☆ Properties of wastewater / Characterstics

Stages of treatment :-

1. Preliminary treatment
2. Primary treatment
3. Secondary treatment (Biological treatment)
4. Complete / final treatment

Preliminary treatment →

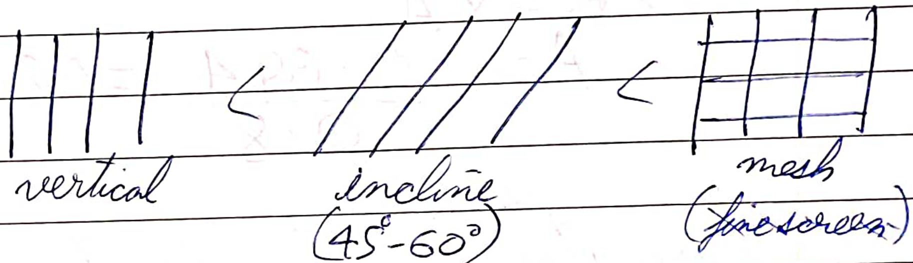
1. Large suspended ~~so~~ body waste are removed via screening

Types of screen

→ Coarse

→ Medium

→ Fine



☆ Coarse screen - Spacing b/w bar is about 50 mm or more and it collects about 5-6 L of solid per million L of sewage
Waste is burned or dumped. $V_{flow\,(horiz)} = 0.8-1$ m/sec

Medium Screen - Spacing b/w bar is 6-40 mm It collect 30-90 L of solid material per million L of sewage. $V_{flow} = 0.8-1$ m/sec

Fine screen - Spacing b/w bar is 1.5 mm - 3 mm and fine screen is remove about 20% of TSS from sewage.
Sewage gets chocked frequently so backflow mechanism is used for clearing.

Q. Calculate the screen requirement for treatment plant for treating peak flow of 60 Mill L per day of sewage. Lits

Ans) $Q = 60\,ML/d = 60 \times 10^6\, L/d$ | $1\,l = 10^{-3}\,m^3$

$\hookrightarrow 24 \times 60 \times 60$

$$Q = \frac{\cancel{60} \times 10^6 \times 10^{-3}}{24 \times 60 \times \cancel{60}} = \frac{1000}{24 \times 60} = 0.694\, m^3/sec$$

Lets assume V flow through screen $= 0.8\, m/sec$

$\Rightarrow Q = VA$

$$A = \frac{0.694}{0.8} = 0.8675\, m^2$$

Using the rectangular steel bar in the screen having 1 cm width and placed 5 cm spacing, so the gross area of steel required $= \frac{0.8675 \times (5+1)}{5 \text{ (clear spacing)}}$ (spacing + width)

Gross Area $= 1.041\, m^2$

Assuming the screen bar are placed at 60° from horizontal, So now Gross area of screen needed

$$= \frac{1.04 \times 2}{\sqrt{3}} = 1.2\, m^2$$

Commutator

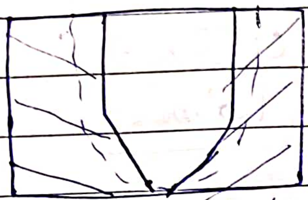
Grit chamber is used to remove oil and grease
It has always more than one chamber.

$\boxed{\text{Clariflocculator}}$ → wet (chlorinated carbon)
→ dry (alum)

Velocity (Horizontal) $V_H = 3 \text{ to } 4.5\sqrt{gd(G-1)}$ (stokes law size is > 1mm)

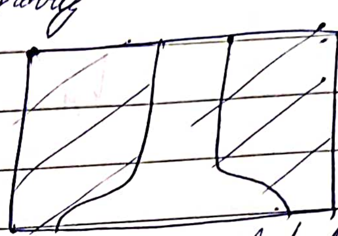
d → diameter of particle
g → gravity
$(G-1)$ → S_s
$G = S.G$

parabolic or weir shape is used.



not exactly parabolic

parabolic shaped



weir shaped

Retention time = 40–60 seconds (Grit chamber)
for water depth of 1–1.8 meter.

Q. A grit chamber is designed to remove particle w/ a diameter of 0.2 mm, SG = 2.65, $V_{settling} = 0.016 - 0.022$ m/sec
A flow through vel is 0.3 m/sec will be maintained over proportioning weir. Determine chamber/channel dimension for max W. Water flow of 10000 m³/day

Ans) $d = 0.2$ mm, $S_G = 2.65$, $V_{flow} = 0.3$ m/s

$$Q = \frac{10000}{24 \times 60 \times 60} = 0.1157 \text{ m}^3/\text{sec}$$

$$Q = V_H A \Rightarrow A = \frac{Q}{V} = \frac{0.1157}{0.3} = 0.385 \text{ m}^2$$

Assuming depth (width) of channel = 1m

$\frac{0.385}{1} = 0.\cancel{4}385$ is the length ($l \times b = A$) → width actually, $b \to 1$, B × Depth actually

Now assume V settling is 0.02 m/sec (from the range)

Detention time = $\frac{depth}{V_S} = \frac{1}{0.02} = 50\ sec$
(b/w 40 to 60) within limit

$V_H = \frac{length}{time} \Rightarrow length = 0.3 \times 50$
$= 15\ meter$

$\boxed{\text{Shredder}}$ → 6mm or less (Communitator)

$\boxed{\text{Skimming tank}}$ → removes oil & grease (after grit chamber)

The surface area req. for skimming tank is $0.00622 \frac{q}{V_r} m^2$
q = rate of flow of sewage (in m^3/day)
V_r = Min. rising vel of greasy material to be removed. (m/min)
$V_r \approx 0.25$ m/min

$\boxed{\text{Sedimentation tank (on your own)}}$

1. Vel of flow
2. Viscosity
3. Shape & S.G of particle

→ stokes law depend on

Stokes Law - $v_s = \frac{g}{18} \frac{(G-1)d^2}{\nu}$ $d < 0.1mm$ (viscosity) — ①

$v_s = 418 (G-1) d^2 \left(\frac{3T + 70}{100}\right)$ (temp) ($d < 0.1mm$) — ②
T → °C

Calculated v_s > actual v_s always | in primary sedimentation tank

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Newton eqⁿ → when $0.1 < d < 1$ mm

$1.8\sqrt{gd(G-1)}$ — $d = (0.1 - 1$ mm$)$

~~Hedge end equation ($d > 1$mm)~~ $d > 1$mm

Hazen equation ⇒ $v_s = 418\,(G_s - 1)\,d\left[\frac{3T+70}{100}\right]$

↳ d not d^2

Primary Sedimentation tank -

$$v_s = 60.6\,(G-1)\,d\left(\frac{3T+70}{100}\right)$$ [for $0.1 < d < 1$ mm]

$\boxed{\text{For inorganic solid } G = SG = 2.65}$

So for inorganic solids →

$$\boxed{v_s = d(3T+70)}$$ IMP

$\boxed{\text{For organic solid } \boxed{G_s = 1.2}}$

For organic matter →

$$\Rightarrow \boxed{v_s = 0.12\,d\,(3T+70)}$$

What is sedimentation tank? Principle & it's types & Theory of sedimentation

Intermediant sedimentation tank is also called Quiescend tank

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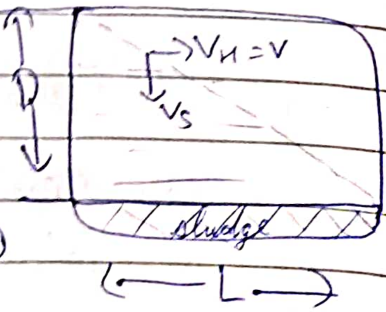
Design of primary sedimentation tank-

$$\boxed{\frac{V}{V_s} = \frac{L}{H}}$$ By Mathematical prop. - ①

$H = D$



$$\boxed{V = V_s \frac{L}{H}} \rightarrow ② \Rightarrow \boxed{V_s = \frac{VH}{L}} - ②$$

$$V = \frac{L}{t} = \frac{Q}{A} = \frac{Q}{BH} - ③$$

Now, put V from eq^n ② & ③
we get,

$$\boxed{V_s = \frac{Q}{BL}} - ④ \Rightarrow$$ this is overflow rate or surface loading or overflow velocity

$\rightarrow L/m^2/Day$

For plain primary tank (initial) the value of overflow rate = 40,000 - 50,000 L/m^2 per day

For secondary sedimentation tank ~~or~~ w/ coagulation then overflow rate = 50,000 - 60,000 L/m^2 per day

The depth of sedimentation tank varies b/w 2.4 to 3.6 meters. (excluding depth of sludge)
Try to take depth less than 3 (ie 2.4-3 is alright)

If shape of tank is rectangular, Vol = L×B×H

$$\text{Detention time} = \frac{\text{Volume}}{\text{Discharge}} = \frac{BHL}{Q}$$

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...... circular, $D.T = \frac{d^2(0.011d + 0.785H)}{Q}$

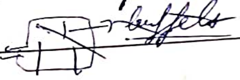
Range of D.T for plain S.T (sedim' tank) = 1–2 hours

.... secondary " = 1–2 hour

The width of S.T should be 6 meter & not greater than 7.5 meter

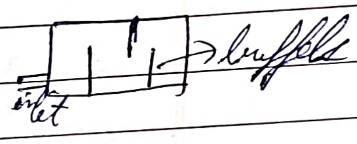
The length of ST .. not be allowed more than 4–5 times the width of the tank.

The horizontal vel for S.T is about 0.3 m per minute



Numerical →

Requirement for sedimentation tank →



1. Proper inlet & outlet
2. Buffels
3. Proper cleaning & sludge removal

Numerical

1. Design a suitable rectangular sedimentation tank for treating sewage from a city provide with a public water supply system w/ man water demand of 12M liters per day (daily demand). Assume the suitable value of detention period & velocity of flow in the tank. Assume suitable data when required. (WW (waste water) = 80% of total water discharge)

Ans) $Q = 0.8 \times 12 \times 10^6\ l/day \Rightarrow \frac{0.8 \times 12 \times 10^6 \times 10^{-3}\ m^3}{24 \times 60}$

$= 400\ m^3/hour$

Lets assume $D_{Period} = 2$ hours

$$\text{Volume} = \underset{}{\overset{Q_s}{400}} \times \overset{D_P}{2} = 800\,m^3$$

Assume → $v_{flow} = 0.3\,m/\text{minute} = \frac{L}{t}$ ($t \to D_p$)

$L = 0.3 \times 2 \times 60$

$L = 36$ meter

$\text{Volume} = L \times B \times H$

$\Rightarrow H \times B = \frac{800}{36} = 22.2\ \text{meter}^2$

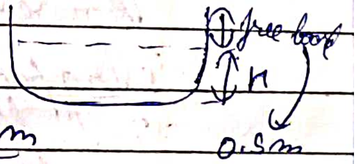
Lets assume depth of tank = 3 meter

$\Rightarrow$ Width $= 7.4\,m$

Lets assume free board = 0.5 meter

Total height = 3+0.5 = 3.5 meter

$\Rightarrow$ Dimension of tank = $36m \times 7.4m \times 3.5\,m$



Or

After $L = 36\,m$

Lets assume $Q = 40000\ l/m^2/\text{per day} = \frac{Q}{BL}$

$$\Rightarrow 40000 = \frac{0.8 \times 12 \times 10^6}{B \times 36}$$

$$B = \frac{\cancel{40000} \times \cancel{36}}{\cancel{12 \times 10^6}} = 6.66\ \text{meter} \approx 6.7\ \text{meter}$$

$$\to \frac{0.8 \times 12 \times 10^6}{40000 \times 36}$$

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$$H \times B = \frac{800}{36} = 22.2 m^2$$

$$H = \frac{22.2}{6.7} = 3.31 \text{ meter} \approx 3.3 \text{ meter}$$

free board = 0.5 m

$\Rightarrow$ Dimension $\Rightarrow$ 36m × 6.7m × 3.8m

Assignment – Grit chamber → Everything about it
1 numerical also | PDF

→ Why we dont use coagulant?
Ans) Nowadays more advance method of sewage treatment based on biological action are available & they are preffered to coagulation

Disadvantages of coagulation–

1. Chemical use in coagulation react w/ sewage & during these rxn they destroy certain micro-organisms which are helpful in digestion of sludge so they create difficulty in sludge digestion

2. The cost of chemical increases cost of treatment

3. The process of coagulation requires strict skills & supervision of supervisor

4. During coagulation, sludge produce is more

Good Write

5. ~~If~~ During process, it produces foul smell and gases.

Certain industries uses clogulation still.

Secondary Treatment process -

~~The primary treatment remove about 60-80% of unstable organic matter.~~

The effluent from the primary sedimentation tank has about 60-80% of unstable organic matter orignally present in sewage

The secondary treatment is divided into 2 main groups:

(i) Filtration
(ii) Activated Sludge process.

V.M Note - All secondary treatment process are designed to work in aerobic condition (aerobic bacterial decomposition). This is because of the fact that aerobic decomposition does'nt produce bad smell & gases as compared to anaerobic decomposition. (Eg Trickling filter)

Trickling filter →

It's sec trt. & aerobic

They are of 2 type

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(i) Conventional T. filter (low rate)(ordinary filter)(standard)
(ii) High rate filter

Advantages of trickling filter →

1. The rate of filter loading is high as such it required lesser land area & small quantities of filter media for their installation.

2. Effluent coming from trickling filter is sufficiently neutralised & stabilized. They can remove 75% of BOD & 80% of suspended solids & the effluent can be easily disposed in smaller quantity of diluted water

3. The working of filter is simple & does not req any skill or supervision

4. They are self cleaning.

5. Mechanical wear & tear is small because they contain less mechanical equipment.

warm atmosphere ∝ better filtration efficiency

Disadvantage →

1. Cost is high
2. The head loss is also high in case of trickling filter

Q: The sewage is flowing at the rate of 4.5 ML per day from a primary clarifier to a standard rate (low rate) trickling filter. 5 Day BOD of influent is 160 mg/L. ☆ The value of the organic loading is to be 160 g/m^3/day and the surface loading is 2000 L/m^2/day. Determine vol. of filter & it's depth. Also calculate efficiency of filter.

Design of ~~low rate~~ filter →

Low rate - Surface ~~Organic~~ loading → 22-44 ML/Hectare/day (28 usually)
High rate - ~~Organic~~ Surface ~~Surface~~ loading → 110 - 330 ML/Hectare/day (surface/hydraulic loading)

~~Surface loading~~
Low rate - Organic loading → 900-2200 Kg of BOD_5/Hectare meter/day

High rate Organic loading → 6000 - 18000 Kg$_{BOD_5}$/Hectare meter/day

Low rate → Efficiency (η) $= \dfrac{100}{(1+0.0044\sqrt{u})}$, u = Organic loading in Kg/Hec/m/day

☆ Ans) 5 day BOD of sewage = 160 × Q (disch) = 720 kg/day (convert accordingly mg to kg = 10^6, ML to L = 10^6 & they cancel out)

Volume of filter media $= \dfrac{\text{Total BOD}}{\text{Organic loading}} = \dfrac{720}{160/1000}\ m^3$

$= 4500\ m^3$

Surface area of filter $= \dfrac{\text{Total flow}}{\text{Surface loading}} = \dfrac{4.5 \times 10^6}{2000}$

$= 2250\ m^2$

99.82

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~~Diameter~~ Depth $= \frac{Vol}{Area} = 2$ meter

$1\,m^2 = 10^{-4}$ hectare

$\eta = \frac{1}{1+0.0044\sqrt{u}}$ → $\frac{160}{1000} = 0.16$

$\eta = 85\%$ (85.03)

High Rate filter → $\frac{R\ (\text{recirculated}\ldots\text{ of sewage})}{I\ (\text{inflow})}$ = Recirculation ~~flow~~ ratio

F = Recirculation factor $= \frac{1+\frac{R}{I}}{\left[1+0.1\left(\frac{R}{I}\right)\right]^2}$

First stage trickling filter (high rate)
Two ~~Second~~ " " " (2 filters) (high rate)

Single stage → $\eta = \frac{100}{\left(1+0.0044\sqrt{\frac{Y}{VF}}\right)}$

Y = Total organic loading in kg/day
V = Vol in hectare meter
F = Recirculation factor

~~Second~~ Two stage → $\eta' = \frac{100}{1+\frac{0.0044}{1-\eta}\sqrt{\frac{Y'}{V'F'}}}$

η = Efficiency of ~~se~~ first stage
Y' = Total BOD in effluent from 1st stage in kg/day
V' = Vol of second stage filter in hec meter
F' = Recirculation factor for second stage

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area of circle $= \frac{\pi}{4} d^2$

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Types of high rate trickling filter →

(i) Bio-filter
(ii) Accelo filter
(iii) Aero-filter

(i) thickness (depth) = 1.2 m − 1.5 m
O.L = 9000 − 11000 kg BOD_5/hec m/day
S.L = 110 − 330 ML/hec/day

(ii) thickness (d) = 1.8 m − 2.4 m

(iii) thickness (d) ≤ 1.8 m
OL = 11000 to 12000 kg BOD_5/hec m/day
SL = > 150 ML/hec/day

Q. Determine the size of high rate trickling filter for the following data sewage flow → 4.5 ML/day. $\frac{R}{I} = 1.5$, BOD_5 = 250 mg/L, BOD removal in primary tank = 30%, so (sec will have 70% of 250)
Final effluent BOD = 30 mg/L

Ans) ~~For first filter~~ − $\frac{R}{I} = 1.5$

$$F = \frac{1 + R/I}{\left(1 + (0.1)(R/I)\right)^2} = \frac{2.5}{(1 + 0.1 \times 1.5)^2} = \cancel{2.04}\ 1.89$$

$$\text{So } \eta = \frac{100}{1 + 0.0044\sqrt{\frac{Y}{VF}}}$$

Total BOD present → $4.5 \times 10^6 \times 250 \times 10^{-6}$
$= 1125$ kg

BOD removed in primary tank $= 30\%$

BOD ~~waste~~ left in sewage entering $= 0.7 \times 1125$
$= 787.5$ kg

Desired BOD conc^n $= 30$ mg/l

Total BOD left in effluent per day $= 4.5 \times 30$ kg $= 135$ kg

⇒ BOD removed by filter $= 787.5 - 135$
$= 652.5$ kg

$$\eta = \frac{\text{BOD removed}}{\text{total}} \times 100 = \frac{652.5}{787.5} \times 100 = 82.85\%$$

We know, $$\eta = \frac{100}{1 + 0.0044\sqrt{\frac{Y}{VF}}}$$

$$\Rightarrow 82.85 = \frac{100}{1 + 0.0044\sqrt{\frac{787.5}{V \times 1.89}}}$$

$V = 0.188$ hectare m $= \underline{1880\ m^3}$

Assume depth $= 1.5$ m

Surface area req $= \frac{1880}{1.5} = 1253\ m^2$

$\frac{\pi}{4} d^2 = 1253$

$\underline{d = 40 \text{ meter}}$

aerobic activated

facultative bacteria

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Activated sludge process → (ASP)

Flow diagram of secondary (biological) treatment with activated sludge process

raw sewage → Screening → Grit Chamber → Skimming tank

→ Primary Sedimentation tank → Aeration tank

4-8 hour

Activated sludge

→ Secondary sedimentation tank → recirculated (to aeration tank)

→ Sludge collection tank

→ effluent from secondary treatment

BOD removal of ASP = 80-95%, BOD is removed

Bacteria is also removed by 90-95%

20%-30% of effluent coming is put in as sludge

What is an Aeration tank → Theory

- diffused ~~air~~ air aeration
- Mechanical aeration
- Combination of both

Moisture content of sludge sometime is 98% : Activated sludge recirculated by thickening it by removing MC.

It can be thickened by following unit →

Sludge thickner unit

1. Gravity thicker
2. Flotation thickner
3. Centrifugal Thickner

Theoretical

Good Write

influent vs effluent

30 M WR
S.S book

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Important terms for Activated sludge –

1. Aeration period – Hydraulic retention time (HRT)
(Detention period) – t (small t) = $\frac{\text{Vol of tank } (V)\ m^3}{\text{Rate of sewage flow } (Q)\ m^3/day}$

$$\boxed{\therefore t = \frac{V}{Q} \times 24 \text{ hour}}$$

Note – Q = Quantity of waste flow in AT (aeration tank) including quantity of activated sludge (recycling)

Volumetric BOD loading → (Organic loading) (same thing)

Mass of BOD (kg/hec m/day) applied per day to the aeration tank through influent sewage in gram (g)

divided

Volume of aeration tank (m^3)

$= \frac{g}{m^3/day}$

Volumetric BOD $= \frac{Q\ Y_0}{V}$

Q = Sewage flow in aeration tank (m^3/day)

Y_0 = BOD_5 in mg/l or g/m^3 of influent sewage

V = Vol of aeration tank in m^3

V.M

Assignment

All 3 questions below

Q1 What is Sludge digestion process → it has 3 forms → (i) Digested sludge
(ii) Supernatant liquor
(iii) Gases of decomposition

Assignment-1 →

Q2 Stages in the sludge digestion process.

V.M

→ Three stages - (i) Acid fermentation
(ii) Acid regression
(iii) Alkaline fermentation

Q3 Explain the factor affecting sludge digestion process and their control

V.M

1. Temprature
2. PH value
3. Seeding with digested sludge
4. Mixing & steering of raw sludge with digested sludge.

Design Consideration for sludge digestion tank (digestor)

Note →

1. The shape is generally cylindrical w/ diameter of 3m to ~~10m~~ 12 meter.

2. Depth of digestion tank is generally 6 m or deeper. (it's anaerobic ~~due~~ as it cant be in the open due to foul smell)
In some exceptional area like big plants at ≃ 2 meter

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3. In sludge digestion we assume that the process of sludge digestion is linear so the capacity of digestion tank is equal to $\left(\frac{V_1+V_2}{2}\right) \times t = V$ (vol) (m^3) —(1)

V_1 = Raw sludge added per day (m^3/day)
V_2 = Equivalent digested sludge produced per day on completion of digestion (m^3/day)
t = Digestion period. (day)

V.M — Note - When the daily digested sludge could not be removed due to factors (rainy ~~etc~~ season, winter season, etc) we have to make another storage tank ~~called~~ for seperate capacity ~~tank~~ which is provided and this capacity is V_2.

For this case → The total digestor volume is equal to $\left(\frac{V_1+V_2}{2}\right)t + V_2T$

$$\boxed{V_{total} = \left(\frac{V_1+V_2}{2}\right)t + V_2T} \quad —(2)$$

T = No. of days for which the digested sludge is stored.

Question maybe be framed as → Design for monsoon / rainy season, or if we are not removing sludge In that case eqⁿ (2) is used

Imp

V.V.M → The capacity provided per capita may range 21 - 61 liter / capita per day

Note → In general ~~of~~ digestion period = 1 month

Good Write

Imp ☆ The amount of sludge gas produced in the digestion tank range b/w 14 - 28 L/capita/day

Q. Design a digestion tank for primary sludge with the help of following data. Avg flow = 20 ML/day TSS in a raw sewage = 300 mg/L. The moisture content of digested sludge = 85%. Assume other suitable data if required.

Ans) $Q = 20 \times 10^6$ L/day

~~Total sludge TSS = 300 mg/L~~ ✗ TSS = 300 mg/L

Weight of suspended solids in 20 ML of sewage flowing per day = 6000 kg/day $(20 \times 10^6 \times 300 \times 10^{-6} = 6000 \text{ kg})$

Assume 65% solids are removed in primary settling tank

Weight of solid removed in P.S tank = 65% of 6000
= 3900 kg/day

☆ Assume the fresh sludge has moisture content = 95%

IMP 5 kg of dry solids = 100 kg of wet sludge

So 3900 kg of dry sludge = 390000/5
= 78,000 wet sludge per day

Assume SG of wet sludge = 1.02

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research (S.G. to unit wt)

So the unit weight $= 1{,}020\ kg/m^3$

The volume of raw sludge produced per day

$$(V_1) = \frac{78000\ (kg/day)}{1020\ (kg/m^3)} = 76.47\ m^3/day$$

Volume of digested sludge (V_2) at 85% M.C is given following

95% (90%)

$$V_2 = V_1 \left[\frac{100 - p_1}{100 - p_2}\right] \quad \Big| \quad \begin{array}{l} p_1 = (\text{MC of fresh sludge}) \\ p_2 = (\text{MC of d.s}) = 85 \end{array}$$

$$V_2 = 76.47 \times \left[\frac{5}{15}\right]$$

$$V_2 = 25.5\ m^3$$

Assume d.p = 1 month (30 days)

~~use 2~~ use 1

$$V = \left(\frac{V_1 + V_2}{2}\right) t \quad \cancel{t \times V_2 \times T}$$

$$V = \left(\frac{76.47 + 25.5}{2}\right) \times 30$$

Activated Sludge Process

Activated sludge process is an excellent method for treating the raw sewage. The sewage effluent coming from primary S.T is used in this process and we add at ₹20-30% of ~~ratt~~ own value of activated sludge which contain large concⁿ of highly active micro-organism.

The mixture is entered into aeration tank it's mixed in presence of oxygen almost 4 to 8 hours

raw sewage → screening → grit chamber

→ skimming tank → primary S.T

→ Aeration tank (4-8 hour) → SST → chlorination → effluent

activated sludge (20-30% of total sewage) ← sludge

extra sludge → ST (sludge collection tank) → SDT (sludge digestion tank) → sludge drying

Flow diagram for conventional activated sludge plant

BOD removal = 80-95%

Bacteria removal = 90-95%

(IMP) → Depth of AT = 3-4.5 meters → width = 4-6 meters → length = 20-200 meter

ASP requires high degree of controlled condition due to -

1. It requires ample supply of oxygen.

2. It is also require intimate & continuous mixing of sewage and Activated sludge

3. Ratio of volume of AS added to vol of sewage is kept constant

Sewage DT is about 2 hour (1-4) in PST

Method of aeration →

1. Diffuse air aeration (air diffusion (abbreviation))
2. Mechanical aeration

Design of Activated sludge part →

Aeration period / hydraulic retention time (HRT) (t)

$$t = \frac{\text{Volume}\,(m^3)}{\text{Discharge}\,(m^3/\text{days})}$$

Volumetric BOD loading – Mass of BOD applied per day

$$= \frac{\text{Mass of BOD applied per day through influent sewage in grams}}{\text{Volume of the Aeration tank}}$$

$$= \frac{QY_0}{V}$$

where Q = Sewage flow in AT
Y_0 = BOD_5 (mg/L) or (gm/m³)
V = m^3 or L

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(iii) F/M ratio, F = Food
M = Microorganism

$$F/M = \frac{\text{Daily BOD load applied to Aeration system}}{\text{Total Microbial mass}}$$

(no unit usually)

$$F/M = \frac{Q Y_0}{V X_t}$$

X_t = Mix liquor suspended solids (MLSS)

Note - F/M ratio for an ASP is the main factor controlling BOD removal

Lower F/M value, more will be the BOD removal

The F/M ratio can be varied by varying the MLSS ~~ua~~ concⁿ in the tank.

Sludge age - Denoted by θ_c : mass of suspended solids (MLSS) / Mass of solids leaving the system per ~~of~~ day.

It's also called solid retention time (SRT) or it's also called mean cell ~~retention~~ residence time (MCRT) or sludge age only.

Good Write

Mass of solid in reactor (M) $= V \times MLSS$
or (aeration tank) $= V \times X_t$

Mass of solid removed w/ the wasted sludge per day $= Q_w X_R$ — ①

Mass of solid removed w/ the effluent per day $= (Q - Q_w) X_E$ — ②

So total solid removed from system per day = ① + ②.

$$\theta_c = \frac{V \cdot X_t}{Q_w X_R + (Q - Q_w) X_E}$$

X_t = Concⁿ of solid in reactor (MLSS) → mg/L
V = Vol of aerator tank
Q_w = Vol of wasted sludge / day
X_R = Concⁿ of solids in written sludge or wasted sludge (mg/L)
Q = Sewage inflow / day
X_E = Concⁿ of solid in the effluent (mg/L)

More info on survey group

Design parameter for $\boxed{\text{conventional}}$ ASP. in
If not given in ques

Parameter	Design Value
1. MLSS	1500 - 3000 mg/L
2. F/M	0.4 - 0.2
3. Aeration period (HRT)	4-8 hours
4. Volumetric loading as gm of BOD applied / m³ of tank. (g/m³)	300 - 700 g/m³
5. Sludge age	5 - 15 days
6. Q_R/Q	0.25 - 0.5
7. BOD removal efficiency	85 - 95 %.

8.	Kg of O_2 required / kg BOD removed	0.8 - 1.1
9.	Air required per kg of BOD_5	$40 - 100\ m^3$

(Activated sludge treat' plant)

Q: The data for conventional ~~area~~ ASTP is given below →

WW flow = $35,000\ m^3/day$
Vol of AT = $10,900\ m^3$
Influent BOD = 250 mg/L
Effluent BOD = 20 mg/L
Mix liquor SS = 2500 mg/L
Effluent SS = 30 mg/L
Waste sludge SS = 9700 mg/L
Quantity of waste sludge = $220\ m^3/day$

Calculate -

(i) Aeration period (HRT)
(ii) F/M ratio
(iii) % efficiency of BOD removed.
(IV) Sludge age. $\boxed{\text{(days only)}}$

scum - oil + greese

DATE: __/__/__
PAGE: ____

Unit - 3

Septic tank →

It's anaerobic process / decomposition - Principle. The sludge settle at bottom of septic tank & oil & greese rising at top of tank are allowed to remain in tank for several month.

Design consideration for septic tank →

Fig - In surveying group.

→ Capacity of septic tank (Vol) = 40-70L / capita/d
capita - per head - per person - h

→ Detention period for septic tank = 12-36 hours (24 usua

The septic tank shall be capable of storing sewag flow during detention period & additional volume of sludge for 6 months - 3 years depending upon the cleaning of septic tank.

→ Note - Rate of accumulation of sludge as recommended as 30L / ~~per~~ person / year. As minimum capacity for septic tank for 8-10 person ~~the minimum~~ maybe kept 2250 liters. For only washroom it's 1400 liters for 8-10 persons.

→ In case of septic tank free board = 0.3m when designing septic tank.

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The length of rectangular septic tank shall be 2-3 times it's width

width shall not be less than 90 cm. The depth of septic tank range b/w 1.2 - 1.8 meter.

Q: Design dimension of septic tank for a small colony of 150 persons provided w/ an assured water supply from municipality head ~~@~~ at rate of 120L/p/person/day. Assume any data required.
~~A Assume~~ Find (L, B, D) of septic tank

Ans) ~~If~~ Assume 80% become sewage
Total flow = 150 X 120 = 18000 L/day
80% = 14,400 L/day ⇒ sewage
Assume DT = 24 hours
Quantity of sewage produce during d.P = 14,400L

Lets assume rate of deposited sludge as 30L/capita/year and also assume period of cleaning = 1 year

∴ Vol of sludge deposited = 30 X 150 X 1
= 4,500L.

Total required capacity of tank = Capacity of sewage + Capacity of sludge = 14400 + 4500
= $18.9 m^3$

Now assume depth = 1.5m

Surface area = $1.66 m^2$

9th Nov → Test → 5 marks

subj & obj (calc)

$L = 3 \times width$

$\Rightarrow L \times \frac{L}{3} = 1.66$

$L = 6 ~~3~~ m$, $W = 2 cm$, $D = 1.5 + 0.3 = 1.8 m$

DIY ✓IMP

Septic tank – Def, Design, num, adv & disadv, principle.

~~Drawback~~ Imhoff tank → anaerobic
septic tank → anaerobic

→ 2 story tank

free board → 45 cm (clearspace also called) ☆

→ shaped like hopper

Designer → Mr. ~~Dr~~ Karl Imhoff

Construction →

→ It's 2 story tank or double chamber rectangular tank

→ Upper chamber is called the sedimentation chamber or flowing through chamber

→ Vel of flow in upper chamber is very low

→ Lower chamber is called digestion chamber.

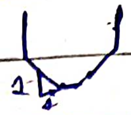
· The sludge digested due to anaerobic decomposition takes place in this chamber

→ Also called double story digestion tank (Imhoff)

→ The clearspace is called neutral zone ☆

→ Compartments)

→ Top chamber is rectangular, bottom of each digestion compartment is made in form of inverted cone or hoper with side slope 1:1.



It's to concentrate the sludge at bottom of hoper.
The digested sludge from bottom of hoper is removed periodically 1-1.5 months depending upon temp of sludge

→ D.P. of sedimentation chamber → 2-4 hours
Flow thru vel < ~~4~~ 0.3 m/min

→ Surface loading of sedimentation chamber shall not exceed 30,000 L/m^2 of plan area per day but for activated sludge plan ~~when~~ where recirculation is adopted, ~~SC~~ SC ~~the~~ may be ↑ upto 45,000 L/m^2/day

L<30m, L:W vary 3-5, depth (total) of Imhoff tank = 9-11 m & depth of sedimentation chamber is 3-3.5 m & so on.

(Digi chamber) D.C is generally designed for min capacity of 57L per capita

☆ In warmer climate, ~~this capacity~~ then sludge withdrawal is shorter ~~so~~ the capacity is reduced ~~to~~ 35-40L per capita

→ Design Imhoff tank to treat sewage from small town w/ 30,000 population. The rate of sewage is assumed 150L/c/day. Make a suitable assumption when needed.

Test → 9th Nov → Sewer (Unit - 5)

Unit
Topic
Name
Adm no
} ppt → 1st slide

Sewer topics →

When we design for water, ~~water~~, max daily flow rate = 1.8 × Avg daily
When we talk about sewer design/sewage, max daily flow = 2 × Avg daily flow
(sewage) Max hourly flow = 1.5 × Max daily flow ~~for~~
Max hourly flow = 3 × Avg daily flow (from prev eq^n)

(sewage) Minimum daily flow = $\frac{2}{3}$ × Avg daily flow
wrong ... hourly flow = $\frac{1}{3}$ × Min daily flow

water line is above sewer line

self cleaning veloci

→ What is sewer
→ What is basic diff b/w design of water supply pipe & sewer pipe
→ Generally sewer pipe less than 0.4 m diameter is running half full at max discharge
(1/2)

→ If $d > 0.4\,m$, then it's running 2/3 or 3/4 times the max discharge

(Peak discharge) P Q (m^3/sec)	free board (m)
<0.3	0.3
0.3-1	0.4
1-5	0.5
5-10	0.6
10-30	0.75
30-150	0.90
>150	1

$Q = V \times A$ | A = C/S area

Chezy's formula / rule

Acc to Chezy's, $v = C\sqrt{rS}$ (velocity)

were v = vel of flow in channel in m/sec

r = hydraulic mean radius of channel or hydraulic mean depth

$r = \frac{a\,(\text{area})}{p\,(\text{perimeter})}$] wetted only

$$\boxed{r = \frac{d}{4}} \rightarrow \frac{\pi d^2}{4 \times \pi d}$$

S = hydraulic gradient / slope

C = Chezy's constant