INTRODUCTION

❖ SEWAGE:-

- Sewage is a ductile mixture of the various types of wastes from the residential, public & industrial places
- The characteristic and composition of sewage mainly depend on this source
- Sewage contains organic & inorganic matters which may be dissolved suspension and colloidal state.

❖ CHARACTERISTICS OF SEWAGE:-

- **The characteristics of sewage can be classified as under:-**

1. Physical Characteristics
2. Chemical Characteristics
3. Biological Characteristics

1. Physical Characteristics

a. Colour:-

Fresh domestic sewage has a soap solution colour with the time of the collapse & sewage begins to set black as the decomposition stats

b. Odour:-

Fresh domestic sewage has slightly soapy or oily odour, but the state sewage has offensive odour of hydrogen sulphate and other sulphide compounds.

c. Temperature:-

Generally the temperature of the sewage is slightly higher than that of the water supply when the sewage flows in the close conduits, then its temperature further rises, which results in the increase of bacterial activity & viscosity.

The temperature of the sewage varies with the season also. if the temperature of the sewage is below normal atmosphere temperature of sewage then it indicates the infiltration of ground or surface water. But if case of higher temperature, it indicates the infiltration of ground or surface water, but in case of higher temperature, it indicates the addition of industrial sewage.

d. Turbidity:-

The turbidity of sewage directly depend on the quantity of solid matters present in the suspension state.

2. Chemical Characteristics:-

- Sewage contains complex organic matters divided from urine, faces, etc. and inorganic chemicals. Normally fresh sewage is alkaline in nature but tends to acidity as it becomes state.

- The compounds can be divided as containing nitrogen and free from nitrogen, proteins, amines and amino acids are nitrogenous compounds

- The sand, gravel, debris etc. are the inorganic matters present in the sewage, which come from street washing, kitchen and courtyard washing.

- The salty and alkalinise as inorganic matters are also present in sewage which came from bathrooms, kitchens and industrial washing
- The salts and alkalise as inorganic matters are also present in sewage which come from bathrooms, kitchens and industrial plants

3. <u>Biological characteristics:-</u>

- Sewage contains large quantity of bacteria which come from excremental matter

- All the bacterial present in the sewage are not harmful more quantity is of harmless bacteria which helps to heat the sewage and reduce the cost of treatment plant.

- Only pathogenic bacteria which are discharged by sick persons, injected with diseases like cholera, typhoid etc. are harmful to the human health and give difficulties at the treatment plants.

AIMS AND OBJECTIVES OF SEWAGE DISPOSAL

The following are of the aims and objectives of sewage disposal:-

1. Proper disposal of human excreta to a safe place before it stats decomposition and may cause insanitary condition in the society.

2. To take out all kinds of waste water from the locality immediately after it are so that the mosquitos, flies, bacteria etc. may be not braced in it and cause nuisance

3. Final disposal of sewage on typical as in hereby water sources after some treatment. So that receives after treatment load on water may not get polluted and unsafe for its further use

   If the sewage is disposed of on load it should have a such a degree of treatment that it may not affect the subsoil in any way

❖ METHODS OF COLLECTION:-

The Sanitation of a town or city is done by two methods which are called Conservancy System.

a. Conservancy System

b. Water- Carriage System

a. CONSERVENCY SYSTEM:-

Sometimes it is also called dry system. This system is in plastic from very ancient times.

Various types of refused and storm water are collected conveyed and disposed of separately by different methods in this system. Therefore it is called Conservancy system.

b. WATER CARRIAGE SYSTEM

- In this system water is in the main substance therefore it is called water carriage system
- In this system, the excremental matters are mixed up in large quantity of water and are taken out from the city through properly designed sewerage system, where they are disposed off after necessary treatment in a satisfactory manner.
- The sewage so formed in the water carriage system consist of 99.9 % of water and remaining 0.1 % of solid matters

❖ SITE SELECTION

- The treatment plants should be located as near to the point of disposal as possible.
- If the sewage is to be disposal of finally in the river or natural streams the site should be located on the river bank are should be taken that the site is on the downstream side of the city and sufficient away from the water intake works.
- If finally the sewage is to be applied on the land, the treatment plant should be located near the land at such a place from where the treated sewage can easily flow under gravitational forces towards the discharge points.
- The site should not be much fed away from the town to reduce the length of the sewer line.
- The site should not be so close to town, that it may cause difficulties in the expansion of town and pollute the general atmosphere by odour and by nuisance.

❖ PLANNING OF SEWAGE TREATMENT PLANT

- All the plant should be located in order of sequence, so that sewage from one process should directly go into the next process

- If possible all the plants should be located at such elevation that sewage can flow from one plant into next under its force of gravity only

- All the treatment unit should be arranged in such a way that minimum area is required. It will favours economy

- Sufficient area should be supplied for further expansion in the beginning.

- By piers and overflow weirs should be provided to out of operation any unit when required.

- Off Channels conduits should be laid in such a way as to be obtaining flexibility, convenience and economy in the operation

❖ SCREENING :

Screening is the very first operation carried out at a sewage treatment plant, and consist of passing the sewage through different types of screens so as to trap and remove the floating matters, such as pieces of clothes, papers, wood, dead solids etc.

These floating materials, if not removed, will choke the pipes and will adversely affect the working of the sewage pumps. Thus the main idea of providing the screens is to protect the pumps and other equipment from the possible damage due to floating matter of sewage.

➢ DESIGN CRITERIA :

Depending upon the size of opening, screens may be classified as -

a. Course Screen opening size – 50 mm or more.
b. Medium Screen opening size – 4 mm to 40 mm
c. Fine Screen opening size – 1.5 mm to 3 mm

The screens are kept inclined at about 300 to 600 to the direction of flow, so as to increase the opening area and to reduce the flow velocity.

While designing the screen, its cleaning frequency is to be designed. The cleaning frequency is governed by head loss though the screens.

The more the screens openings are clogged more will be the head loss though the screens. Hence when the screens are completely clean, the head loss will be negligible.

❖ GRIT CHAMBERS :

➢ FUNCTION :

Grit chambers also called as grit channels are intended to remove the inorganic particles(specific gravity about 2.65) such as sand, grit etc. of size 2mm or larger to prevent damage to pumps and to prevent their accumulation in sludge digesters.

➢ DESIGN CRITERIA :

The most important point in the design of the grit chamber is that the flow of the velocity should neither be too low so as to cause the settling of the lighter organic matter nor should be fast that as not to cause the settlement of the entire silt & grit present in the sewage.

The flow velocity should be enough to scour out the settled organic matter and reintroduce it into the flow stream.

➢ CONSTRUCTION DETAILS :

1. The depth and detention time provided for a grit basin are independent and are based on the consideration of settling velocity of inorganic particles.
2. A detention time of about 40 to 60 seconds (1 minute) is generally sufficient for a water depth about 1 to 1.8 meter
3. After fixing the depth and detention time, we can design the tank dimensions as the length will be equal to the velocity x detention time.

We have designed the grit chamber provided with a Parshall Flume.

Parshall Flume – A Parshall flume also called as a venial flume, is a horizontally constructed vertical throat in an approach channel.

Such a venial flume used as a discharge measuring device also as a velocity control device.

The venial flume as a velocity control device is preferable to the proportional flow weir etc., as it involves negligible head loss.

❖ SEDIMENTATION TANK :

➢ PRINCIPLE :

The very fundamental principle under lying the process if sedimentation is that to settle the organic matter present in the sewage having specific gravity less than that of water (i.e. 1.0). In still sewage the particles will tend to settle by gravity whereas in flowing sewage they are kept in suspension because of turbulence in water. As soon as the turbulence is affected by offering storage to the sewage, these impurities tend to settle down at the bottom of the tank offering storage. The basin in which the flow of sewage is retarded is

➢ DESIGN CRITERIA :

The theory which is applied to design of such sedimentation basin assumed that the settlement is uniformly distributed as the sewage enters the basin.

If 'Q' is the discharge entering the basin with flow velocity 'V' is given by

$$V=Q/BH$$

Where :

B-width of basin and
H-depth of basin.

Q/BL i.e. the discharge per unit of plan area is a very important term for the design of continuous flow type sedimentation tanks and is known as "overflow rate" or surface loading or overflow velocity.

For primary sedimentation tank SOR is kept between 50,000 to 60,000 liters per square meter per day.

The detention time is used for design of primary sedimentation tank. The detention time for a sewage sedimentation tank usually ranges between 1 to 2 Hrs.

➢ CONSTRUCTION DETAILS:

INLET AND OUTLET ARRANGEMENT – This sort of arrangement is required in order to reduce short circuiting and to distribute the flow uniformly.

BAFFLES – Baffles are required to prevent the movement of organic matter & to escape along with the effluent

CLEANING AND SLUDGE REMOVAL – The suspended organic solids contained in the sewage settle down at the bottom of the sedimentation tank and have to be removed periodically before it becomes state of septic.

❖ TRICKLING FILTERS :

The conventional trickling filters and their improved forms known as high rate trickling filters are now almost universally adopted for giving secondary treatment to the sewage.

➢ CONSTRUCTION DETAILS :

1. Trickling filters tanks are generally constructed above the ground.

2. They may either be rectangular or circular.

We have provided circular filter tanks with rotary distributors having a number of distributing arms.

Rotary Distributors – These distributors rotate around a central support either by an electric motor or move generally by the force of reaction on the sprays.

The advantage of having two or more arms is not only to get reaction sufficient to rotate the entire mechanism but is also to pass the fluctuating demands by taking low flows in two arms and the remaining two arms coming into operations only at the times of higher flow.

➢ DESIGN CRITERIA:

1. The design of the trickling filter primarily involves the design of the diameter of the circular filter tank and its depth.

2. The design of rotary distributors and under drainage system is involved in the filter design.

3. Filter diameter and depth – the filter diameter and depth is designed for average value of sewage flow.

4. The rotary distributors, under drainage system and the other connected pipe lines etc. are designed for peak flow and is checked for average flow.

5. Design of Orifices – Each arm section will be provided with different number of orifices depending upon the discharge to be passed though each section.

❖ AERATION TANK :

From the primary settling tank the sewage flows to the aeration tank and is mixed with the activated sludge.

The aeration tanks are normally rectangular tanks 3 to 4.5 meter deep and about 4 to 6 meter wide length and detention period between 4 to 8 Hrs. for sewage.

➢ **Methods of Aeration** –

There are two basic methods of introducing air into the aeration tanks i.e. –

1. Diffused air aeration or air diffusion
2. Mechanical aeration
3. Sometimes a combination of both called combined aeration.

❖ SECONDARY SEDIMENTATION TANK:

From the aeration tank the sewage flows to the final sedimentation tank. This tank will normally be general type.

➢ **DESIGN CRITERIA:**

Good design should provide as per their overflow rate, not exceeding 180 cubic meter per day per number of weir.

Solids Loading – is another important factor which is governs the design of the secondary basin. The solids loading rate based on the mixed flow to the settling tank may be kept at about 100 to 150 Kg/m2 per day at average flow.

Detention period – the detention period for such sedimentation tank may be kept between 1 to 2 Hrs. which is usually found to be given optimum results.

The length to depth ratio may be kept as about 5 for circular tank and for rectangular tank one from 3.5 to 4.5.

❖ SLUDGE DIGESTION TANK:-

The sludge which is settled in the P.S.T. as well as the extra activated sludge from the secondary settling tank in excess of that required for recreation is digested before drying & displacing.

Since the sludge obtained in S.D.P. contains too much moisture is therefore very heavy so we have to reduce moisture by sending it to a sludge thighs unit.

➢ DESIGN CRITERIA:

The important term which deals with the design of tank.

I. AERATION PERIOD:-

The aeration period (t) decide the loading rate at which the sewage is applied to A.T.

D.T.(t) = Volume of tank / Rate of sewage have in tank

= V in m^3 / Q in m^3/day

II. Volumetric B.O.D. loading-

A is designed as the BODS load applied per unit volume of aeration tank. This loading is known as organic loading.

III. Food(F) to Microorganism (M)ratio:-

F/M ratio is an important rational dynamic loading rate < adapted for an activated sludge process.

$$\text{F/M ratio} = \frac{\text{Daily BOD load applied to the aerated system in gm}}{\text{Total microbial mass in the system}}$$

IV. MLSS (Mixed liquor suspended solids)

The total microbial mass in the aeration system (M) is computed by multiplying the average concentrated of solids in the mixed liquor of the aeration tank

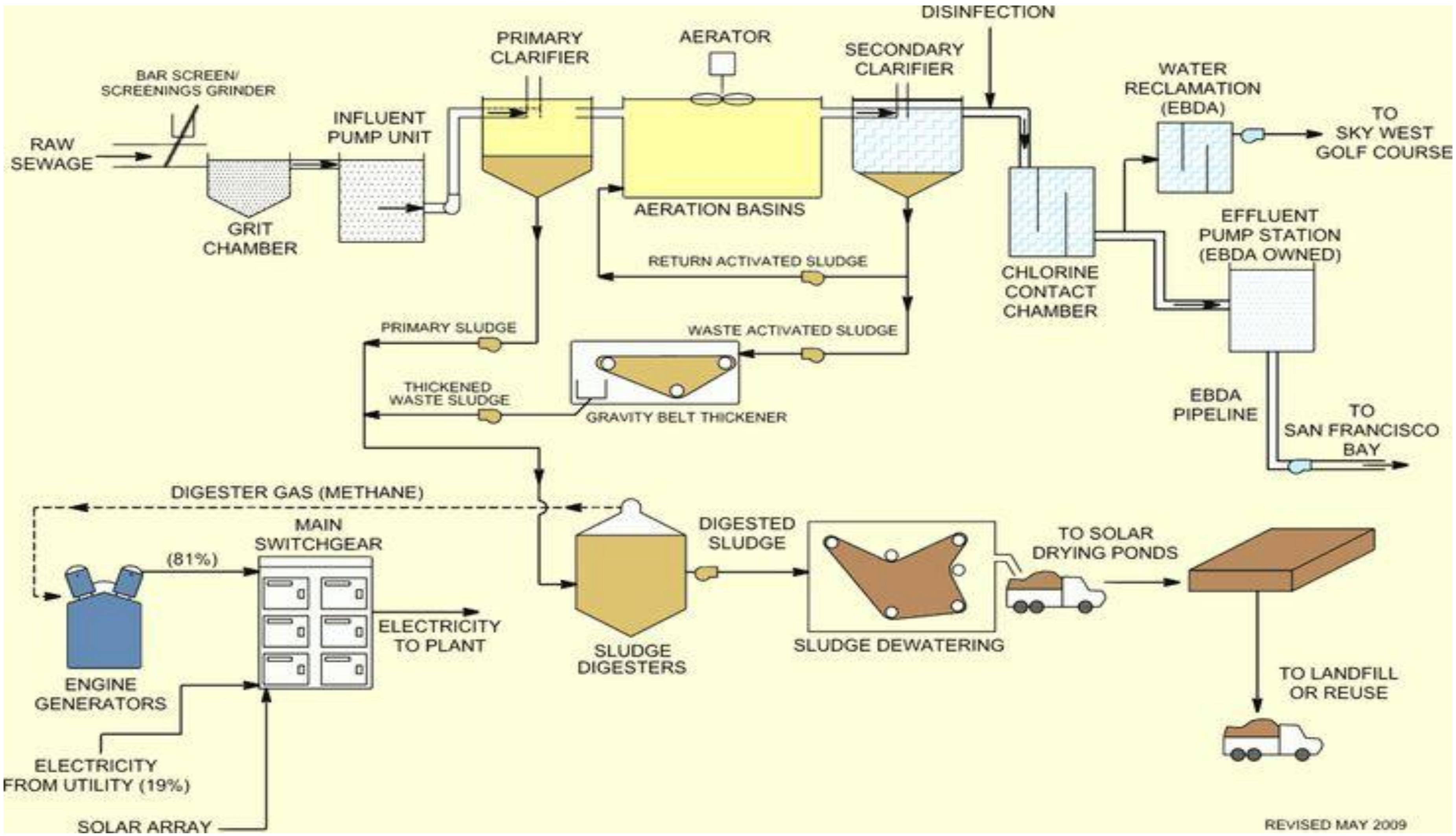
❖ SLUDGE DRYING BEDS

- **PURPOSE :**
  The digested sludge from the digestion tank contains a lot of water and is therefore first of all dewaters at dried up before further disposal either by burning or dumping.sp
- **DESIGN CRITERIA:**
  S.D.B. are open beds of lands ,45 to 60 cm. deep and consisting of about 30 to 45 cm. thick graded layer of gravel as crushed stone varying is size from 15 cm at bottom to 1.25 cm. at top & overlain by 10 to 15 cm. thick course sand layer.

  The sewage sludge from the digestion tank is brought & spread over the top of the drying beds to a depth about 20 to 30 cm. through distance through, having opening about 15 cm x 20 cm at a distance of 2 m

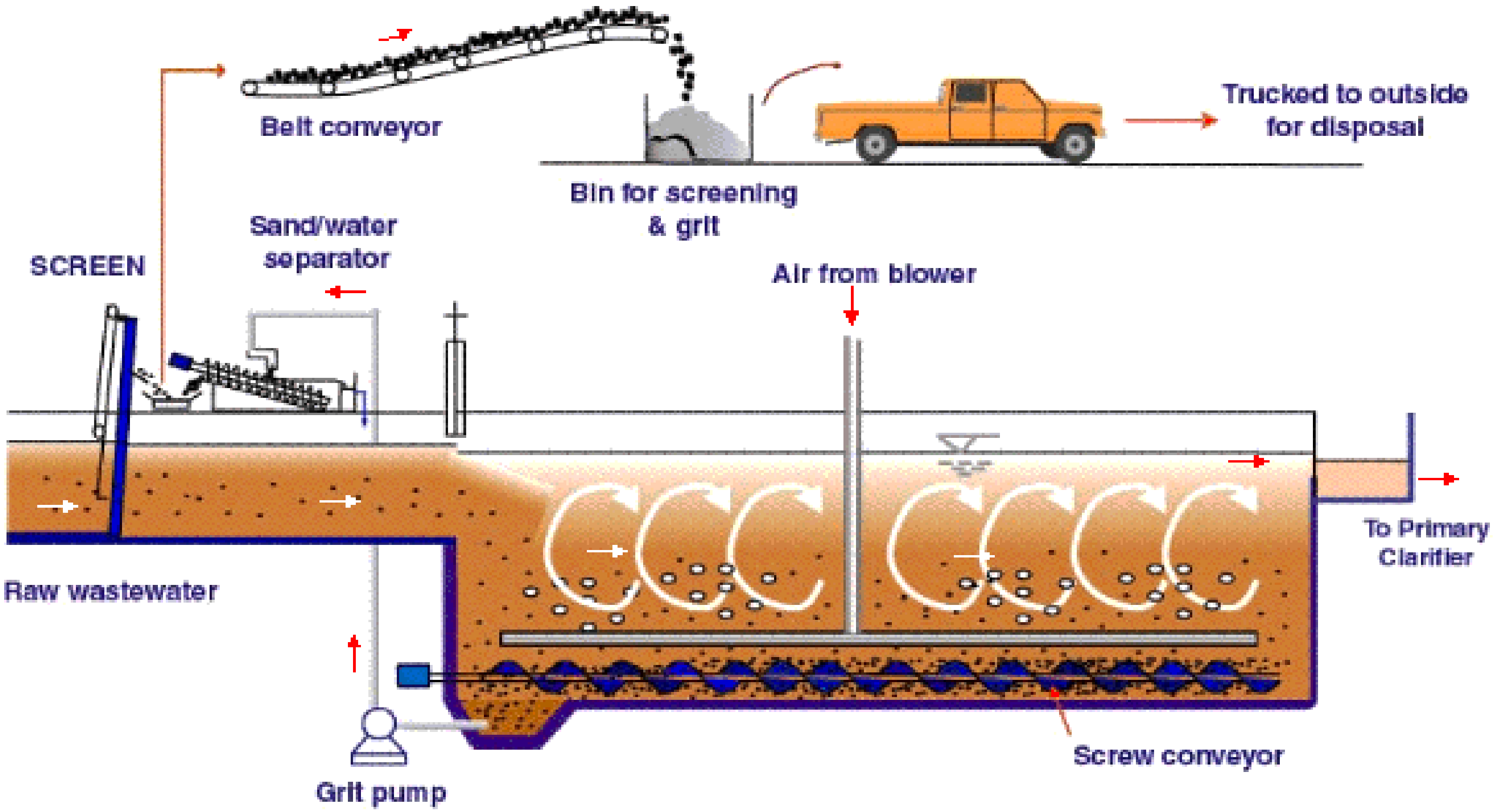
- **Required Area:-**
  The required area for SDB normally ranges between 0.05 to 0.2 sq.mt. per capita.

  Sludge is removed from beds after period of about 7-10 days as within this period about 30 % of the moisture goes away the surface of sludge gets cracked.

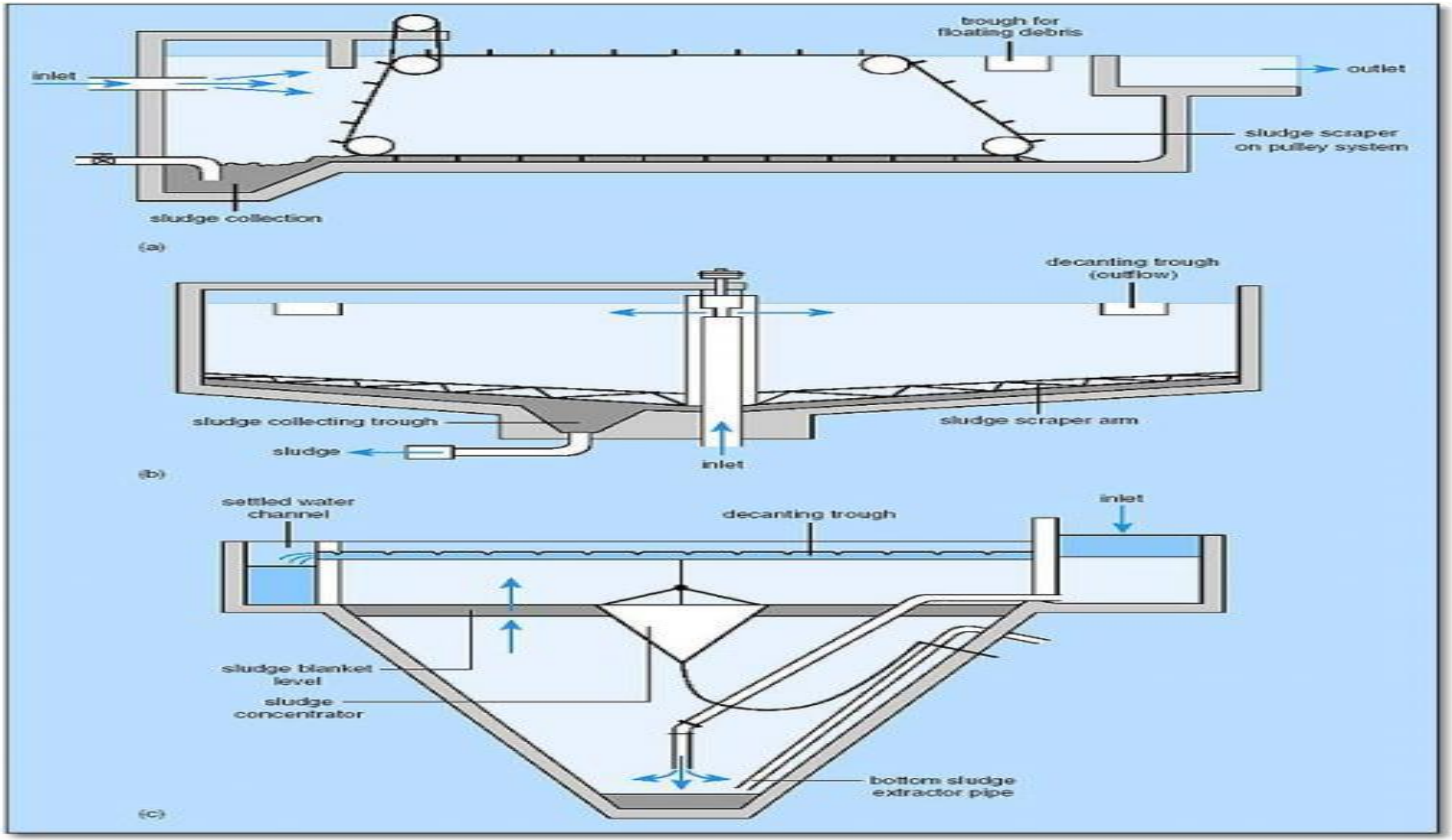


SEWAGE TREATMENT PROCESS

SCREENING OF SEWAGE TREATMENT PLANT

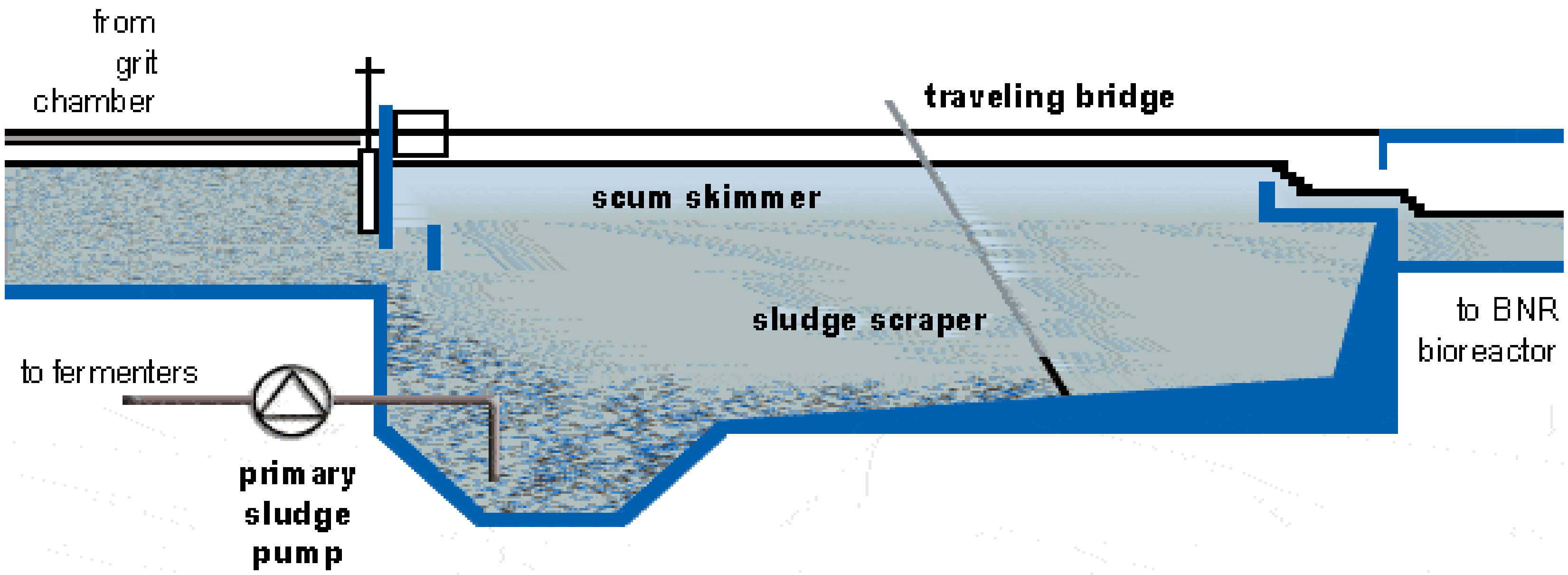


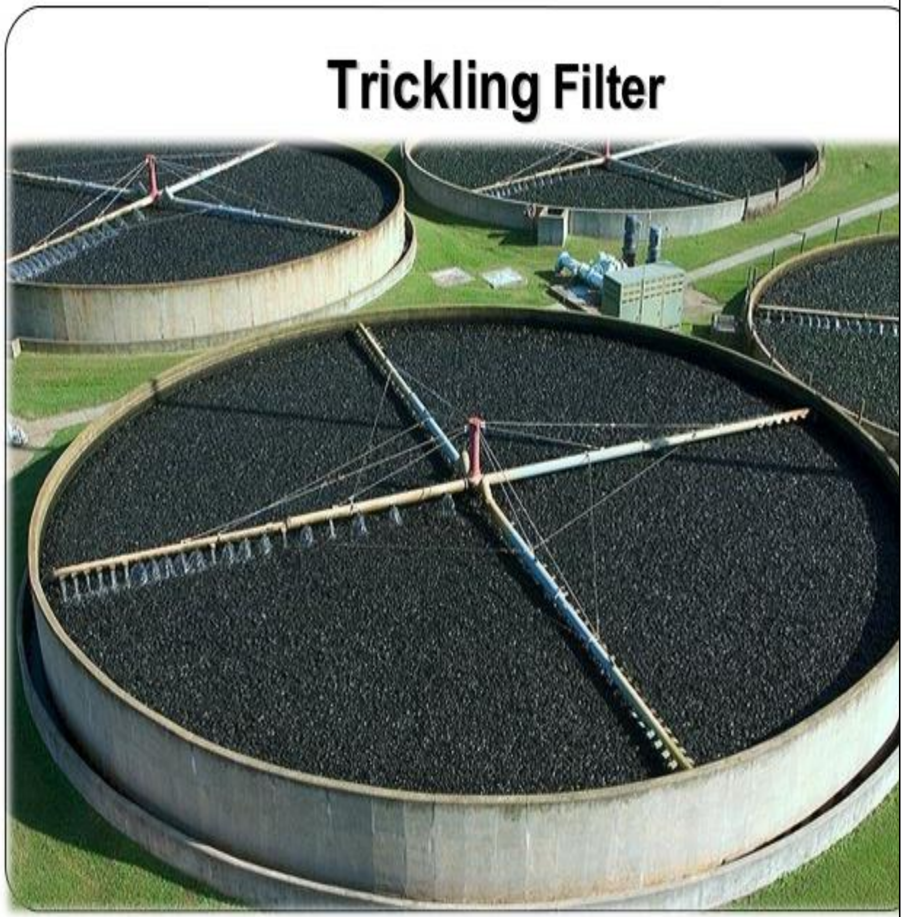
SCHEMATIC OF HEADWORKS (Screen & Aerated Grit Chamber)



SEDIMENTENTION TANK

Primary Sedimentation Tank





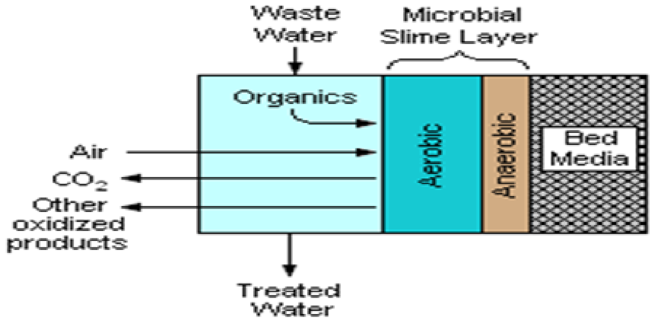
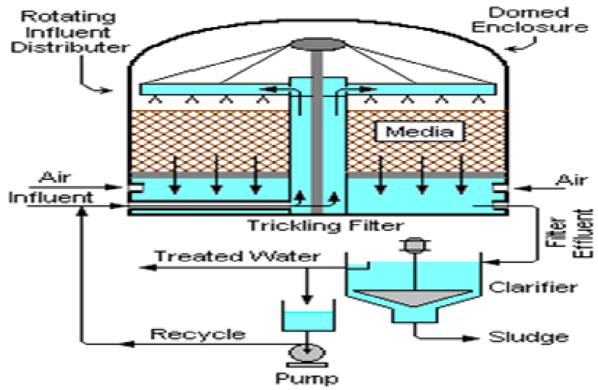
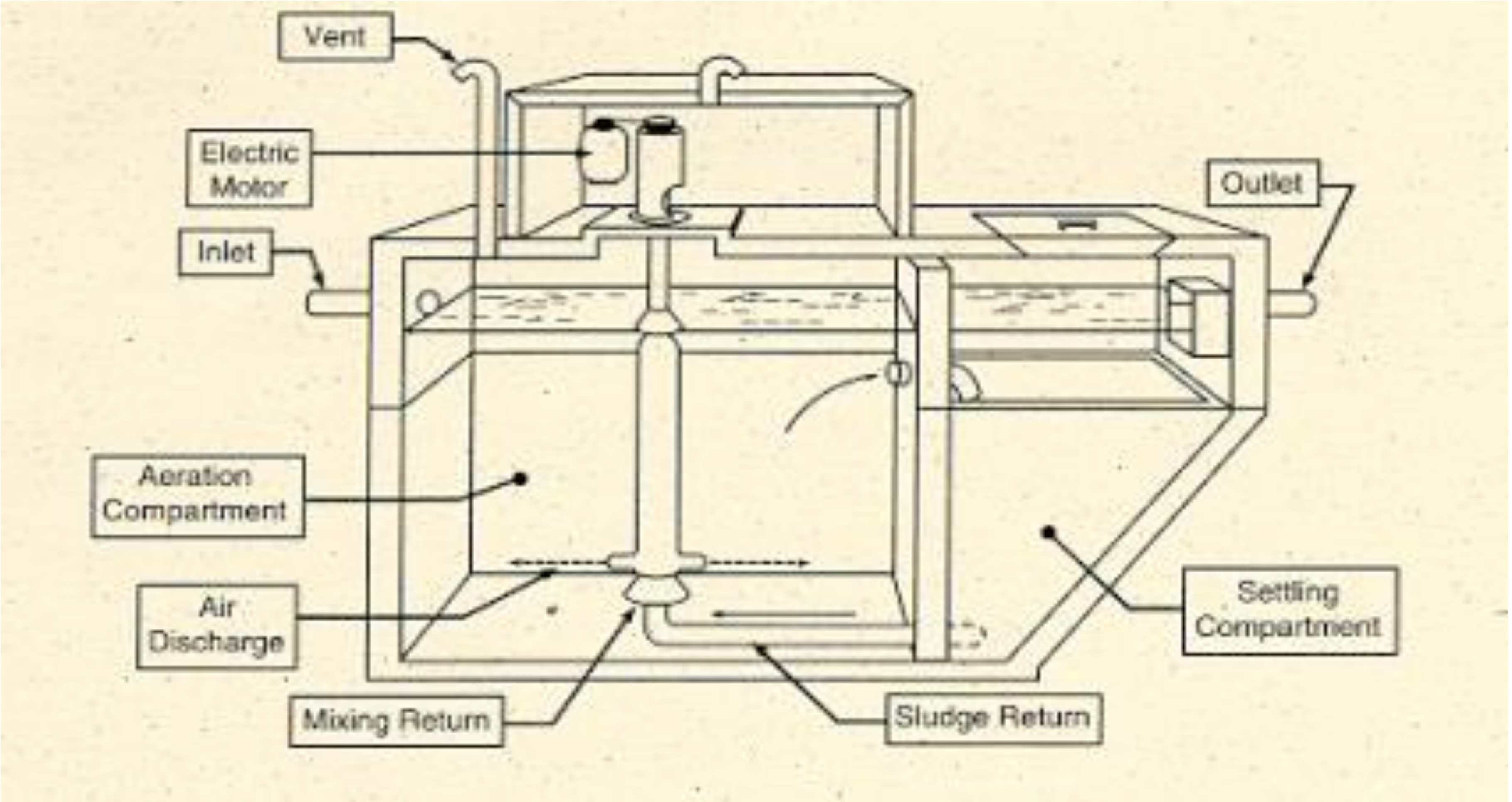


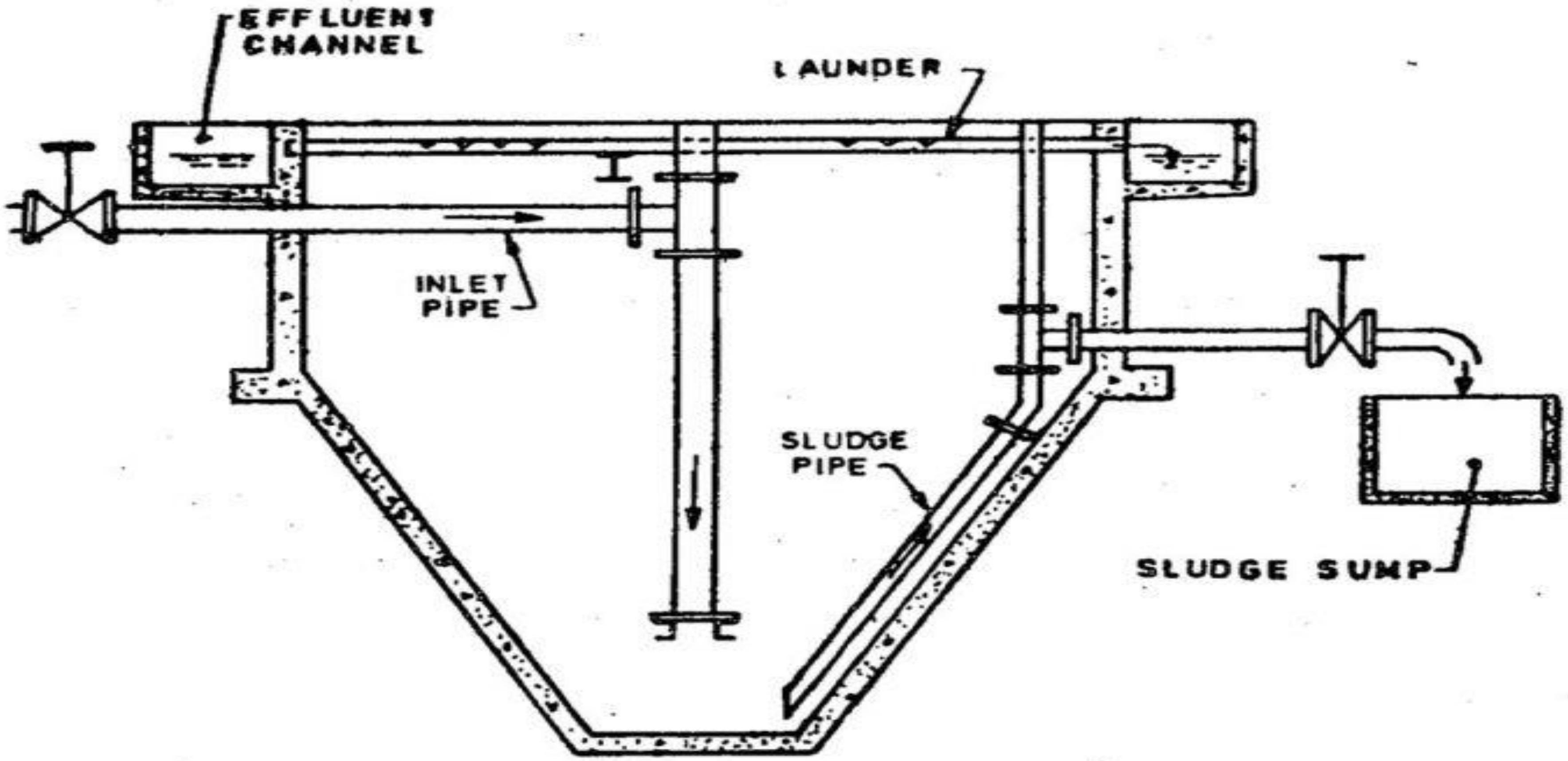
Image 1: A schematic cross-section of the contact face of the bed media in a trickling filter[5]



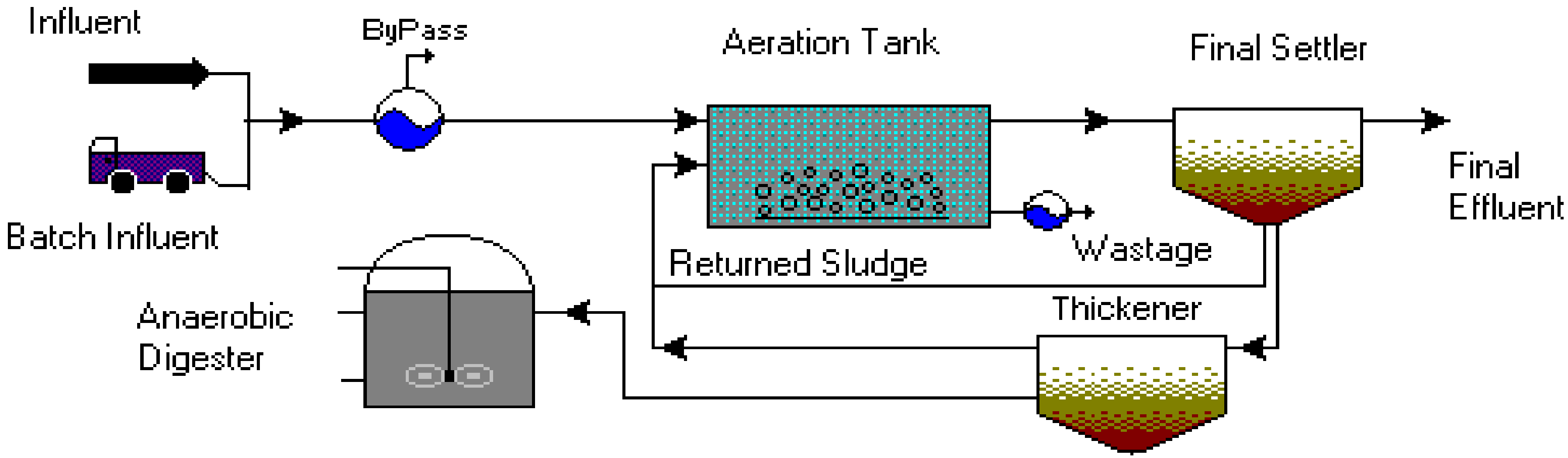
TRICKLLING FILTER



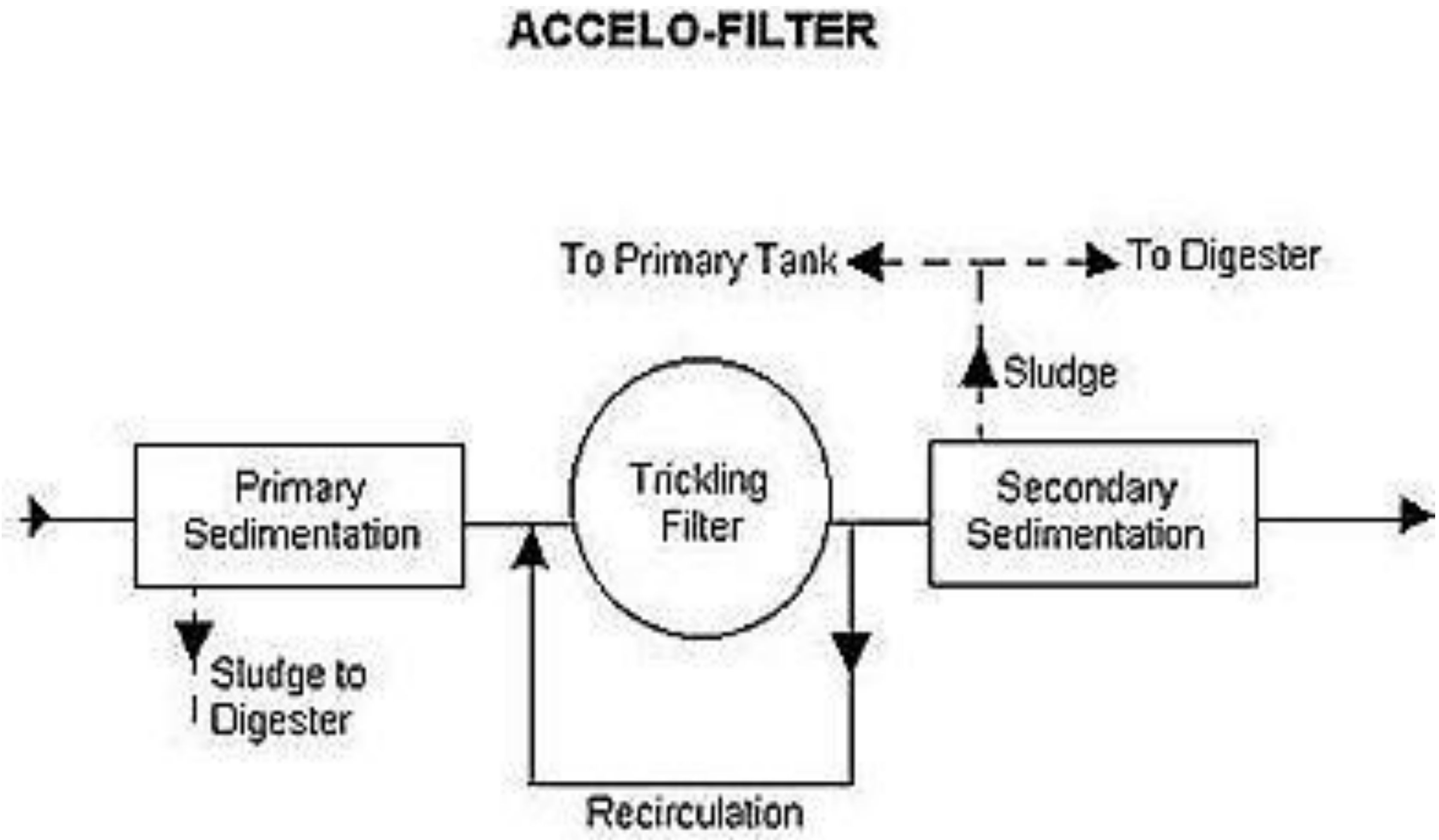
AERATION TANKS



PRIMARY SETTELMENT TANK



Simple Activated Sludge Plant With Digester (4 Cells)



ARM TRICKLING FILTER

GUJARAT TECHNOLOGICAL UNIVERSITY

6th Semester Civil Engineering – PDDC

Subject Code & Name : X60602 - Civil Engineering Project-I

Design a waste water treatment plant for a town

a) A town having following population

Year	Population
1976	1,25,000
1986	2,10,000
1996	3,25,000
2006	4,30,000

b. Design period : 20 Years

c. Peak Factor : 1.5

d. Wastewater Characteristic (BOD520) : 270 mg/l

e. Total Suspended Solids in raw wastewater : 600 mg/l

f. Source of Disposal is River, which having High Flood Level is 105 M & Bed Level is 95 M at the site of disposal

g. Average Ground Level in the town 106.5 M

h. The rate of water supply 135 l/c/d at present

i. The source of water is River

j. The town has already built up Water Treatment Plant considering of design period 30 years

Design the treatment plant in two stage of 10 years. Workout the land requirement for plant and draw the sectional elevation and layout plant. Also prepare the design drawing of all units with dimensions. Calculate the BOD520 and SS of the final effluent and check whether it is complying the GPCB norms laid down for effluent disposal in river. In case it doesn't satisfy, modify the design properly.

Design

- **POPULATION FORECAST**

Year	Population In Lacs
1976	1,25,000
1976	2,10,000
1996	3,25,000
2006	4,30,000

- **POPULATION FORCASTING TABLE:-**

Year	Population In Lacs	Increase in Population	Percentage Increase per decade	Incremental Increase
1976	1,25,000			
1976	2,10,000	85000	40.47	
1996	3,25,000	115000	35.38	30000
2006	4,30,000	105000	24.41	-10000
Total		305000	100.20	20000
Avg. Per Decade		**i = 101667**	**r = 33.40**	**r 1 = 6667**

A) Arithmatical increase method:-

$$Pn = Po + ni$$

$$P_{2026} = P_{2006} + 2\ (101667)$$

$$= 430000 + 2\ (101667)$$

$$= 633334/-$$

B) Geometrical increase method:-

$$Pn = Po\ ((1+r/100)^n$$

$$= 430000\ (1 + 33.40/100)^2$$

$$= 760627/-$$

C) Incremental increase method:-

$$Pn = Po + ni + (n(n+1)/2)\ r1$$

$$= 430000 + 2\ (101667) + 2((2+1)/2)\ (6667)$$

$$= 653335/-$$

Taking the maximum population = 760627/-

Daily water calculation:-

Water Supply = **135** lcpd

Water demand = population x lcpd

= 770627 x 135

= 104034645 Liters/day

Average Daily water supply = 104034.645 M^3/day

Average water supply = ((104034.645)/(24*60*60))

= 1.20 M^3 / sec.

Taking 80% of total water supplied as waste water (sewage) Q = 1.20 * 0.8

Qavg = 0.96 M^3 / Sec.

Peak Factor = 1.5

Qmax = Qavg x peak factor

= 0.96 x 1.5

= 1.44 M^3 / Sec

Hydraulic Characteristic of Circular Sewer

FOR 2/3 FULL

d/D = 0.67

From Interpolation,

Proportional Area:- a/A = 0.7114

Proportional wetted Perimeter = p/P = 0.6109

Proportional Discharge = q/Q = 0.7879

Proportional hydraulic mean depth:- r/R = 1.1625 proportional velocity:- v/V = 1.1056
Assume, v = 1 m/ sec

Q= A*V

A= Q/V = 0.951/1 = 1.44 M^2

$3.14*D^2/4 = 1.44$

D = 1.35 meter

R=D/4 = 1.35/4 = 0.3375 meter

r/R = 1.1625

r = 1.1625*0.3375 = 0.39 meter

a/A=0.7114

a= 0.7114*.1.44 = 1.0244 M^2

$v/V=1.1056$

$v=1.1056$ m/ sec

$p/P=0.6109$

$p=3.14*1.35*0.6109$

$p=2.59$ meter

<u>From Manning's Formula,</u>

$V=(1/N)*(R)^{2/3}*S^{1/2}$ Take N=0.013 .

$1 = (1/0.013)*(0..3375)^{2/3} *S^{1/2}$

S=0.001 Means S=1 in 1000

$q_{min} = 1/3\ q_{avg}$

$= 1/3*0.96$

$= 0.32\ M^3/Sec$

qmin/ Q = 0.32 / 1.44 = 0.29

DESIGN OF SUMP WELL

The Sump well can be designed for peak flow of 1 min.

Peak flow rate = 1.44M^3 / Sec.

Peak Flow rate for 1 minute = 1.44 *60 = 86.4 M^3

Given the depth of sump well is 5 m

∴ C/S Area of Sump Well = Capacity / Depth

= 86.4 / 5

= 17.28 M^2

Therefore , required diameter of Sump Well = $(17.28*4/3.14)^{1/2}$

D = 4.69 ≈ 4.70 meter.

Also, we have to provide the 0.5 meter free board.

Therefor size of sump well = 4700 mm dia. And 5m depth (with 0.5 m free board)

DESIGN OF PUMPS

Provide 04 nos pumps.

Therefore, Capacity of each pump = 86.4 /(60*4)

= 0.36 M^3

DESIGN OF SCREENINGS

Peak Flow = 1.44 M^3/Sec

Let us assume a desired velocity through the screen at peak flow 0.8 m/sec(V).

Now Q=A/V

1.44 =A X 0.8

A = 1.80 M^2...................net area of screen required.

Using rectangular bars in the screen, having 1 cm width and clear spacing 5 cm

Net rectangular steel bars in the screens having 1 cm width & placed at 5cm, we have, gross area of the screen required,

A = 1.80 X (c/c spacing / clear spacing)

= 1.80 x (6/5)

= 2.16 M^2

Assuming that the screen bars are placed at 60^0 to the horizontal

Gross area of screen required (in inclined plane)

A^I = A/sinø

=2.16/sin60

=2.49 M^2

Velocity of flow above screen v = 0.8 m/sec x (5/6)

= 0.67 m^2

Now, we have

$V = 0.8$ M/sec

$v = 0.67$ m/sec

Head loss through the screen,

$h_L = 0.0729 (V^2 - v^2)$

$= 0.0729 (0.8^2 - 0.67^2)$

$= 0.0139$ Say 0.014 M

$= 1.4$ cm

When the screen openings get half clogged, then

$V = 0.8 \times 2$

$= 1.6$ m/s

So, $h_L = 0.0729 (1.8^2 - 0.67^2)$

$= 0.154$M

$= 15.4$ cm

Thus, we find that when the screen is clean (in the initial stage) the head loss is only 1.4 cm, whereas the headloss shoots up to about 15.4 cm, when the screen is half clogged. The bar screen should be frequently cleaned in order to keep the head loss with in the allowable limits.

DESIGN OF GRIT CHAMBER

Let us provide the rectangular channel section since a proportional flow weir is provided for controlling velocity of flow –

Assume Horizontal velocity V_h = 0.3 m/Sec

Settling velocity is between 0.016 to 0.022 m/sec

Therefore, we assume = Vs = 0.02 m/sec

Now, Q = Velocity * C/s area

= Vh x A = 0.3 A

And provide 02 units –

=1.44/2 = 0.3 A

A = 1.44 /(2*0.3)V

A = 2.4 M^2

Assuming the depth of 1.0 meter, we have the width (B) –

1*B = 2.4

B=2.4 meter

Detention Time= Depth of basin / Settling velocity

= 1/0.02 sec

= 50 Sec

Length of tank = V_h * Detention Time

= 0.3 * 50

= 15 meter

Hence,

Use a rectangular tank with following dimensions –

Length (L) = 15 meter

Width (W) = 1.8 meter

Depth (D) = 1.0 meter

PARSHALL FLUMES:-

Flow , Q = 1.44 M^3/sec

= 1440 ml/sec

From, standard dimension Parshall flumes for Q = 750 ml/sec

W = 1500 mm

A = 2100 mm^2

B = 2060 mm

C = 2100 mm

C1 = 2625 mm

F = 600 mm

G = 900 mm

K = 75 mm

M = 225

DESIGN OF PRIMARY SEDIMENTATION TANK

Assume surface overflow rate = 40 m3/m2/day = 40000 liters/m^2/day.

Provide 02 units of primary sedimentation tank.

Assume, detention period as 2 hrs.

Capacity of tank = 1.44*2*60*60 / 2

= 5184 M^3

Assume depth = 3 meter

Therefore, area = 5184/3 = 1728 M^2

$3.14*D^2 / 4 = 1728$

$D^2= (1728 \times 4)/3.14$

D = 46.91 m

≈ 47.00 m

Dimension of Tank = 47 meter diameter and 3 meter depth.

Discharge (Q) / Area = $1.44* 10^3 * 10^2 / 1728$

= 83.33 m3/m2/day > Assumed. Hence Safe.

Influent Structure: Assume = V = 1 m/sec

Q/V = A

1.44 / 2*1 = Area (For 02 Units)

A = 0.375 M^2

$d = (0.72 * 4 / 3.14)^{1/2} = 0.96$ meter

Effluent Structure –

Maximum Perimeter = 3.14 * D = 3.14 * 47 = 147.58 meter ≈ 148 meter

Weir loading = Q / perimeter = (1.44 / 2) / 148 = $4.865 * 10^{-3}\ m^2/sec$

$= 4.865 * 10^{-3} * 60*60*24\ m^3/m/day$

$\approx = 420\ m^3/m/day$

Number of 90^0V-notches. Assuming c/c spacing at 200 mm –

Number of weirs = Weir length / spacing = 148/0.2 = 740 Nos.

Discharge through one weir = Q_w = Q / No of weir = 1.44 / (2*740) = $9.73 * 10^{-4}\ M^3$ /sec

$Q_w = (8/5) * C_d * (2g)^{1/2} * \tan 45^o * H^{5/2}$

$9.73 * 10^{-4} = (8/5) * 0.02 * (2*9.81)^{1/2} * \tan 45^o * H^{5/2}$

$H^{5/2} = 6.864 * 10^{-3}$

H = 0.136 meter

DESIGN OF TRICKLING FILTER

Quantity of Waste water

Average Daily Water Supply = 104034.645 M^3/day

80 % of total water supplied as waste water

= 104034.645 x 0.8

= 83227.71 m^3/day

= 83.22 MLD (Million liters per day)

BOD of Raw sewage = 270 mg/ltr

BOD removal in primary tank = 30 % (Assume) Final effluent BOD desired = 20 mg/ltr, BOD left in sewage entering the filter unit is = W = 83.22 * 270 * 0.7 = 15728.58 Kg/day

Assuming the value of organic loading = 2200 Kg/hec/day

Therefore, volume of filtering media required = (Total BOD of raw sewage / permissible BOD loading)

= (15728.58/2200)

= 7.15 hec. = 7.15 * 10^4 M^3

Assume the effective depth of filter = 2 meter

Therefore, Surface area of the filter required = 7.15 * 10^4 / 2 = 35750 M^2

Using a circular trickling filter of diameter 40 meter –

Number of units required = (Total area required / Area of one unit)

= (35750 / ((3.14/4)*40*40))

= 28.46 ≈ = 29 units

Check for hydraulic Loading –

Surface area of the filter bed required can also be worked out by assumed the value of hydraulic loading, say as 25 million liters per day.

Therefore surface area required = (Total sewage to be treated per day / Hydraulic loading per day)

$$= (83.22 / 25) * 10000 = 33288 \ M^2$$

So we are taking surface 33288 > 13800.

Hence, 29 Units each of 40 meter diameter and 2 meter effective depth can be adopted.

DESIGN OF ROTARY DISTRIBUTION

Rotary distributors are to be designed for peak flow, which may considered as 2.25 times the average flow.

Therefore, peal sewage flow / day = 2.25 * 83.22 = 187.25 ML/day

This flow is divided into 5 filters.

Therefore, flow through each unit = at peak flow = $(187.25*10^3)/(5*60*60*24) = 0.433$ M^3/sec

Assuming that the velocity at peak flow is 2m/sec through the central column of the

distributors. Diameter of central column = $[(0.433*4\ )\ /\ ((3.14*2)]^{1/2} = 0.525$ m

Provide central column of 0.525 meter in diameter but check the velocity through the column at average flow, as it should not be less than 1 m/sec.

Check for velocity at average flow –

Discharge through each unit at average flow = (0.433*2.25)/5 = 0.195 M^3/sec.

Therefore, velocity at average flow = [0.195 / (3.14*0.5252/4)]

= 0.90 m/sec < 1 m/sec.

Therefore, we should reduce the adopted diameter to 0.4 meter.

Therefore, velocity at average flow = $[0.195/\ (3.14*0.4^2/4)] = 1.55$ m/sec > 1 m/sec. **Hence OK.**

The velocity at peak flow will then be = $[0.433\ /\ (3.14*0.4^2/4)] = 3.45$ m/sec.

Hence we use a central column of 0.4 meter diameter. However the central column of 0.3 meter diameter is not available, we may permit 0.4 meter diameter.

DESIGN OF ARMS

Now, let's use rotary spray type distributors with 4 arms.

Therefore, the discharge per arms = 0.433/4 = 0.1083 M^3/sec.

Diameter of filter are = 40 meter

Therefore, Arm length = (40-2) / 2 = 19 meter.

We can use each arm of 19 meter length with its size reducing from near the central column towards the end. The first two sections each of 6 meter length and third section of 7 meter can be used.

The flow in the arms has to be adjusted in the proportion of the filter area covered by these lengths of arms therefore the areas covered by the different lengths of arm are calculated first.

Let A1, A2 and A3 be the circular filters areas covered by each length of from the central column 0.3 meter diameter or 0.15 meter radius in the centre to be used for central column etc. these area would be used –

A1 = $3.14\ (r_2^2 - r_1^2) = 3.14\ (6.15^2 - 0.15^2) = 118.75\ M^2$

A2 = $3.14\ (12.15^2 - 6.15^2) = 344.95\ M^2$

A3 = $3.14\ (20.0^2 - 12.15^2) = 792.86\ M^2$

Total area of the filters = $3.14\ (20.0^2 - 0.15^2) = 1256.56\ M^2$

Therefore, proportional areas served by each section of arm are worked out as –

1^{st} = A1 / A = (118.69 / 1256.56)*100 = 9.45%

2^{nd} = A2 / A = (344.99 / 1256.56)*100 = 27.45%

3^{rd} = A3 / A = (792.86 / 1256.56)*100 = 63.10%

=100%

Now full discharge through as arm i.e. 0.1083 M^3/sec, will flow through the 1st section and this will go on reducing through the second and third section.

1. **Design of 1st section –**

   Discharge = 0.1083 M^3/sec

   Assuming the velocity through the arm as 1.2 m/sec

   The area required = 0.1083 / 1.2 = 0.0903 M^2

   Diameter required = $[(0.0903 \times 4) / 3.14]^{1/2}$ = 0.339 meter say = 0.340 meter = 340 mm

2. **Design of 2nd section –**

   Discharge through 2nd section = (100-9.45) / 100 * 0.1083 = 0.0981 M^3/sec

   Assuming the velocity through the arm as 1.2 m/sec

   The area required = 0.0981 / 1.2 = 0.0818 M^2

   Diameter required = $[(0.0818 \times 4) / 3.14]^{1/2}$ = 0.322 meter say = 0.325 meter = 325 mm

3. **Design of 3rd section –**

   Discharge through 3rd section = (100-9.45-27.45) / 100 * 0.1083 = 0.0683 M^3/sec

   Assuming the velocity through the arm as 1.2 m/sec

   The area required = 0.0683 / 1.2 = 0.0569 M^2

   Diameter required = $[(0.0569 \times 4) / 3.14]^{1/2}$ = 0.269 meter say = 0.270 meter = 270 mm

Each arm length is made up of 3 sections i.e. first 6 meter from centre of 300 mm diameter, next 6 meter of 300 mm diameter and the last 7 meter of 250 mm diameter. If economy is not much affected and as if different sizes of pipes are difficult to joint then the entire arm length may e kept of 300 mm diameter.

DESIGN OF ORIFICES

Each arm section will be provided with different number of orifices depending upon the discharge to be passed through each section.

Total discharge through each arm = 0.1083 M^3/sec.

Assuming that 10mm diameter orifices are provided with co-efficient of discharge (cd) being 0.65, we have –

The discharge through each orifice with an assumed water head, causing flow as 1.5 meter .

$= cd – A (2gh)^{1/2}$

$= 0.65 – (3.14*0.01^2/4)*(2*9.81*1.5)$

$= 2.769*10^{-4}$ M^3/sec

Total number of orifices through each arm –

= (Total discharge through each arm / Discharge through each orifice)

$= [\ 0.1083/(2.769*10^{-4})$

= 391.11 say = 392 Nos.

Hence use total 392 orifices in the arm, then the number of orifices in each section of the arm is given as

Number of orifices through 1^{st} section = (9.45/100)*392 = 37.04 say 38 nos.
Number of orifices through 2^{nd} section = (27.45/100)*392 = 107.60 say 108 nos.
Number of orifices through 3^{rd} section = (63.10/100)*392 = 247.35 say 248 nos.

Spacing of orifices –

In 1^{st} section 20 nos in 6 meter length = 6/38 = 0.158 meter c/c
In 2^{nd} section 57 nos in 6 meter length = 6/108 = 0.0556 meter c/c
In 3^{rd} section 20 nos in 7 meter length = 7/248 = 0.0282 meter c/c

DESIGN OF UNDER-DRAINAGE SYSTEM

Total discharge through each filter unit of peak flow = 0.433 M^3/sec

The size and slope of the rectangular effluent channel should be such as to allow say a velocity of 1m/sec through it.

Therefore, area of channel = discharge / velocity = 0.433 / 1 = 0.433 M^2

We have assumed width = 0.225 meter. So we have its depth = 0.433/0.225 = 1.92

say 2.00 meter

The slope of the bed of this channel is given by –

$Q= (1/N) * A * R^{2/3} * S^{1/2}$

Where,

N = 0.018

A = 0.225*1.5 = 0.3375 M^2

P = 0.225+1.5+1.5 = 3.225

R = A/P = 0.3375 / 3.225 = 0.105

Now putting all the above values in the equation, we get –

$0.433 = (1/0.018) * 0.225 * 0.105^{2/3} * S^{1/2}$

$S = [\ (0.433*0.018) / (\ 0.225 * 0.105^{2/3})]^2$

S = 0.024 say 1 in 10.

Hence use a central effluent channel 0.225 meter in width and 1.5 meter depth below the bottom level of laterals. The channel may laid at a slope of 1 in 10.

Let's use a 10 cm diameter semicircular under drainage blocks. These laterals should be designed to run approximately ½ full so as to ensure proper ventilation. Let's assume that laterals runs at a depth of say 0.3 D, where D is the diameter of circle of which the laterals section is a semi circle.

By interpolation,

d/D = 0.30, so we get –

We know that,

$d = 0.3D$

$D = 10cm = 0.10m$ $d/D = (1/2)(1 – \cos(ø/2))$

$0.3 = (1/2)(1 – \cos(ø/2))$

$\cos(ø/2) = 0.4$

$ø = 132.84$ so, $\sin ø = 0.733$

For partially full running drain,

$a/A = [(ø/360) – (\sin ø/2*3.14)]$

$= [(132.84/360) – (0.733/2*3.14)]$

$= 0.252$

So, $a = 0.252A$

$= 0.252 \text{ X } 3.14/4 \text{ X } (0.10)^2$

$= 1.98 \text{ X } 10^{-3} M^2$

$r/R = [1 – (360* \sin ø /2*3.14* ø)]$

$= [1 – (360* 0.733 /2*3.14* 132.84)]$

$= 0.968$

So, $r = 0.968R$ $R = D/4$

$= 0.968*0.025$ $= 0.10/4 = 0.025M$

$= 0.0242$

$V = (1/N) * R^{2/3} * S^{1/2}$

$= (1/0.013)*(0.0242)^{2/3}*(1/40)^{1/2}$

$= 1.02$m/s > 0.75m/s………………(min. required) So,

$q = a \times v$

$= 1.98 \times 10^{-3} \times 1.02$

$= 0.002$ M^3/sec

Also

$Q = (1/N) * A * R^{2/3} * S^{1/2}$

Q (discharge) through circular sewer of D=0.1 meter is given by –

$Q = (1/0.013)*(3.14*0.1^2/4)*(0.1/4)*(1/\sqrt{40}) = 0.0023$ M^3/sec

q = 0.196 Q ……by interpolation d/D = 0.3

q = 0.196*0.0023 = 0.0045 M^3/sec Discharge

through the filter = 0.1083 M^3/sec Discharge

through each laterals = 0.0045 M^3/sec

Number of laterals required = 0.1083/0.0045 = 24.07 say 25 numbers.

So use 25 numbers laterals in all void laterally in the circular filter tank of 40 meter in diameter.

Now, velocity through the laterals at peak flow = $q_{avg}/Q = 0.002 / (0.252*3.14*0.1^2/4)$

= 1.01 m/sec > 0.75 m/sec .

Hence OK.

Velocity at average flow –

Q at average flow = 0.002 / 2.25 = 0.00089

M^3/sec q_{avg}/Q = 0.002/0.0023 = 0.386

For q/Q of 0.087, d/D = 0.2 and

Q/A = 0.143

Therefore, Q_{avg} = 0.143A

$V_{avg} = q_{avg} / Q_{avg} = 0.0046 / (0.143*3.142*0.1^2/4) = 4.095 > 0.5$ m/sec.

Hence
OK.

Hence use 19 semicircular radial laterals for discharge into the effluent channel.

DESIGN OF AERATION TANK

Number of tank = 05

Average flow to each tank = 83.22/5 MLD

= 16.64 MLD

= 16640 M^3/day

The total BOD entering S.T.P = 270 mg/ltr

Assuming that 30% BOD removal in Grit chamber.

BOD removed = (270*30 / 100) = 81 mg/ltr

BOD entered in Aeration Tank –

γ_o= 270-81 = 189 mg/ltr

BOD left in effluent γ_E = 20 mg/ltr

BOD removed in activated plant = 189-20 = 169 mg/ltr

Minimum efficiency required in activated plant = 169/189 = 89.42% Hence OK.

Volume of activation tank can be designed by assuming a suitable value of MLSS or Q or F/M

Let us assume F/M ratio = 0.12 (0.10 to 0.18)

MLSS = 3000 mg/ltr Now, using equation

– F/M = (Q/V)*(γ_o/γ_E)

Where, Q = 16640 M^3/day

V = ?

$F/M = 0.12$

γ_o = 189 mg/ltr

γ_o = 3000 mg/ltr

0.12 = (16640 / V) * (189 / 3000)

V = 8736 M^3

Aeration tank dimension let us adopt an aeration tank of liquid depth 5 meter and 11 meter width.

Length of tank = Volume / (breadth * depth) = 158.83 meter ≈ 159 meter

Therefore, V provided = 159*5*11= 8745 M^3

i) Check for aeration period as H.R.T (t) –

t=Y/Q*24

= (3410/8680)*24

= 9.42 h ≈ 9.5 h

ii) Check for volumetric loading –

= S Y_o / V gm of BOD 1M^3 volume of tank.

= (16640*189)/(8745*10^3)

= 0.360 gm/M^3

Since it should lies between 0.2 to 0.4.Hence OK

iii) Check for return sludge ratio –

$Q_R / Q = Vt / [(10^6/SVI) – Xt]$

$= 3000 / [(10^6/169) – 3000]$ Since SVI = 169 mg/gm.

$=1.02$ say 1.00

Since it should lies between 0.5 to 1.0. Hence OK.

iv) Check for S.R.T. (Qc)
–

$V*Xt = \alpha_y * Q (Y_o - Y_E)*Qc / (1+Kc*Qc)$

α_y= 1.0 (Constant for municipal sewage W.R. to MLSS)

Y_o = 189 mg/ltr

Y_E = 20 mg/ltr

V = 8745 M^3

Xt = 3000 mg/ltr

Q = 16640 M^3/day

8745*3000 = 1.0 * 16640 (189 - 20)*Qc / (1+0.6*Qc)

Qc = 23.91 Days.

Since it lies between 10 to 25 days. The adopted size is therefore OK.

Hence adopt an aeration tank having an overall size of 133 meter x 11 meter, overall depth 6 m from free board.

AERATION SIZING –

BOD applied to each tank = 189 mg/ltr

Average flow in each tank = 16640 M^3/day

BOD to be removed in each tank = 16640*0.189

= 3145 Kg/day ≈ 3150 Kg/day

= 131.25 Kg/Hr

Oxygen required = 1.2% / kg BOD applied. Peak Oxygen demand = 125 %

Oxygen transfer capacity of the applied in standard condition = 1.9 Kg/Hwh = 1.41 Kg /Hp /Hr.

Oxygen transfer capacity of the aerators in field condition = 0.70 * 1.41 = 0.98 Kg /Hp /Hr.

Oxygen to be applied in each tank = 1.2 * 131.25 * 1.25

= 196.88 ≈ 197 Kg/Hr.

Hp of aerators required = 197 / 0.98 Hp = 201 ≈ 200 Hp

Provide 10 generators of 18 Hp.

Check for mixing considerations –

As per practice, power required for mixing = 0.02 KN / M^3

Volume of each aeration tank = 3410 M^3

Hp required = 0.02 * 8745 = 175 KN

Provide 4 or 6 aerators & considering the efficiency as 97% .

Hp of each aerator required for 4 aerators = [68.2 / (4*.97)] = 25.23 Hp

Consider power margin of 25% on motor rating = 25.23*1.25 = 31.54 Hp
Provide 04 nos of 20HP motors in each tank of aerators –

For 6 nos = 97.9 / (6*0.97) = 16.82 HP

Considering a power margin of 25% of motor rating –

Motor HP required = 16.82 * 1.25 = 21.03 ≈ 22 HP

Provide 06 nos of 22 HP motor aerators in each tank.

DESIGN OF SECONDARY SEDIMENTATION TANK

Assuming detention period =2 Hrs.

Surface Loading =60000 m^3 /m/day

The quantity of sewage to be treated per 2 hrs.

$$= \frac{1.44 \times 60 \times 60 \times 24 \times 2}{24}$$

= 10368 m^3

Capacity of tank = 10368 m^3

So, provide 2 units of S.S.T

Now surface Loading = $(86.4 \times 10^6) / (2 \times \frac{3.14}{4} \times d^2)$

So, d = 39.5

D = 40 m

Now effective depth of tank

= $10368 / (2 \times 3.14 \times 40^2)$

= 4.12

= 4.25 m

Max Perimeter =3.14 x d

=3.14 x 40

=125.6 m

Weir Length 125.6 m

Weir Loading = Q/WL

= 1.44/125.6

$= 0.0114 \times 10^3 \; m^3/sec$

$=687.6 \; m^3/m/day$

$=687.6 \; m^3/m/day > 500$ Hence OK

No. of 90 V- notches assuming c/c spacing at 200 mm

No. of Weirs = 125.6/2

= 62.8

= 63. Nos.

DESIGN OF SLUDGE DIGESTION TANK

Provide 2 Units

Average Sewage flow = 86.4 ml/day

= 43.2 mlQ

Total Suspended Solids = (600 x 85)/100

=510

(By assuming 15 % of TSS removed in screening and grit chamber)

Suspended solids in 43.2 ml of sewage flowing/day

= (510 x 43.2 x 10^6)/ 10^6

=22032 kg/day

Assuming that 65 % solids are removed in primary settling tank

Wt. of solids removed in the PST

= 65 % of 22032

= 14320.80 kg/day

Assuming that the fresh sludge has M.C. of 95 %

5 Kgs of dry solids weir make

= 100 kg of wet sludge

For 14320.80 Kgs. = (100/5) x 14320.80

=286416 kgs of wet sludge / day

Assuming specific gravity of sludge as 1.02

i.e. density = 1020 kg/m^3

Volume of raw sludge produced /day

V_1= 286416/1.02

=280.80 m^3/day

Volume of digested sludge V_2 at assumed 85 % M.C.

$V_2 = V_1$ ((100- S)/(100-S))

= 280.80 ((100-95)/(100-85))

=93.33 m^3/day

Assume the digestion period as 30 days

Capacity = (280.80)- ((2/3 x (280.80-93.33)) x 30

= 4074.60 m^3

Now provide 5 m depth of the cylindrical digestion tank

Cross section are of tank

= 4074.60/5

= 814.92 m^2

So dia of tank = $((814.92/(3.14/4))^{1/2}$

= 32.21

= 33 m

Hence provide a cylindrical sludge digestion tank 5 m deep and 33 m dia with an additional hopper bottom of 1 : 1 slope for calculation of digested sludge.

DESIGN OF SLUDGE DRYING BEDS

Volume of wet sludge produced = mass of sludge /density of sludge

= 286416/1.02

=280.80 m^3/day

Assume that it should be spread in 22.5 cm thick layer (i.e between 20 to 30 cm)

Area of beds required = 280.80/0.0225

=1248.0 m^3/ day

Assuming that it should be spread in 22.5 cm thick layer. Under tropical Indian condition the beds fet dried out on about 10 days and hence tacking and weeks as average drying time including wet days of rainy season

We can utilized same bed = 52/5

= 26 (52 weeks in a year)

Area of bed required per year = (1248 x 365)/26

= 17520 m^2

Making 100 % allowance for space for storage repairs and resting of bed etc.

Total area of beds required = 2 x 17520

=35040 m^2

Now using 15 x 30 m size bed No. of beds required = (35040) /(15 x30)

= 77.87

So let us use 78 Nos beds with size as,

Area = 35040/78

= 450 m^2

Using 15 m width

Length = 450 /15

= 30 m

Hence use 78 beds of size (15 x 30) min plan

The beds should be provided with under drain and side walls

SUMMERY

1) POPULATION

Population from geometric growth method= 760627/-

2) SUMP WELL

a. Depth of Sump well =5 m
b. Area of Sump well =17.28 m^2
c. Dia. of Sump Well= 4.70 m

3) PUMP

4 Pumps

4) SCREENING

a. Gross area of Screen =2.49 m^2
b. Depth of Screen = 0.5 m
c. Width of Screen =0.01 m

5) GRIT CHAMBER

a. 2 Units provided
b. Length (L) =15 m
c. Width (B) =1.8 m
d. Depth (D) = 1.0 m

6) PRIMARY SEDIMENTATION TANK

a. Dia. Of tank =47 m
b. Depth of tank = 3 m
c. Provide 740 no. of weir with height =0.136 m

7) TRICKLING FILTER

a. Provide 5 no. of trickling filter = 29 nos.
b. Dia. Of trickling filter = 40 m
c. Effective depth of trickling filter = 2 m

8) ROTARY DISTRIBUTION

a. Provide central column 0.4 m dia.

9) ARMS

a. Discharge through arm =0.1083 m^3/sec
b. In 1^{st} section =29 Nos
c. In 2^{nd} section =82 Nos.
d. In 3^{rd} section = 188 Nos.

10) UNDER DRAIN SYSTEM

a. Provide 25 laterals in all, filled radial in circular filter tank of 40 m in dia

11) AERATION TANK

a. Provide 5 no of tanks
b. Dimension = 133 x 11 m (Overall depth with 0.6 m free board)

12) SECONDARY SEDIMENTATION TANK

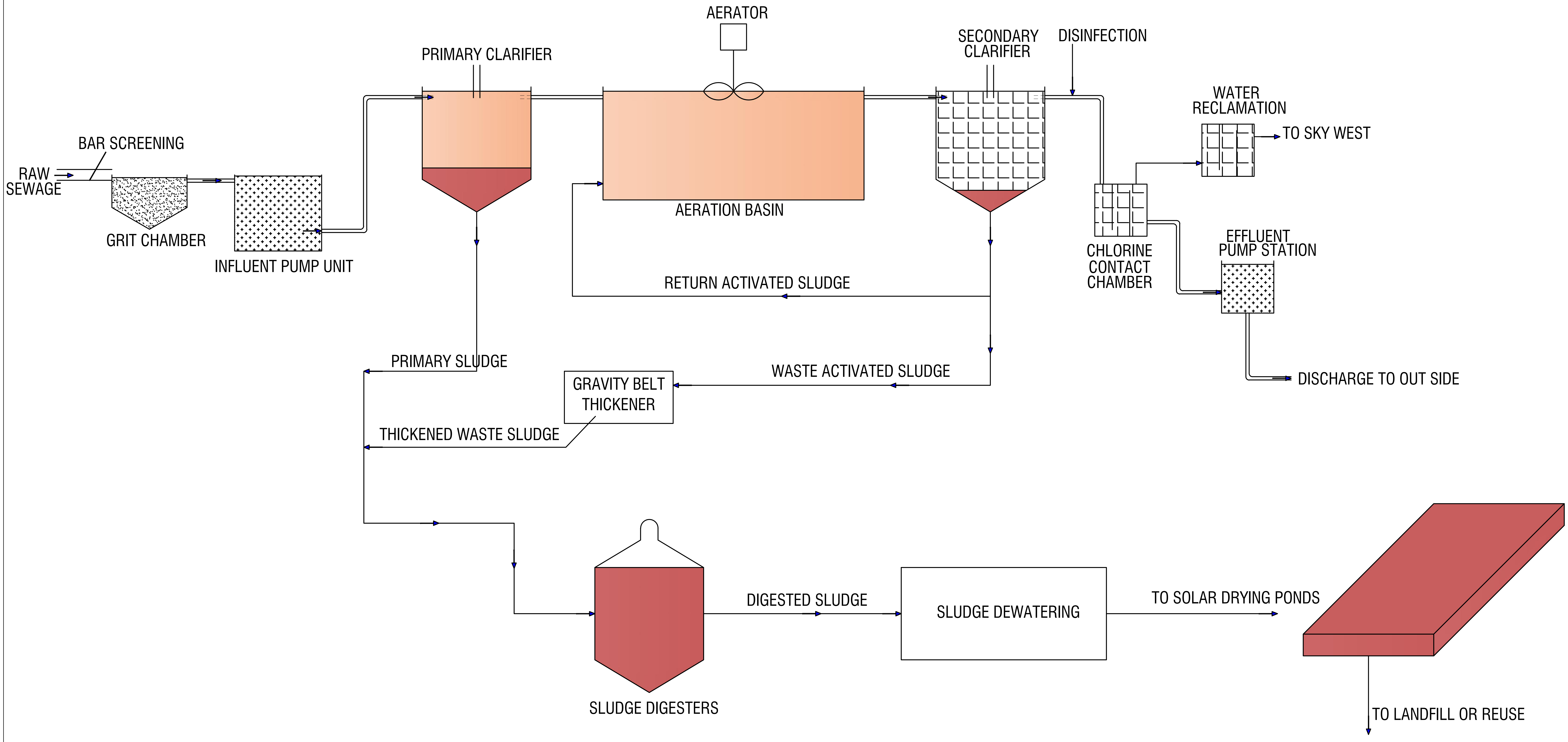
a. Provide 2 Nos
b. Dia of Tank = 40 m
c. Effective Depth = 4.25 m
d. Provide 63 Nos. of weir with
e. Height = 0.030 m

13) SLUDGE DIGESTION TANK

a. Provide 2 Nos. cyclical sludge digestion tank with,
b. Depth = 5 m
c. Diameter =33 m

14) SLUDGE DRYING BEDS

a. Provide 78 beds of size 15 x 30 m
b. Bed should provide with under drain and side walls



SEWAGE TREATMENT PROCESS LAYOUT

ENROLLMENT NO. : 130280906048 to 130280906120		
SUB : PROJECT - I	SEM : 6	BRANCH : PDDC CIVIL
L D COLLAGE OF ENGINEERING		