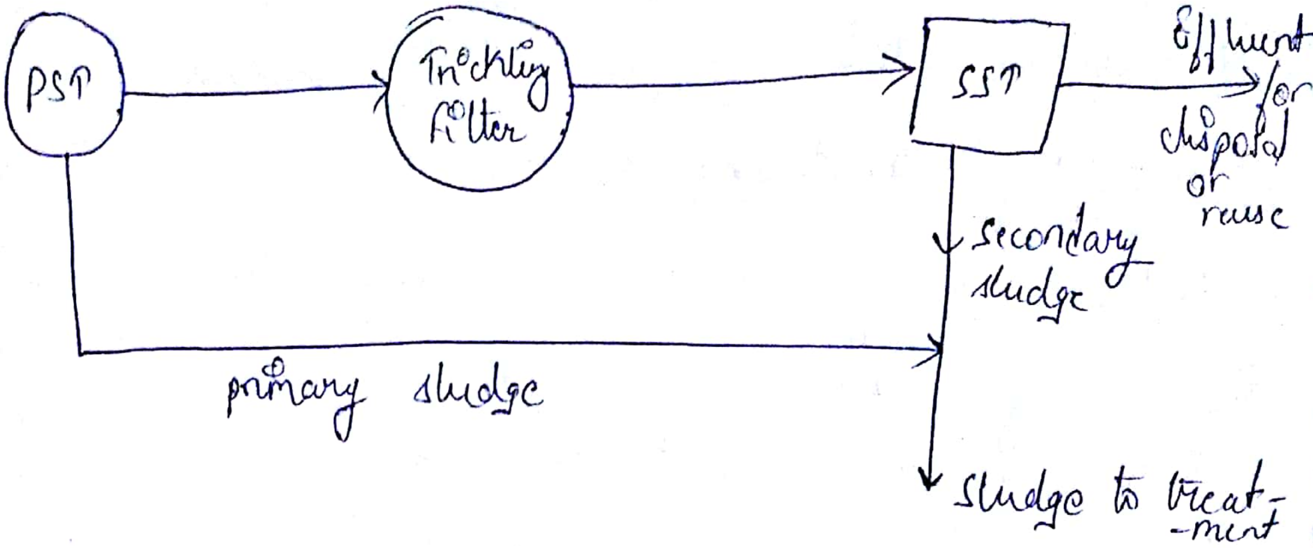


Fig:- Secondary Treatment system with Trickling filter.

⇒ An activated sludge process the part of settled sludge in secondary clarifier / secondary sedimentation tank is recycle back to aerator.

⇒ This re-circulating portion of biomass contains a large no' of microorganisms so it will speed up the conversion process.

⇒ The classical attatch biomass system is trickling filter in which stones or other solid media are used to increase or provide the surface area for the growth of bio film.

⇒ Secondary system produce excess biomass that is biodegradable.

⇒ Secondary sludge is usually combined with primary sludge for further treatment by Anaerobic biological process, the results are gaseous end products, liquids & inert (निष्क्रिय) solids.

⇒ The methane has significant heating value & may be used to meet part of power requirements to the treatment plant.

⇒ The liquid containing large conc. of organic compounds / matter is recycle back to treatment plant.

Primary Treatments

1) Screening

⇒ It is used for removal of certain materials such as peices of wood, floating debris, leaves etc

⇒ Primary purpose of screens is to give the protection to the pumps or other mechanical equipments from these impurities.

⇒ Hence screening is Normally the first operation performed for treatment of sewage.

⇒ Depending upon spacing screens may be classified as —

(i) Coarse screens — above 50 mm
(ii) Medium screens — 20-50mm
(iii) Fine screens — less than 20mm

Bars are usually 10mm thick

⇒ Screening devices generally adopted before grit chamber & pumps.

⇒ Head loss through screen may be given as

~~v = avg vel~~

$$\boxed{H_L = 0.0729(V^2 - v^2)}$$

(in m)

V = velocity throug opening of screen m/s

v = approch velocity in u/s channel

2) Grit Chamber

⇒ Grit removing devices are locating at either before or after pumping station.

⇒ In sewage treatment plant, to prevent the clogging of pipelines, channels etc due to settling of these grits.

⇒ The also assist in prevention of grit accumulation in digesters & clarifiers.

⇒ Grit chambers should not allow the settling of organic materials, velocity should such that organic matter do not settle or if it has settled velocity should be sufficient to scour this settleable organic matter.

⇒ They are provided in form of channels of longer length.

a) Design criteria of Grit chamber

⇒ It removes particle of size ≥ 2mm

⇒ Sp. Gravity of Grit is usually in the range of 2.4 — 2.65, adopt 2.65 if not given.

⇒ For particles of size 0.2mm settling velocity is 0.025 m/s

⇒ Surface over flow rate V_s (Q/A) = 2160 m³/m²/day

⇒ Stokes law is not applicable for grit chamber.

⇒ Velocity of discrete particles can be determin using the appropriate equation depending upon the reynolds no.

$$\boxed{V_s^2 = \frac{4}{3}\frac{(\gamma_s - \gamma_w)d}{C_D \cdot \rho_w}} \text{ ——— } ①$$

$$\boxed{C_D = \frac{18.5}{(Re)^{0.6}}} \quad \boxed{Re = \frac{V_s \cdot d}{\nu}} \rightarrow \text{Kinematic viscosity}$$

By putting value of C_D in eqn

$$\boxed{\underset{(m/s)}{V_s} = \left[0.707(G_s - 1)\, d^{1.6}\, \nu^{-0.6}\right]^{0.714}}$$

⇒ Horizontal velocity of flow is 0.15 – 0.3 m/s (for particle size ≥ 0.2m

⇒ Horizontal critical flow velocity -

$$\boxed{V_c = K_c\sqrt{g(G_s - 1)d}} \quad \boxed{K_c = 3 - 4.5 \text{ Generally 4 is adopte}}$$

⇒ Detention time = 40 - 60 sec

⇒ Depth = 1 - 1.5 m

⇒ Free board - 0.3 m

⇒ length of channel is increased by 20% for inlet & outlet turbulence

⇒ Loss of Head in Grit chamber varies from 6 - 60 cm depending upon particles velocity

⇒ Efficiency $\boxed{\eta = \frac{V_s}{V_o}}$

V_s = settling velocity

V_o = surface overflow rate

⇒ Settling velocity of Grit particles in transition zone is also ~~fall~~ calculated by—

$$\boxed{V_s = 60.6(G_s-1)d\,\frac{3T+70}{100}}$$ → Hazen modified formula

where d in cm – particle dia.
T = temperature °c
V_s = velocity (cm/sec)

Q Design grit chamber to remove particles of diameter 0.2mm & specific Gravity 2.6 Settling velocity of particles are in range of 0.0160 to 0.020 m/s The proportioning weir will having a velocity of 0.28 m/s, The max waste water flow is expected to be 12000 m³/day Assume depth = 1m = H

Sol^n

Horizontal velocity = 0.28 m/s

Settling velocity = 0.0160 – 0.020

Avg V_s = 0.018 m/s

$$\text{C/s area } (B \times H) = \frac{\text{Discharge}}{\cancel{V}\ \text{Hori.}}$$

~~Discharge~~ $$B \times H = \frac{12000\ m^3}{\overset{0.28}{\cancel{0.018}} \times 86400}$$

$$B \times H = \cancel{7.71\ m^2}\ 0.49\ m^2$$

$$\boxed{\cancel{B = 7.71m}}$$

$B = 0.49 \simeq$
$B = 0.5m$

$$\text{Detention time} = \frac{H}{V_s}$$

Detention time = 55.55 sec

Volume = Discharge × D.P

$$= \frac{12000}{86400} \times 55.55$$

Volume = 7.71 m^3

B × H × L = 7.71

0.5 ~~7.71~~ × H × L = 7.71

~~$L = \frac{7.71}{7.71} = 1m$~~

$$L = \frac{7.71}{0.5} = 15.42 m$$

we take for increment of 20% = ~~15~~ 15.42 + 15.42 of 20%

= 15.42 + 3.084

$$\boxed{L = 18.50 m}$$

Imp

Q Design grit chamber for flow is equal to 40MLD
G_s = 2.65 size of particles to be removed 0.2mm
D = 1×10^{-2} cm^2/s find (i) settling velocity of 0.2mm size particles (ii) critical horizontal flow velocity
(iii) size of grit chamber.

Sol K_c = [illegible] 4

Soln

$$V_s = \left[0.707\,(G_s-1)\,d^{1.6}\,\nu^{-0.6}\right]^{0.714}$$

~~$V_s = \left[0.707\,(2.65-1)\,(\overset{0.02}{20})^{1.6}\,(1\times10^{-2})^{-0.6}\right]^{0.714}$~~

$$V_s = \left[0.707\,(2.65-1)\,(0.2\times10^{-3})^{1.6}\,(10^{-2}\times10^{-4})^{-0.6}\right]^{0.714}$$

$V_s = 0.0246$ m/s $= 24.6$ mm/s

Critical horizontal flow velocity

$$V_c = K_c\sqrt{g(G_s-1)d}$$

$$V_c = 4\sqrt{9.81(1.65)\,0.2\times10^{-3}}$$

$$\underline{V_c = 0.227 \text{ m/s}}$$

Let Detention time $\underline{60\text{ s}}$

$$\frac{H}{V_s} = t_d \Rightarrow \frac{H}{24.6} = 60$$

$$H = \frac{60\times24.6}{1000} = \underline{1.47\text{ m}}$$

length $= V_c \times t_d$

$= 0.227\times60 = \boxed{13.62\text{ m}}$

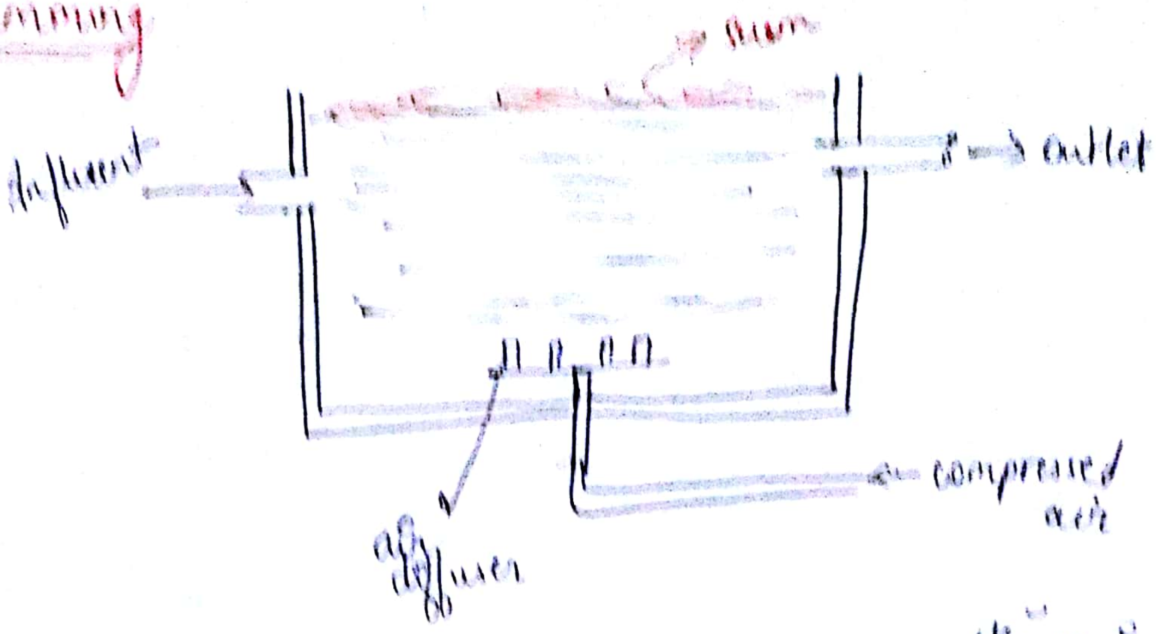
20% extra $L_{adopt} = 1.2\times13.62 = \underline{16.34\text{ m}}$

$$Q\times t_d = L\times B\times H$$

$$\frac{40\times10^{6}\times10^{-3}}{86400}\times60 = 13.62\times B\times1.47$$

$$\boxed{B = 1.38\text{ m}}$$

3) Skimming



⇒ For removal of oil & grease, soap skimming tank is provided.

⇒ It is provided before sedimentation tank

⇒ In skimming tank compressed air blown from below. The rising air coagulates the grease cause to rise to the surface from where it can be skimmed off.

4) Plain sedimentation

⇒ Depending upon the tendency of suspended particle to interact with each other & concentration of suspended matters.

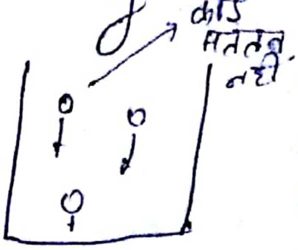
Following types of settling my occur in sedimentation tank —

a) Discrete settling (Type-1 settling)
b) Flocculant settling (Type-2 settling)
c) Hindered or zone settling (Type-3 settling)
d) Compression settling (Type-4 settling)

a) Discrete settling (Type 1 settling)

- ⇒ Shape, size & mass of particle remains same
- ⇒ Settling is independent of each particle
- ⇒ Stoke's law is valid (0.0002 – 0.2 mm)
- ⇒ Settling velocity can be also calculated using transitional law

Ex → settling of particles in grit chamber

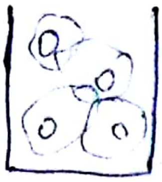


b) Flocculant settling (Type 2 settling)

- ⇒ size, shape & mass changes during settling
- ⇒ Theoritically settling velocity cannot be calculated
- ⇒ Flocs are form this occurs when concentration of particle is less than 1000 mg/lt

Ex → primary settling tank where organic particles settle.

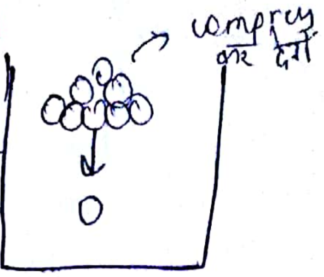
c) Hindered or Zone settling (Type 3 settling)



- ⇒ This type of settling observed when conc. of particles in water is in medium range, such that velocity zone of any particle during settling overlap the velocity zone of other particle (conc. about 1500 mg/lt approx)
- ⇒ Particle settle in the form of zone in order to maintain there relative position with respect to each other.

Ex – Settling in secondary sedimentation tank followed by ASP (activated sludge process)

4) Compression Settling (Type 4 settling)



⇒ Conc. of particles is very high such that they are in physical contact with each other (conc. > 1800mg/L)

⇒ Bottom layer of ~~these each~~ these particles supports the weight of upper layer due to which any further settling results by compression. ~~of~~

5) Primary Sedimentation Tank

⇒ Overflow rate of the PST varies b/w 25-30 $m^3/m^2/$da
(avg).

⇒ Peak overflow rate 50-60 $m^3/m^2/day$

⇒ Detention time – 2-2.5 hrs ✓ } for Pre.

⇒ Depth of tank – 2.5-3.5 m ✓

⇒ Horizontal flow velocity – 0.3 m/min

⇒ width = 6m (approx)

⇒ length – 4-5 times of width

⇒ surface area for avg. flow = $\frac{\text{Avg. discharge}}{\text{Avg. over flow rate}}$

⇒ surface area for peak flow = $\frac{\text{Peak discharge}}{\text{Peak overflow rate}}$

6) Sedimentation with coagulation

It is generally not used for sewage treatment bcoz –

a) Chemical added in this process may destroy certain bacteria which are useful for sludge digestion.

b) It is costly

c) large sludge volume produce in this process, is difficult to remove.

Note Coagulation process removes phosphate from sewage which may help in controlling eutrophication of growth.

Secondary Treatments (Biological treatment)

⇒ Secondary treatment generally carried out aerobically it leads to stable end products

⇒ No foul gases are evolved & rate of re-aeration is faster.

⇒ Unit based on aerobic treatment are –

1) Trickling filter
2) Activated sludge process
3) Oxidation pond

⇒ Unit base on anaerobic treatment are –

A) Septic tank
B) Imhoff tank.

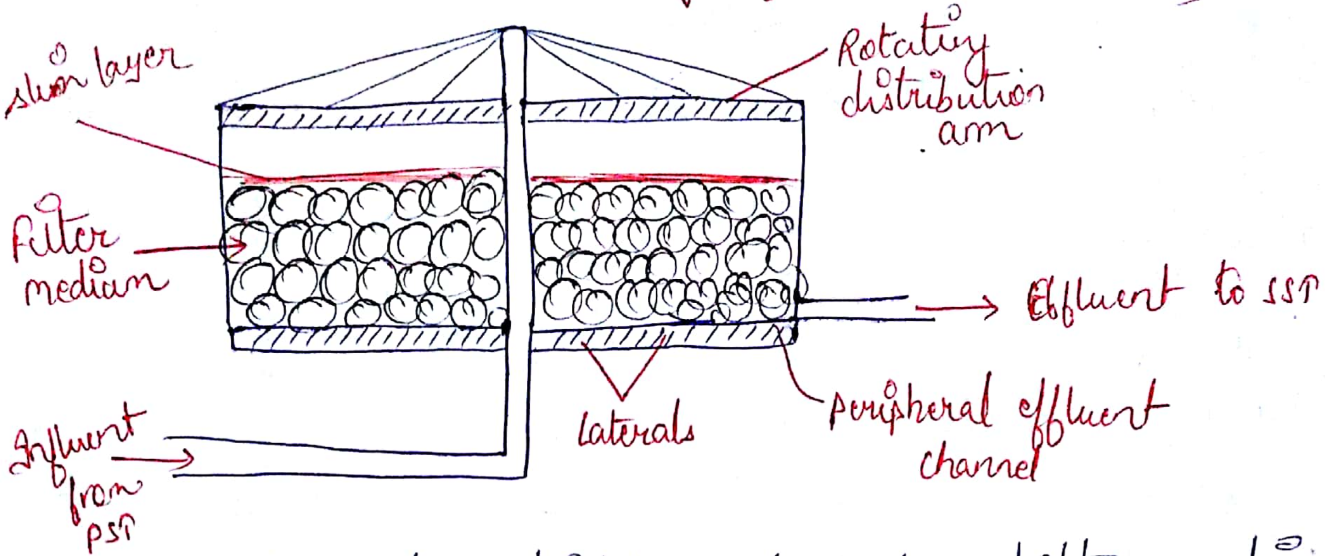
Attatch Growth System & Suspended Growth system

⇒ In attatch growth system biomass is attatched to a medium & sewage containing organic matter is passed through a medium.

⇒ In suspended growth system, biomass is in suspension of liquid containing organic matter.

Trickling Filters (Attatch Growth system)

(Always design for avg discharge)



⇒ As waste water trickles through filter medium biomass layer grows & attatch the medium surface making filter ready for operation.

⇒ When waste water flows through this biomass layer organic matter present in it comes in contact with microorganisms present in biomass layer which carries out oxidation of organic matter. resulting in formation of biomass which gets attatch over the filter medium.

⇒ The layer in which this process takes place is called slim layer.

⇒ The thickness of slim layer varies from 0.1-2mm.

⇒ In top surface of this layer aerobic process takes place (upto 0.1- 0.2mm) & in remaining portion Anaerobic process takes place.

-> Over a period of time scarcity of food & oxygen takes place at bottom part of this layers due to inc in thickness of slim layer leading to endogeneous respiration at the bottom of slim layer, which will result in increase in concentration of dead cells.

⇒ Presence of dead cell mass weakens the bond b/w medium particles & biomass layer so resulting in its sloughing (removal).

⇒ This slough biomas is ~~essentially~~ finally taken to SST for removal.

⇒ The rate of removal of organic matter ~~fro~~ from the system depends upon –

a) hydraulic loading rate (flow rate)
b) Organic loading rate
c) Temperature (inc. in temp. increases rate of removal)

⇒ Higher organic loading leads to rapid growth of biomass & this excessive growth may result in ~~to~~ clogging.

Operational Trouble in standard rate filter

1) **Filter media ponding & clogging**

⇒ It is caused by excessive organic loading in inappropriate ~~&~~ hydraulic loading rate & inadequate choice of packing of filter media.

⇒ Remedy may be washing of filter by high pressure stream of water at the surface.

⇒ Or by adding $CuSO_4$ or chlorinating the sewage.

2) Excessive Fly breeding

⇒ It can be reduced by flooding the filter for about 24 hrs, by chlorinating or by adding anti-insecticides.

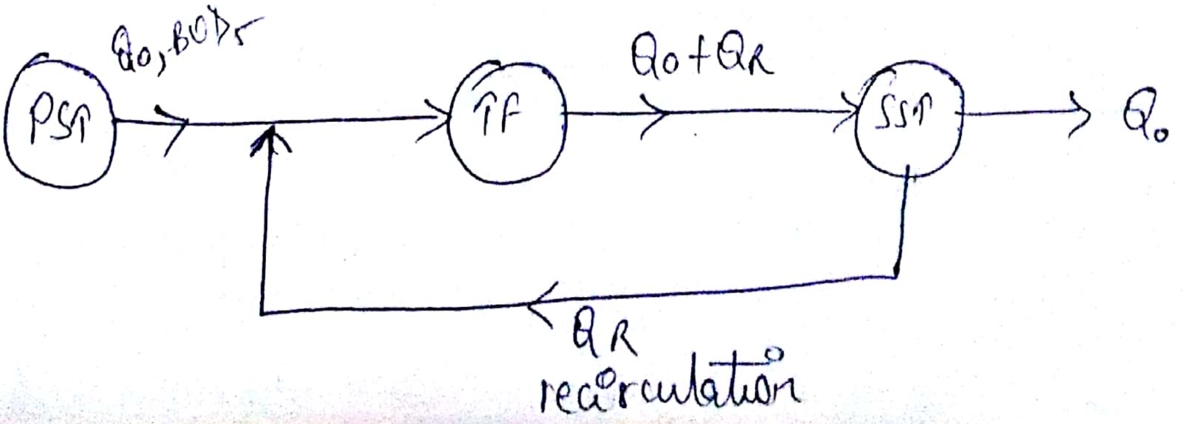
3) Odour

⇒ It can be partially controlled by providing recirculation & by providing facility to ventilation.

Design data for trickling Filters

Parameters	Standard trickling filter	High rate trickling filter
1) Hydraulic loading rate ($m^3/m^2/day$)	1 – 4	10-40 (including recirculation)
2) Organic loading ($Kg\ BOD_5/m^3/day$)	0.08 – 0.32	0.32 – 1 (excluding recirculation)
3) Depth	1.8 – 3	0.9 – 2.5
4) Recirculation ratio Q_R/Q_0	0	0.5 – 3

High Rate trickling Filter



⇒ All others things are same except that re-circulation is done by pumping effluent.

⇒ Recirculation has advantage of bringing the organic matter in the waste in contact with biological film more than once.

⇒ Thus w/o compromising ~~at~~ the Quality of effluent, hydraulic loading rate will increase.

⇒ High hydraulic loading rate reduces chances of clogging.

Efficiency of Trickling Filter (Standard Rate Trickling Filter)

As per GOI manuals

$$\boxed{\eta\% = \frac{100}{1+0.0044\sqrt{u}}}$$

u = organic loading rate (kg/hect-m/day).

$$\boxed{\eta\% = \frac{100}{1+0.44\sqrt{u}}}$$

u = organic loading rate (kg/m³/day)

Eff for High rate trickling filter

Stage 1

$$\boxed{\eta_1 = \frac{100}{1+0.0044\sqrt{\frac{W_1}{V_1F_1}}}}$$

W_1 = amt. of BOD entering in stage 1 (kg/day) $= Q_0S_0$

V_1 = Vol. of filter in stage 1 (ha-m)

$$\boxed{F_1 = \frac{1+R}{(1+0.1R)^2}}$$ F_1 = recirculation factor

$$\boxed{R = \frac{Q_R}{Q_o}}$$

For stage 2

$$\boxed{\eta_2 = \frac{100}{1+\frac{0.0044}{1-\eta_1}\sqrt{\frac{W_2}{V_2 F_2}}}}$$

$W_2 = W_1(1-\eta_1)$

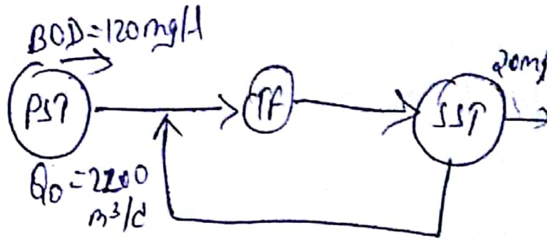
Overall efficiency

$$\boxed{\eta = \eta_1 + (1-\eta_1)\cdot\eta_2}$$

Q Calc. the diameter required for single stage trickling filter, which is to yield effluent BOD of 20mg/l when treating settled domestic sewage of BOD of 120mg/l. Waste water flow is 2200 m³/day & Recirculation discharge (Q_R) is 4000 m³/day. Depth of tank is 1.5m.

Solⁿ

$$\eta = \frac{100}{1+0.0044\sqrt{\frac{W_1}{V_1 F_1}}}$$



$$\eta = \frac{120-20}{120}\times 100 = 83.33\%$$

$S_o = 120$ mg/l
$S_e = 20$ mg/l
$Q_o = 2200$ m³/day
$Q_R = 4000$ m³/day
$h = 1.5$

$$R = \frac{Q_R}{Q_o} = \frac{4000}{2200} = 1.81$$

$$F_1 = \frac{1+1.81}{(1+0.1\times1.81)^2} = 2.01$$

~~$W_1 = Q_0 Y_0 = \frac{120 mg}{l} \times \frac{2200 m^3}{d} \times 10^{-3} \frac{l}{m^3}$~~

~~$= 1200 \times$~~

$W_1 =$ BOD$_5$ in kg/day

$$W_1 = \cancel{2200} \cdot \frac{120 \times 10^{-6} \times 10^{3} \times 2200}{kg/m^3 \times m^3/day}$$

$W_1 = 264$ kg/day

$$\eta_1 = \frac{100}{1 + 0.0044\sqrt{\frac{264}{V_1 \times 2.01}}}$$

$$\eta_1 = \frac{100}{1 + \frac{0.050}{\sqrt{V_1}}} \Rightarrow 83.33 = \frac{\sqrt{V_1}\, 100}{\sqrt{V_1} + 0.050}$$

~~$83.33 V_1 + \overset{42.02}{4.166} = V_1 100$~~

$V_1 = 0.063$ ha-m

$= 0.063 \times 10^4 m^3 = 630 m^3$

$H = 1.5 m$

$$\text{Area} = \frac{630}{1.5} = \frac{V}{H} = 420 m^2$$

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

$$420 = \frac{\pi}{4} D^2$$

$$\boxed{D = 23.12 m}$$

Note Surface area = $\frac{Q_0}{\text{hydraulic loading Rate}}$ [excluding recirculation]

$= \frac{Q_0+Q_R}{HLR}$ (including re-circulation)

Volume = $\frac{Q_0 S_0}{\text{Organic loading rate}}$ (excluding re-circulation)

Volume = $\frac{Q_0 S_0 + Q_R S_R}{OLR}$ (~~And~~ Including recirculation)

Q Population of town is 30,000, domestic sewage produce is 120lpcd. having BOD of 200mg/l, industrial sewage produced is 3×10^5 l/day having BOD 800mg/l. Design high rate single stage trickling filter with following data -

BOD removal in PST - 35%

organic loading rate - 10000 kg/hac·m/day (excluding re-circulation)

Hydraulic loading rate - 170×10^6 l/ha/day (including re-circulation)

Recirculation ratio = $Q_R/Q_0 = 1$

Find efficiency of trickling filter

Initially BOD_{mix}

$$\frac{Q_s S_s + Q_I S_I}{Q_s + Q_I} = \frac{30000 \times 120 \times 200 + 3 \times 10^5 \times 800}{120 \times 30000 + 3 \times 10^5}$$

$$= 246.5 \text{ mg/l}$$

After PST BOD $= 0.65 \times 246.15$
35%.
$$= 160 \text{ mg/l}$$

$$\text{Vol. of T.F} = \frac{Q_o S_o}{OLR}$$

$$Q_o = 3 \times 10^5 + 30000 \times 120$$
$$= 39 \times 10^5 \text{ l/day}$$

→ लाव डाला

$$V = \frac{39 \times 10^5 \text{ l/d} \times 160 \text{ mg/l} \times 10^{-6} \text{(kg)}}{10{,}000 \text{ kg/hac-m/day}} = 0.0624 \text{ ha-m}$$

$$V = 0.0624 \times 10^4 = \boxed{624 \text{ m}^3}$$

$$\text{Area} = \frac{Q_o + Q_R}{HLR} \qquad \{R = 1 \Rightarrow Q_o = Q_R\}$$

$$\text{Area} = \frac{2Q_o}{HLR} = \frac{2 \times 39 \times 10^5 \text{ l/day}}{170 \times 10^6 \text{ l/ha/day}} = 0.0458 \text{ ha}$$

$$\boxed{\text{Area} = 458 \text{ m}^2}$$

$$\frac{\pi}{4} D^2 = 458$$
$$D = 24.17 \text{ m}$$

$$\text{Depth} = \frac{V}{A} = \frac{624}{458} = 1.36 \text{ m}$$

Efficiency $\eta = \dfrac{100}{1 + 0.0044\sqrt{\dfrac{W_1}{F_1 V_1}}}$

$W_1 = \text{BOD (kg/day)}$

$= 160 \times 10^{-6}\ \text{kg/l} \times 39 \times 10^{5}\ \text{l/day}$

$W_1 = 624\ \text{kg/day}$

$V_1 = 0.0624\ \text{ha-m}$

$F = \dfrac{1+R}{(1+0.1R)^2} = 1.65$

$\eta = \dfrac{100}{1+0.0044\sqrt{\dfrac{624}{1.65 \times 0.0624}}}$

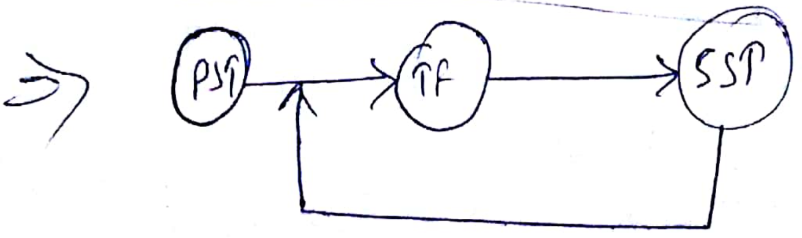
$\boxed{\eta = 74.48\%}$

Note If asked BOD of effluent.

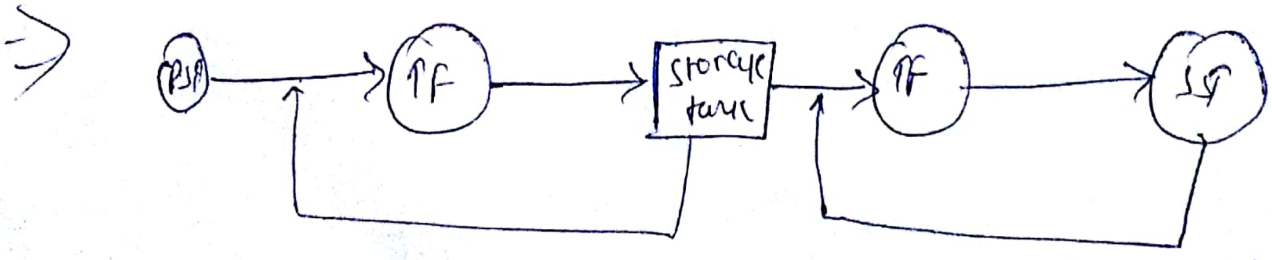
$\eta = \dfrac{S_o - S_e}{S_o} \times 100$

$74.62 = \dfrac{160 - S_e}{160} \times 100$

$\boxed{S_e = 40.60\ \text{mg/l}}$



⇒ single stage



⇒ Double stage

Q Determine dimension of high rate trickling filter for following Data - Sewage flow 3 MLD

Recirculation ratio = 1.5

BOD of raw sewage = 250mg/l

BOD removal of primary tank = 25%

Final effluent BOD desired = 30 mg/l

By what % of diameter of filter to be modified if it is to be designed as standard rate trickling filter for above requirement. Assumed = 1.5m

Sol^n For single stage HRTF

$$R = \frac{Q_R}{Q_0}$$

$$Q_0 = 3MLD \Rightarrow 3\times10^6 \frac{L}{D} = \frac{3\times10^6\times10^{-3}}{86400}$$

$$Q_0 = \cancel{0.034\ m^3/s}\ 3000\ m^3/day$$

$$Q_R = 1.5 \times \cancel{0.034}\ 3000$$

$$Q_R = \cancel{0.052\ m^3/s}\ 4500\ m^3/day$$

$$F = \frac{1+R}{(1+0.1R)^2} \Rightarrow \frac{1+1.5}{(1+0.1\times1.5)^2} = F = 1.89$$

$$\eta = \frac{100}{1+0.0044\sqrt{\frac{W_1}{V_1F_1}}}$$

$$\text{Total BOD} = 3000\times250\times10^{-6}\times10^3 = 750\ kg/day$$

~~3000 × 250~~

BOD removal in PST = 25%.

Incoming BOD = 0.75 × 750 = 562.5 kg/day

permissible effluent =

$$\eta = \frac{\cancel{250}\ 0.75 \times 250 - 30}{0.7 \times 250} \times 100$$

$\eta = 84\%$.

$$84 = \frac{100}{1 + 0.0044\sqrt{\dfrac{\cancel{750}\ 562.5}{V_1\ 1.89}}}$$

$$84 = \frac{\sqrt{V_1}\ 100}{\sqrt{V_1} + \cancel{19.92}\ 17.24}$$

$$\cancel{84\sqrt{V_1} + \overset{1448.93}{1673.28} = \sqrt{V_1}\ 100}$$

$V_1 = 0.158812$ ha-m

$V = 1588.12\ m^3$

Assume depth = 1.5 m

$$\text{Area} = \frac{1588.12}{1.5} = 1058.74\ m^2$$

$$\frac{\pi}{4} D^2 = 1058.74\ m^2$$

$$\boxed{D = 36.72\ m}$$

For same SRTF 0.104

$$\eta = \frac{100}{1+0.0044\sqrt{u}}$$

$$84 = \frac{100}{1+0.0044\sqrt{\frac{561.5}{V}}}$$

$V = 0.3001$ hac-m

$= 3001 m^3$

Depth = 1.5m

$$\text{Area} = \frac{3001}{1.5} = 2001 m^2$$

$$\frac{\pi}{4}D^2 = 2001$$

$$\boxed{D = 50.47m}$$

$$\% \text{ of diameter} = \frac{50.47-36.72}{50.47} = 27.24\%$$

Q A trickling filter treats 1500m³/day of sewage with a BOD of 220mg/l & a SS of 250 mg/l estimate total solid production assuming primary clarifier (PST) removes 30% of BOD & 60% of influent solids (SS) Take solid production in trickling filter at 0.5/kg of applied BOD.

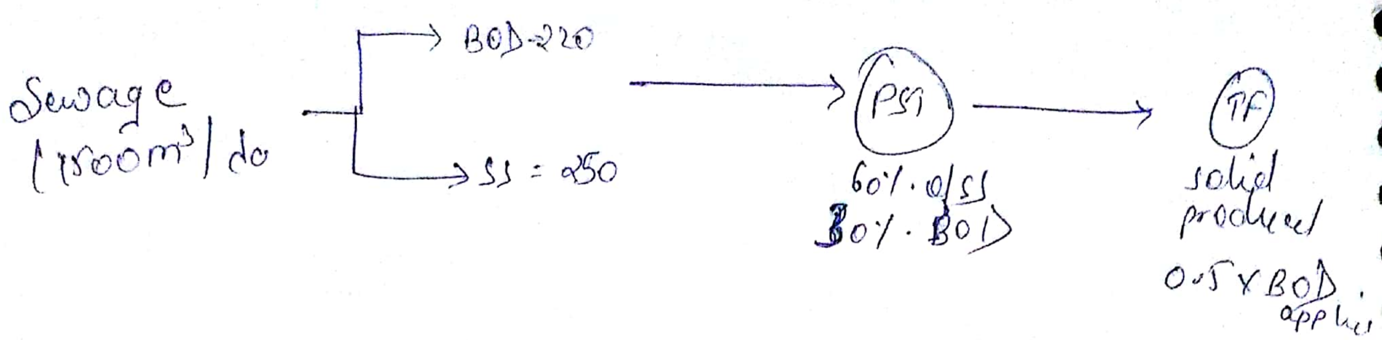
Solⁿ

Total BOD of sewage $= 1500 \times 220 \times 10^{-6} \times 10^{3}$

$= 3300$ Kg/day

Total BOD of SS $= 1500 \times 250 \times 10^{-6} \times 10^{3}$

$= 375$ Kg/day



PST
SS set होता है

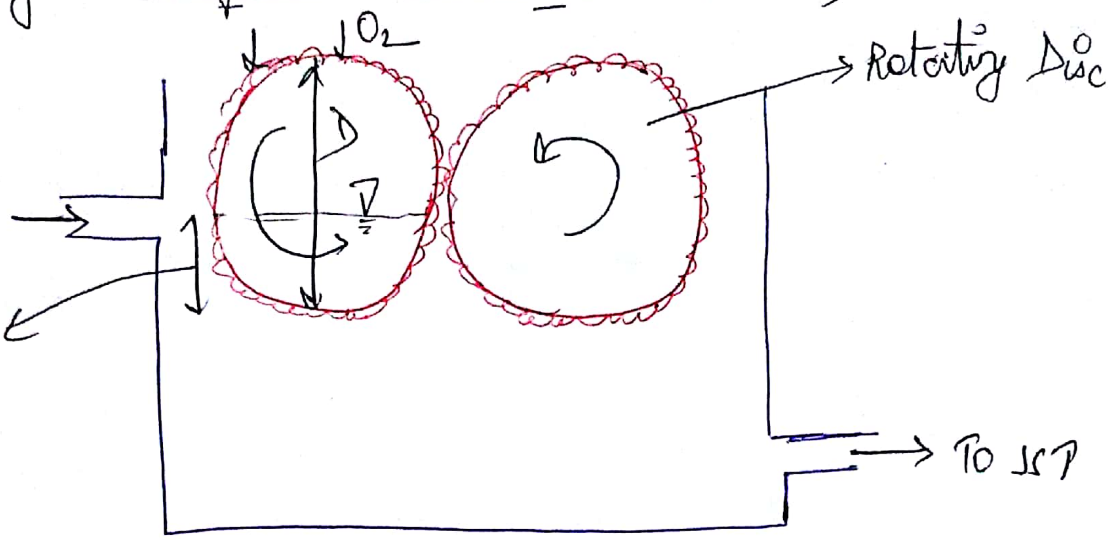
SS settled in PST = 0.6 X 375
= 225 Kg/day

Influent BOD at TF = 0.7 X 330
= ~~23~~ 231 Kg/day

Solids in TF = 231 X 0.5
= 115.5 Kg/day

Total = 225 + 115.5 = 340.5 Kg/day

Rotatory Biological Contractors (RBC)



⇒ In this case rotating disc are used which are closely spaced with each other as a medium of growth of biomass layer.

⇒ This biomass layer comes in contact with organic matter when the disc is immersed in waste water & utilizes ~~oxid~~ oxygen when comes in contact with atmosphere to oxidise organic matter, resulting in formation of Biomass which get attatched itself.

⇒ Over a period of time thickness of biomass layer over the disc increases & it is removed off due to turbulence created by rotating disc in waste water & is finally taken to SSP.

⇒ The depth of immersion of disc is kept about 0.4D dia of disc in waste water.

⇒ This system takes the advantage of both attached & suspended growth system.

Activated sludge process

Reaction Kinetics

⇒ If 'S' represents quantity of soluble food applied 'x' represents total biomass in the reactor to oxidise incoming organic matter.

⇒ MLSS (Mixed liquor suspended solids) represents both living & dead bacteria.

⇒ The entire process of biological decomposition is believe to complete in 4 stages—

a) Lag phase – ⇒ In this phase microorganisms get acclimilated (अभ्यस्त) to food & ~~envt~~ environment given to them.

⇒ If microorganisms are already familiar to this environment & food lag phase is less for them.

⇒ The growth of biomass in this phase is very less so it is called lag phase.

b) log growth phase – In log phase micro-organism reproduce rapidly by cell division leading to rapid increase in the growth of biomass & corresponding decrease in organic matter.

Note In activated sludge process we try to maintain system in log phase.

c) Stationary phase – The stationary phase represents the time during which reproduction of New microorganisms roughly offset by death of bacteria, due to endogeneous respiration.
In this zone food supply becomes limited.

d) Endogeneous phase – In this phase endogeneous respiration predominates in which biomass slowly decreases.

For Biomass – $\boxed{\frac{dx}{dt} \propto x}$ [w/o considering endogeneous respiration i.e log growth.]

$$\boxed{\frac{dx}{dt} = Kx} \quad ①$$

K = represent overall growth rate const depends on type of food, conc. & temperature

Not imp

$$\boxed{K = \frac{K_o S}{K_s + S}} \rightarrow \text{Monod's eqn}$$

K_0 = Max growth rate const

S = conc of food (mg/l)

K_s = half saturation const.

⇒ If all the food converted into biomass

$$\frac{dx}{dt} \propto -x$$

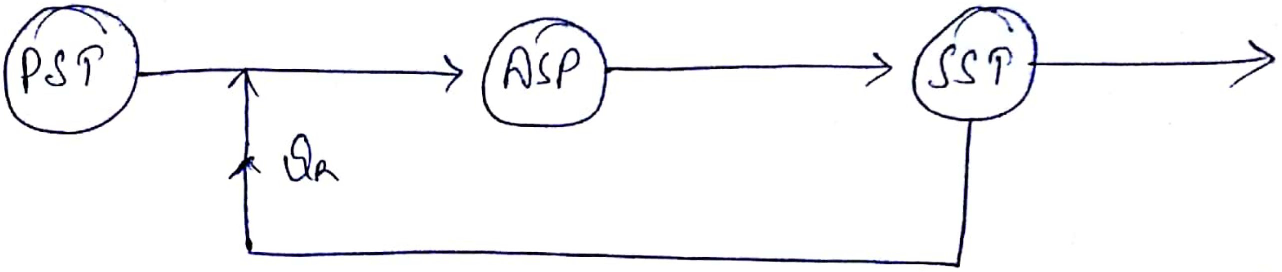
$$\boxed{\frac{dx}{dt} = -K_D \cdot x} \quad \text{— ②}$$

K_D = endogeneous decay rate const

Generally 0.06/day

For whole system

$$\boxed{\frac{dx}{dt} = \left(\frac{K_0 S}{K_s + S}\right) x - K_D x}$$



⇒ The settled sludge in secondary clarifier containing living microorganisms is return to reactor to increase the available biomass & speed up the reaction.

⇒ The return sludge from secondary sedimentation tank is active (अभी भी Bacteria बाकी हैं) so it is called activated sludge.

⇒ ASP is suspended growth system with sludge return.

⇒ The process can be—

1) completely mixed process
2) Plug flow process.

1) Completely mixed process

⇒ It is adopted for plant < 25 MLD

⇒ It utilizes completly mix flow regime.

⇒ In completely mixed process circular or square tank is used. & mixing is acheived by mechanical aerator, installed at the center of tank.

⇒ Complete mixing has a capacity to hold high MLSS

(F/M) → MLSS

⇒ Food/mass ratio & oxygen demand is uniform in completely mixed process.

2) Plug flow process

⇒ It is conventional system & this process has been adopted even for large plant upto $\boxed{\text{300 MLD}}$, ~~so~~ plug flow implies that sewage moves progressively down along aeration tank.

⇒ In this case long narrow channel is used greater than or equal to 5 times of width

⇒ In plug flow F/m ratio & oxygen demand is ~~at~~ high at inlet & will then decreased progressively.

⇒ It is ~~also~~ always preceded by primary sedimentation tank.

⇒ BOD removal efficiency 85-92%

Extended Aeration process

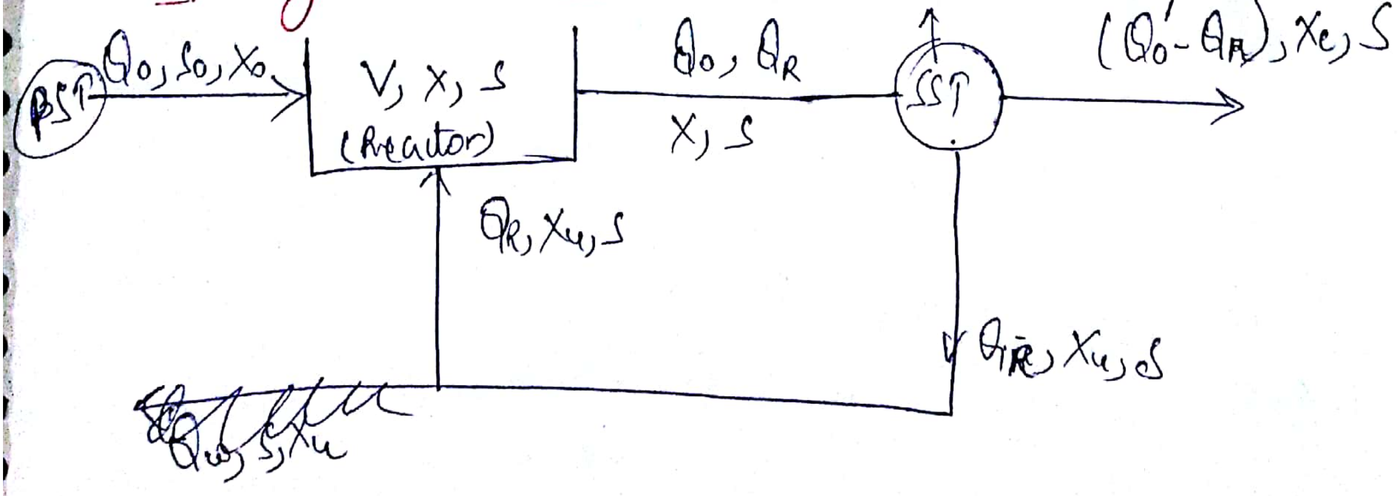
⇒ This is adopted for plant less than 4 MLD the flow scheme of exteded aeration process is completely mixed process, in this case PST is avoided.

⇒ The process employs low organic loading long aeration time high MLSS concentration & low F/m ratio.

⇒ BOD removal efficiency is high in this case.

Note This system works in endogeneous phase

Completely mixed process



Biomass + Biomass = Biomass

$$Q_0X_0 + V\left[\left(\frac{K_0S}{K_s+S}\right)X - K_DX\right] = Q_RX_u + (Q_0 - Q_R)X_e \quad ①$$

Design parameters

a) Hydraulic retention time – (θ)

$$\boxed{\theta = \frac{\text{Volume}}{\text{Rate of flow excluding recirculation}}}$$

b) Volumetric BOD loading

$$= \frac{\text{Mass of BOD applied}}{\text{Vol. of aeration tank}} = \frac{Q_0S_0}{V}$$

c) Specific substrate utilization rate (U)

$$\boxed{U = \frac{Q_0(S_0 - S)}{VX}}$$

d) Sludge ~~ad~~ age (θ_c)

$$\boxed{\theta_c = \frac{\text{Mass of MLSS in aeration tank}}{\text{Mass of MLSS leaving per day}}}$$

Imp

e) Sludge Volume Index (SVI)

SVI indicates the ~~physical~~ physical state of sludge in biological aeration system

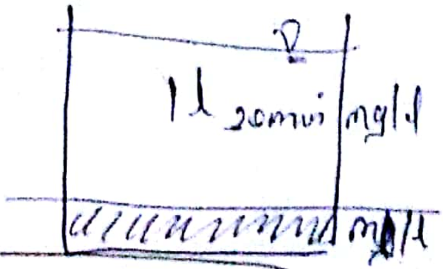
⇒SVI is defined as vol. occupied in ml by 1g of solid after settling of 30min

Its unit is ml/g

Experiment

The standard test which is performed in laboratory to compute SVI of an aeration system in volves collection of 1l sample from aeration tank near its discharge end in a graduated cylinder this 1l sample of mixed liquor is allowed to settle for 30 min & settled sludge volume (V_{ob}) in ml is recorded this volume in ml per l of mixed liquor will represent the quantity of sludge in the liquor in ml/L.

The above sample of mix liquor after remaining the settled solids is further tested in laboratory for MLSS by standard procedure adopted for measuring suspended solids in sewage let this conc. of solids in mix liquor in mg/l is (X_{ob}) then the SVI is given by



$$\boxed{SVI = \frac{ml/l}{mg/l} = \frac{ml}{mg} = \frac{ml}{g} \times 1000}$$

Note The value of SVI b/w (80-150) ml/g is considered to be safe

Q A mixed liquor with 2000mg/l of a SS has a settled volume of 200ml from a 1l of this mixed liquor cal. its SVI.

Sol^n

$$SVI = \frac{ml}{mg} \times 100 = \frac{200 \times 1000}{2000} = 100\,ml/g$$

Process Type	Flow regime	MLSS (mg/l)	MLVSS / MLSS	F/m	HRT (days)	θ_c (days)	Q_R/Q_0	η
conversional	Plug flow	1500-3000	0.8	0.3-0.4	4-6	5-8	0.25-0.5	85-92%
complete mix	completely mixed	3000-4000	0.8	0.3-0.5	4-5	5-8	0.25-0.8	85-92%
extended Aeration	completely mixed	3000-5000	0.3	0.1-0.18	12-24	10-15	0.5-1.0	92-98%

Operation

⇒ The most important aspect in the operation of activated sludge plant is to maintain proper F/m ratio, which is achieved by increasing or decreasing MLSS level in the aeration tank acc. to incoming BOD.

⇒ The MLSS in aeration tank is regulated by controlling the rate of sludge return

⇒ The excess sludge is wasted i.e taken out to the system & this is taken into sludge digestion tank.

Excess sludge wasting

⇒ It is done to maintain steady level of MLSS, in the tank excess sludge quantity will increase with increasing F/M ratio.

Nitrification

⇒ ASP plant is normally design for removal of 1st stage - BOD i.e carbonaceous BOD.

⇒ However there may be some Nitrification taking place which will consume oxygen & reduces dissolve oxygen.

⇒ This problem can be overcome by increasing sludge wasted rate leading to lesser sludge age.

⇒ Nitrification can lead to subsequent denitrificati in secondary sedimentation tank.

Sludge bulking

⇒ Sludge with poor settling characteristics is termed as sludge bulking.

⇒ It is due to inadequate air supply resulting in lower pH this leads to growth of filament organisms. ~~fil~~ filament bacteria also form due to low nutrient conc.

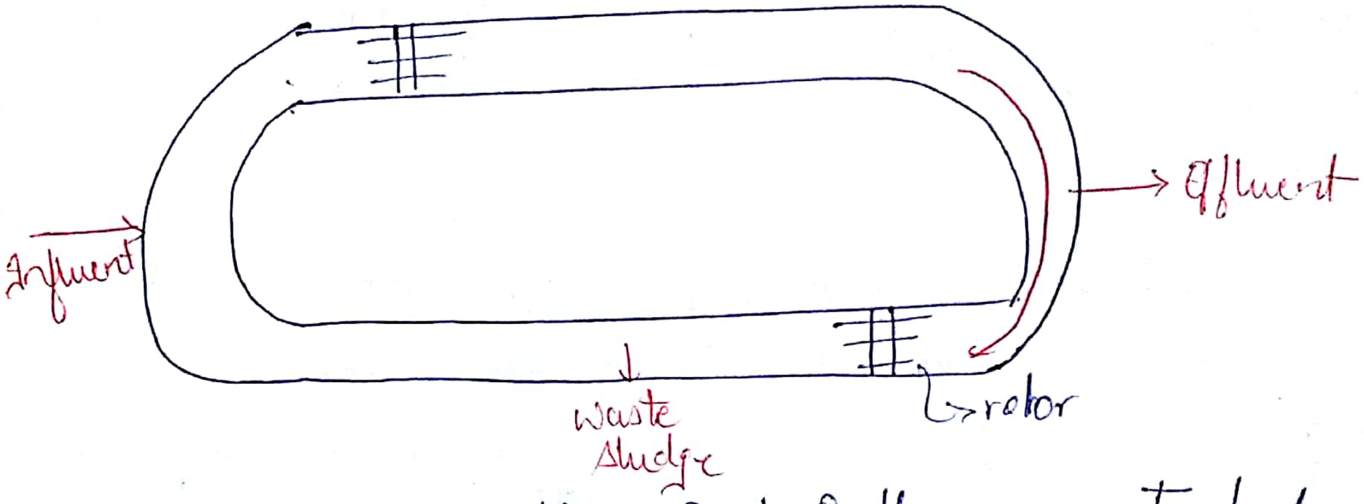
⇒ Sludge bulking can be reduced by—

(i) reducing sludge age

(ii) ~~chlorinat~~ chlorinating the return sludge.

Imp

Oxidation ditch



⇒ An oxidation ditch is basically an extended aeration system, it is designed & operated on same principle of activated sludge process

⇒ Oxidation ditch employed in field consist of oval or ring shape.

⇒ long shallow channel of about 1m depth.

⇒ A rotor or aerator is provided to supply the required oxygen to circulate the wastewater & mix content of the ditch.

⇒ Extended aeration produces a very clear effluent (low BOD). ~~as bl~~

⇒ As biological solids (Biomass) produced are destroyed completely by endogeneous respiration based on operation m/d oxidation ditch may be —

(i) Continuous flow type
(ii) Intermittent flow type

Sludge Thickner

⇒ To reduce the volume of sludge, sludge thickner is used.

⇒ Three types of thickners are used –

(i) Gravity thickner - लेयर बनती चली जाती है
(ii) Air floating
(iii) centrifugation -से

Sludge Digestion

⇒ Waste water sludge is considered hazardous to environment & ~~A~~ must be dispond ofert

⇒ The sludge of PST is called raw sludge or primary sludge.

⇒ Raw sludge is more objectionable it contains about ~~95~~ 95% moisture content.

⇒ Sludge from PST has excess organic matter & sludge from SST has excess biomass.

⇒ Sludge digestion serves both to reduce volume & to make sludge pathogen free, this may be achive by aerobically & Anaerobically.

Aerobic Digestion

⇒ It is adopted for only bio-logical sludge not for primary sludge.

⇒ It is just an extension of extended aeration process.

⇒ In aerobic digestion as there is scarcity of food, endogeneous respiration starts.

⇒ Digested sludge consist of cell wall, & other cell fragments, this process is energy consumptive.

Anaerobic Digestion

⇒ It is carried out primarily for primary sludge.

⇒ The aerobic process produces lesser biomass & also primary function of anaerobic digestor is to convert as much of sludge as possible to liquids & gases.

⇒ The waste water contains wide variety of organics so it requires more variety of biomass.

⇒ In anaerobic process there are two types of micro-organisms – i) acid forming bacteria
ii) Methane forming bacteria

(i) Acid formers

⇒ They consist of facultative & anaerobic bacteria which ~~hydrolys~~ changes the organics into acids by hydrolysis.

⇒ The soluble end product fermented (अड़ना) to acids & alcohols.

(ii) Methane formers

⇒ They are strictly anaerobic & convert the acids to methane, during sludge digestion process sludge is broken into a) digested sludge → Inert (Non-hazardous/pathogen free)
b) Supernated liquor → Clear liquid
c) Gases

Stages in sludge digestion process

1) Hydrolysis   2) Fermentation (Acidogenesys)   3) Methane formation (Methanogenesys)

1) Hydrolysis

⇒ It is the first process ~~for~~ in most of the fermentation process in which particulate matter is converted into soluble compounds.

⇒ In this stage complex organic matter like protein, cellulose, lipid etc are converted into simpler compounds or into simple soluble organic matter.

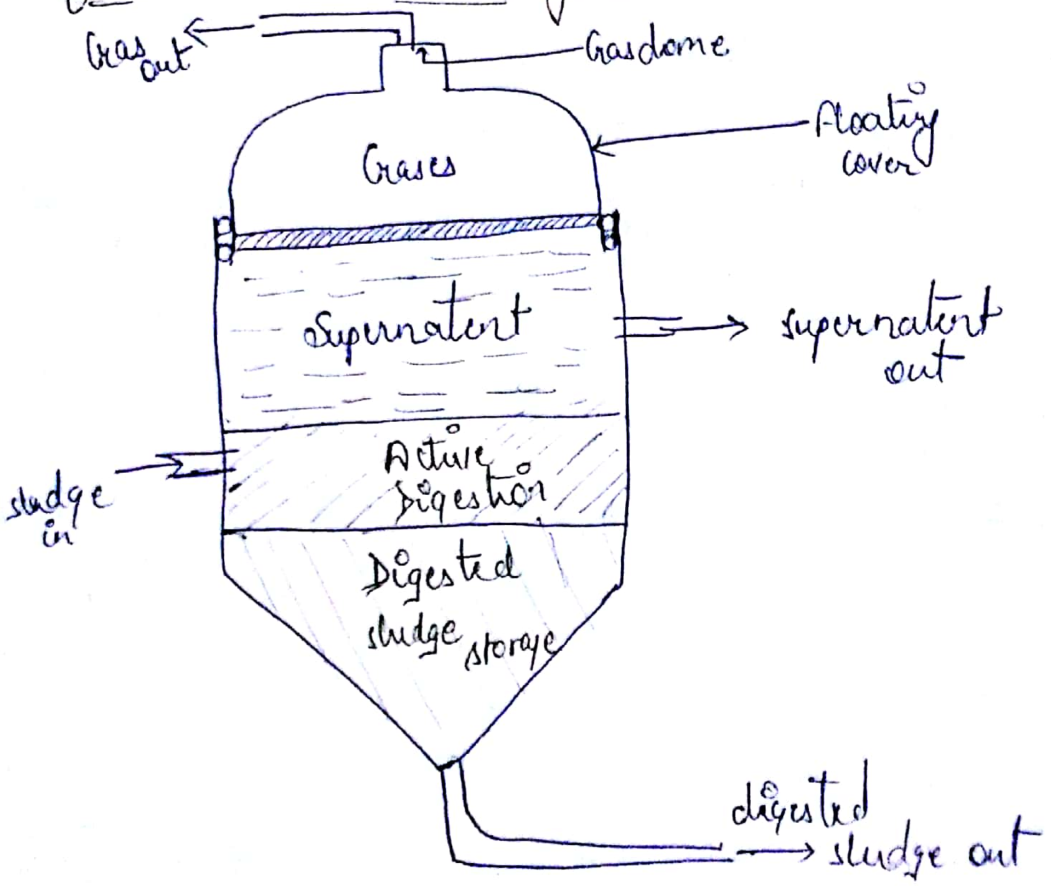
2) Fermentation (Acidogenesys)

⇒ soluble organic matter is converted by acid formers into acetic acid, hydrogen, carbon-dioxide etc.

3) Methane formation

⇒ ultimately acids are converted into methane.

Reactor for Anaerobic digestion



⇒ The reactor consist of closed tank with air tight cover, the conical bottom facilitates withdrawl of sludge & the floating cover accomodates volume change due to sludge additions withdrawl of gases etc.

⇒ The sludge is fed into digestor intermittently & supernated liquor is withdrawl the digested sludge is taken to sludge line bed.

Oxidation Pond or stabilization Pond

⇒ They are open flow through earthen basin.

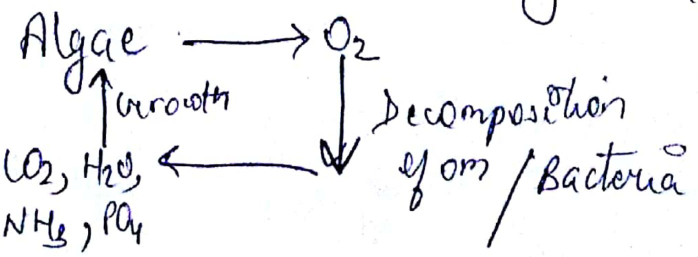
⇒ Such pond provides long detention period during which, the waste gets stablized by the action of natural forces.

⇒ In a totally aerobic pond stablization is brought by aerobic bacteria

⇒ The oxygen demand is met by algal photosynthesis or algal symbiosis.

⇒ In this symbiosis the algae while growing during sunlight produces oxygen by photosynthesis & this ~~oxid~~ oxygen is utilize by bacteria for oxiding waste organic ~~ma~~ matter.

⇒ The end product of the process are CO_2, NH_3, H_2O etc which are required by algae to grow & continue producing oxygen.



⇒ In a totally ~~or~~ aerobic pond depth should be 0·5m at max, so sunlight can penetrate upto the bottom for totally aerobic pond PST is necessary so that turbidity reduces & sunlight can penetrate upto whole depth.

⇒ Efficiency of removal BOD about 95%

⇒ For industrial sewage, anaerobic pond may be used its depth is 2·5 – 5m

⇒ The actual oxidation pond is used for domestic sewage.

⇒ The bond used infact is facultative in nature in which 3 zones etc.

⇒ These are aerobic zone (at surface), anaerobic zone (at bottom) & facultative zone (at middle).

⇒ The applied sewage organics are stablized by both aerobic & anaerobic processes in different zones.

⇒ In top aerobic layer where the oxygen is supplied by algal photosynthesis, in this case dissolve organic matter is oxidised to CO_2, ~~NO_2~~ H_2O etc.

⇒ The settled sludge mass originating from raw waste & microbial synthesis in the aerobic layer is further goes down in anaerobic zone undergoes stablized to CH_4 etc.

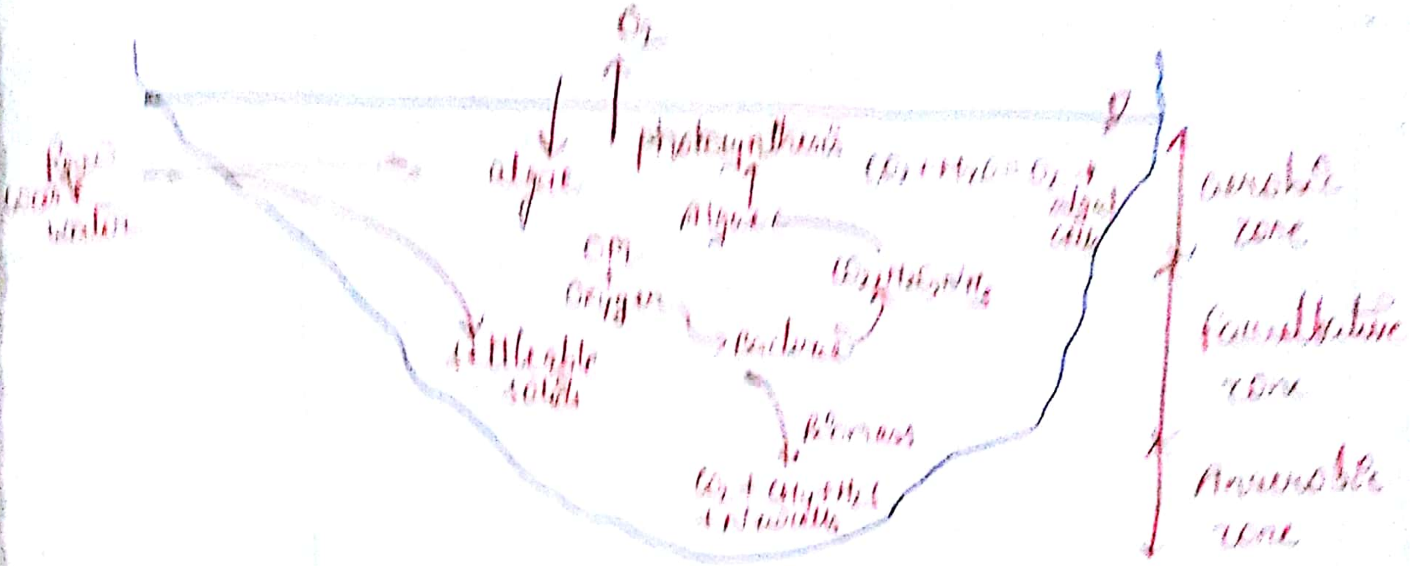


Fig – Removal of organics through facultative oxidation pon

Design parameters for oxidation pond

- ⇒ Depth – 1-1.5m
- ⇒ Detention time – 15-30 days
- ⇒ L/B ratio is normally 3, L ≯ 750m
- ⇒ Each unit should be 0.5-1 hac
- ⇒ Pathogenic bacterial removal 99.9%.
- ⇒ BOD removal – 80-90%.

Note Oxidation pond effluent is not discharged into river because it contains algae.

- ⇒ Effluent is better used for irrigation
- ⇒ Sludge accumulation 2-5 cm/year
- ⇒ Min. depth of water is kept in pond is 0.3m
- ⇒ Oxidation pond is generally used for small communities
- ⇒ organic loading rate depends on latitude of the place.

Latitude	OLR (Kg BOD$_5$/hac/day)
36 →	150
32 →	175
28 →	200
24 →	225
20 →	250
16 →	275
12 →	300
8 →	325

Q Cal dimensions of an oxidation pond & determining detention time for treating sewage for a residential colony with ~~coagulation~~ population of 8000 assume that sewage to be treated is at the rate of 125 lpcd. The 5day BOD of sewage is 325 ppm assume OLR 300 Kg/hac/day.
4ze

Sol^n Assume $\frac{L}{B} = 3$

Total sewage flow = 8000 × 125 lpcd = 1 MLD

BOD = 325 ppm = 325 kg/day

$\frac{mg}{l} \times \frac{10^{-6} kg}{d} \times \frac{l}{d}$

$$\text{Area} = \frac{BOD}{OLR} = \frac{325 \cancel{mg}\ Kg/day}{300\ Kg/ha/day}$$

$$= 1.08 \times 10^4\ m^2$$

$L \times B = 1.08 \times 10^4$

~~L × 3L~~

$B \times 3B = 1.08 \times 10^4$

$B = 60\ m$

$L = 180\ m$

Assuming Depth = 1.5m

Volume = Area × 1.5m

$= \cancel{1.08 \times 10^4 m^3}$

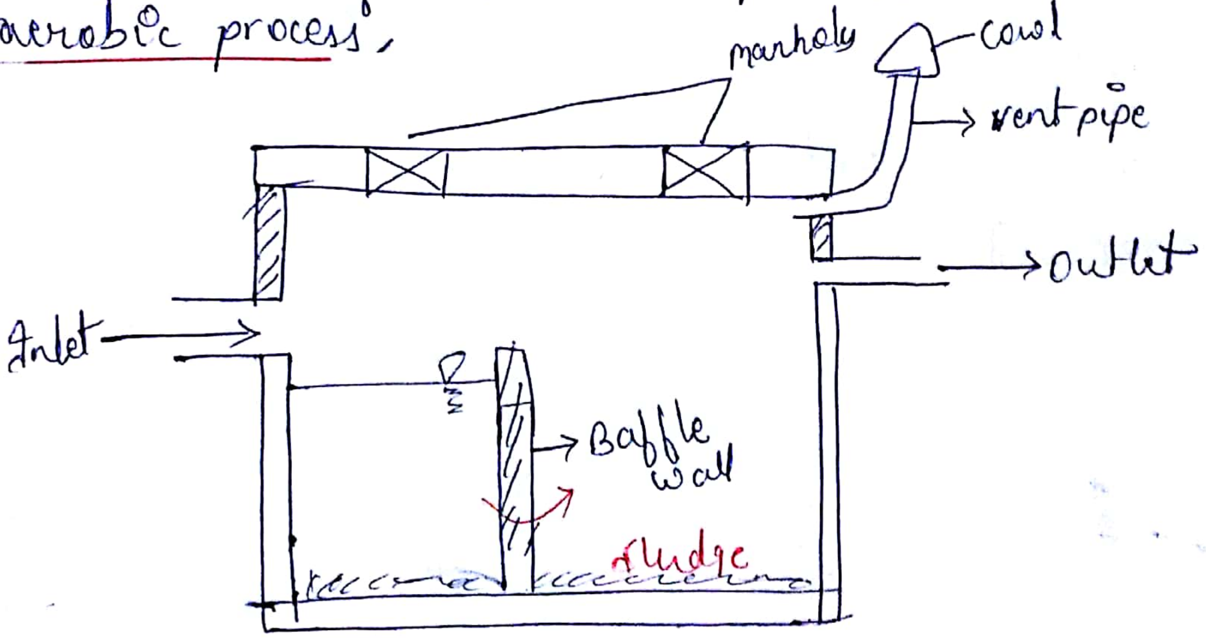
$= 10800 \times 1.5$

Detention time $= \dfrac{10800 \times 1.5 \ m^3}{10^6 \times 10^{-3} m^3/day}$ Q

$= 16.24$ days $= 17$ day

Septic tank

⇒ It is designed as ordinary settling tank except that detention time is 12-36 hrs, with extra provision of digestion of sludge with anaerobic process.



⇒ Directly raw sewage is entered in septic tank the sludge settles at bottom of tank the settled sludge is allowed to remain in tank for 6-12 months during which they are digested anaerobically.

Design parameters for septic tank

- ⇒ Flow is sewage is taken as 40-70 lpcd.
- ⇒ If sullage is allowed, flow is taken as 20-50 lpcd

  (↳ waste water from housholds, kitchen etc but not liquid waste from toilets).

- ⇒ Rate of acculumation of sludge 30 l per year (per capita)
- ⇒ Detention time → 12-36 hrs
- ⇒ L/B → 2-3
- ⇒ width should not be less than 0.75m
- ⇒ min liquid capacity = 1000 l
- ⇒ Depth – 1.2 – 1.8 m
- ⇒ Cleaning period – 6 months – 1yr
- ⇒ freeboard – 0.3m

———X——X——X———