MODULE I

ENVIRONMENTAL ENGINEERING II

SANITARY ENGINEERING

After the water that has been made available from well laid and properly executed water supply project has been consumed, it has to be suitably disposed off. The other wastes from the society are also has to be carried and disposed off at suitable places so as to protect the public health. Thus the end of water supply scheme, in a sence, is the starting point of sanitary project.

SANITARY ENGINEERING

Sanitary Engineering is the branch of public health engineering which deals with the preservation and maintenance of health of the individual and the community, by preventing communicable diseases. It consists of scientific and methodical collection, conveyance, treatment and disposal of waste water, so that public health can be protected from the offensive and injurious substances. Sanitation is the prevention of the sporadic outbreak of diseases, and can be achieved by either controlling or eliminating such environmental factors that contribute in some form or the other to the transmission of disease.

1.3 SEWAGE

Sewage indicates the liquid waste from the community. It includes waste water from bath rooms, kitchens, discharge from latrines, urinals, stables, and industrial waste and also the ground surface and storm water that may be admitted in to the sewer.

1.4 REFUSE

Refuse is a general term used to indicate what is rejected or be left out as worthless. It may be in liquid, semi-solid or solid form and may be divided into six categories.

i) Garbage

ii) Rubbish

iii) Sullage

iv) Subsoil water

v) Storm water

vi) Sewage

i) Garbage

Garbage indicates dry refuse. It includes waste paper, decayed fruits and vegetables, grass and leaves, and sweepings from streets, markets and other public places. Thus, garbage contains large amounts of organic matter.

ii) Rubbish

Rubbish indicates sundry solid wastes from offices, residences and other buildings. It also includes waste building materials, brocken furniture, paper, rags etc. Generally rubbish is dry and is of combustible nature.

iii) Sullage

Sullage is a term used to indicate the waste water from bath rooms, kitchen, washing places and wash basins etc. It does not create bad smell since organic matter in it is absent or of negligible amount.

iv) Sub-soilwater

It is the ground water that finds its entry into sewers through leaks.

v) Storm water

It indicates the rain water of the locality.

vi) SEWAGE

Sanitary sewage or domestic sewage indicates sewage mainly derived from the residential building and industrial establishments. It is extremely foul in nature. Sanitary sewage may be classified as:

i) Domestic Sewage
ii) Industrial Sewage

i) Domestic Sewage

It is the sewage obtained from the lavatory basins, urinals and water closets of residential buildings, office buildings, theatres and other institutions. Since it contains human excreta and urine, it is extremely foul in nature.

ii) Industrial Sewage

It is waste water obtained from the industrial and commercial establishments. It may contain objectionable organic compounds that may not be amenable to conventional treatment processes.

NIGHT SOIL

It is a term used to indicate the human and animal excreta.

1.7 SOURCES OF WASTEWATER

Wastewater or sewage can come from:

1. Water supplied by water authority for domestic usage, after desired use it is discharged in to sewers as sewage.

2. Water supplied to the various industries for various industrial processes by local authority. Some quantity of this water after use in different industrial applications is discharged as wastewater.

3. The water supplied to the various public places such as, schools, cinema theaters, hotels, hospitals, and commercial complexes. Part of this water after desired use joins the sewers as wastewater.

1.7 SOURCES OF WASTEWATER

4. Water drawn from wells by individuals to fulfill domestic demand. After uses this water is discharged in to sewers.

5. The water drawn for various purposes by industries, from individual water sources such as, wells, tube wells, lake, river, etc. Fraction of this water is converted into wastewater in different industrial processes or used for public utilities within the industry generating wastewater. This is discharged in to sewers.

6. Infiltration of groundwater into sewers through leaky joints.

7. Entrance of rainwater in sewers during rainy season through faulty joints or cracks in sewers.

SEWER

It is an under ground conduit or drain through which sewage is carried to a point of discharge or disposal. Separate sewares are those which carry the house hold and industrial waste only. Combined sewers are those which carry both sewage and storm water.

METHOD OF CARRYING REFUSE

Following are the two methods which are employed for the collection and disposal of refuse of a locality.

i) Conservancy system

ii) Water carriage system

METHOD OF CARRYING REFUSE

i) Conservancy system

In this system, the different types of refuse are collected seperately and then each type is carried and suitably disposed off.

The garbage or dry refuse is collected from roads and streets in pan or baskets. It is then conveyed by carts, trucks etc.

The night soil is collected in pans from lavatories and the sewage is carried by labour in carts, trucks etc. It is then burried into the ground. The storm water and sullage are collected and conveyed separately by closed or open channels. They are discharged in natural rivers or streams.

METHOD OF CARRYING REFUSE

ii) Water carriage system

In this system, water is used as medium to carry the sewage to the point of its treatment or final disposal.

The quantity of water to be mixed with solid matter is quite sufficient and dilution ratio of solid matter with water is so great that the mixture behaves more or less like water. The sewage is conveyed in suitably designed and maintained sewers.

TYPES OF SEWERAGE SYSTEM

The sewerage system can be of following three types:

i) Combined system

ii) Separate System

iii) Partially separate system

i) Combined system: In combined system along with domestic sewage, the run-off resulting from storms is carried through the same conduit of sewerage system. In countries like India where actual rainy days are very few, this system will face the problem of maintaining self cleansing velocity in the sewers during dry season, as the sewage discharge may be far lower as compared to the design discharge after including storm water.

Advantages of combined system

- In an area where rainfall is spread throughout a year, there is no need of flushing of sewers, as self cleansing velocity will be developed due to more quantity because of addition of storm water.
- Only one set of pipe will be required for house plumbing.
- In congested areas it is easy to lay only one pipe rather than two pipes as required in other systems.

Disadvantages combined system

- Not suitable for the area with small period of rainfall in a year, because dry weather flow will be small due to which self cleansing velocity may not develop in sewers, resulting in silting.
- Large flow is required to be treated at sewage treatment plant before disposal, hence resulting in higher capital and operating cost of the treatment plant.
- When pumping is required this system is uneconomical.
- During rains over flowing of sewers will spoil public hygiene.

ii) **Separate System:** In separate system, separate conduits are used; one carrying sewage and other carrying storm water run-off. The storm water collected can be directly discharged into the water body since the run-off is not as foul as sewage and no treatment is generally provided. Whereas, the sewage collected from the city is treated adequately before it is discharged into the water body or used for irrigation to meet desired standards. Separate system is advantageous and economical for big towns.

Advantages of separate system

- As sewage flows in separate pipe, hence the quantity to be treated at sewage treatment plant is small, resulting in economy of treatment.
- This system may be less costly as only sanitary sewage is transported in closed conduit and storm water can be collected and conveyed through open drains.
- When pumping is required during disposal, this system is economical due to less flow.

Disadvantages of separate system

- Self cleansing velocity may not develop at certain locations in sewers and hence flushing of sewers may be required.
- This system requires laying two sets of pipe, which may be difficult in congested area.
- This system will require maintenance of two sets of pipe lines and hence maintenance cost is more.

iii) Partially separate system: In this system part of the storm water especially collected from roofs and paved courtyards of the buildings is admitted in the same drain along with sewage from residences and institutions, etc. The storm water from the other places is collected separately using separate storm water conduits.

Advantages of partially separate system

- Economical and reasonable size sewers are required.
- Work of house plumbing is reduced as rain water from roofs, sullage from bathrooms and kitchen, etc. are combined with discharge from water closets.
- Flushing of sewers may not be required as small portion of storm water is allowed to enter in sanitary sewage.

Disadvantages of partially separate system

- Increased cost of pumping as compared to separate system at treatment plants and intermediate pumping station wherever required.
- In dry weather self-cleansing velocity may not develop in the sewers.

QUANTITY OF SEWAGE

The sewage collected from the municipal area consists of wastewater generated from the residences, commercial centers, recreational activities, institutions and industrial wastewaters discharge into sewer network from the permissible industries located within the city limits. Before designing the sewer, it is necessary to know the discharge i.e., quantity of sewage, which will flow in it after completion of the project.

QUANTITY OF SEWAGE

Accurate estimation of sewage discharge is necessary for hydraulic design of the sewers. Far lower estimation than reality will soon lead to inadequate sewer size after commissioning of the scheme or the sewers may not remain adequate for the entire design period. Similarly, very high discharge estimated will lead to larger sewer size affecting economy of the sewerage scheme, and the lower discharge actually flowing in the sewer may not meet the criteria of the self cleansing velocity and hence leading to deposition in the sewers.

QUANTITY OF SEWAGE

The Actual measurement of the discharge is not possible if the sewers do not exist; and where the capacity of the existing sewers is inadequate and need to be increased, still actual present discharge measurement may not be accurate due to unaccounted overflow and leakages that might be occurring in the existing system. Since sewers are design to serve for some more future years, engineering skills have to be used to accurately estimate the sewage discharge.

In order to determine the section of thesewer, it is essential to know the total quantity of wastewater or sewage that would flow through the sewer. The total waste water flow can be divided into two components.

1. Dry weather flow
2. Storm weather flow

DRY WEATHER FLOW

Dry weather flow (DWF) is the flow that occurs in sewers in separate sewerage system or the flow that occurs during dry seasons in combined system. This flow indicates the flow of sanitary sewage. This depends upon the rate of water supply, type of area served, economic conditions of the people, weather conditions and infiltration of groundwater in the sewers, if sewers are laid below groundwater table etc.

Factors affecting dry weather flow

The dry weather flow or quantity of sanitary sewage depends upon the following factors.

1. Rate of water supply.
2. Population growth.
3. Type of area served.
4. Infiltration of ground water.

1. Rate of water supply

The quantity of wastewater produced from a community would naturally depend upon the rate of water supply per capita per day. The quantity of wastewater entering the sewer would be less than the total quantity of water supplied. This is because of the fact that water is lost in domestic consumption, evaporation, lawn sprinkling, fire fighting, industrial consumption etc. However, private source of water supply and infiltration of sub-soil water in the sewers increase the wastewater flow rate. This extra water that enters the sewer can be assumed to approximately equal to the water lost in consumption etc. If however, one is sure that no extra water enters the sewers, the wastewater quantity may be assumed to be 80% of the quantity of water supply. The sewers should be designed for a minimum of 150 lpcd.

2. Population growth

The quantity of sewage is depending upon the population of the city. Just as in case of water supply projects, the future population after two or three decades is determined by applying any suitable method of population forecast.

3. Type of area served

The quantity of wastewater produced depends upon weather the area served is residential, commercial, or industrial. The wastewater from the residential area directly depends upon the rate of water supply. If there is no infiltration of water in the sewers, and if there are no private sources of supply, the wastewater produced from the residential area may be assumed to be equal to 70 to 80% of the water supplied through the public supply system.

4. Infiltration of sub-soil water

Groundwater or subsoil water may infiltrate into the sewers through the leaky joins. Exfiltration is the reverse process which indicates the flow of wastewater from the sewer into the ground. While due to the infiltration the quantity of flow through sewer increases exfiltration results in decrease in the flow and consequent increase in the pollution of ground water. Both infiltration as well as exfiltration are undesirable and take place due to imperfect joints. However, infiltration is much more important from the point of sewer design. Also, infiltration unnecessarily increases the load on the treatment works.

VARIATION IN SEWAGE FLOW

Variation occurs in the flow of sewage over annual average daily flow. Fluctuation in flow occurs from hour to hour and from season to season. The typical hourly variation in the sewage flow is shown in the Figure 9.1. If the flow is gauged near its origin, the peak flow will be quite pronounced. The peak will defer if the sewage has to travel long distance. This is because of the time required in collecting sufficient quantity of sewage required to fill the sewers and time required in travelling. As sewage flow in sewer lines, more and more sewage is mixed in it due to continuous increase in the area being served by the sewer line.

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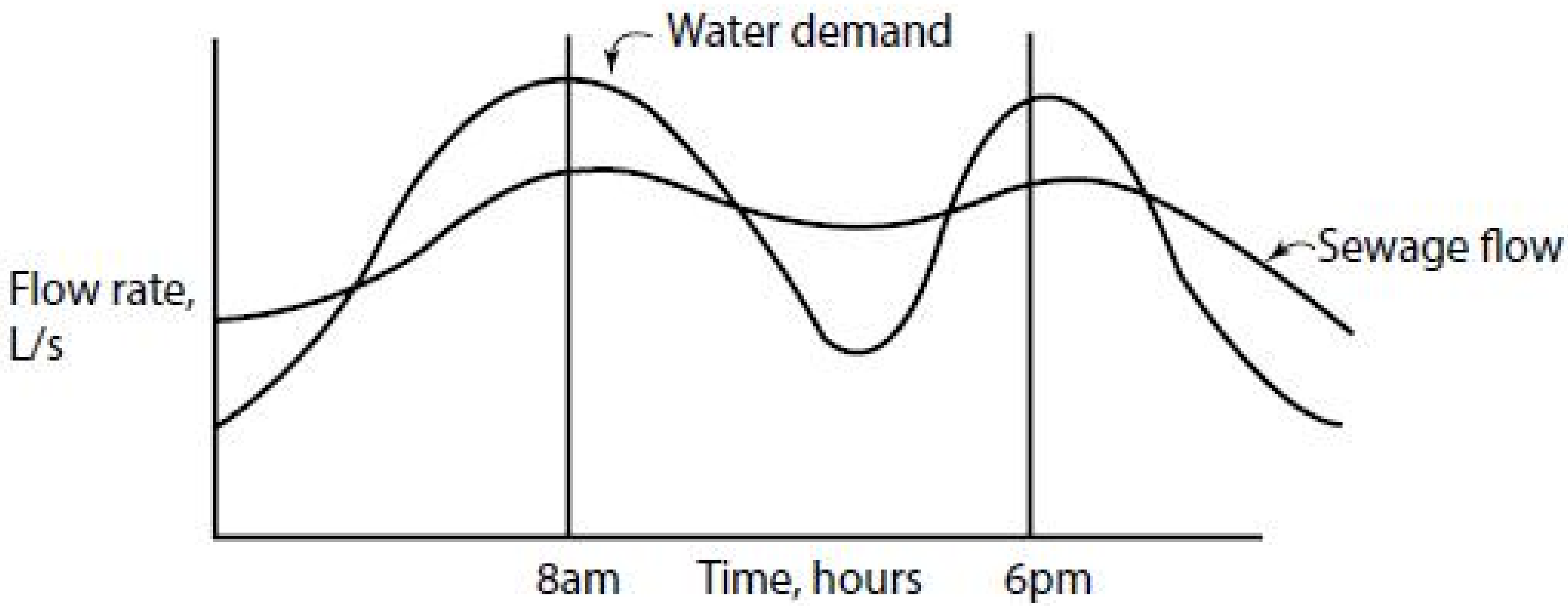

Figure 2.1 Typical hourly variations in sewage flow

This leads to reduction in the fluctuations in the sewage flow and the lag period goes on increasing. The magnitude of variation in the sewage quantity varies from place to place and it is very difficult to predict. For smaller township this variation will be more pronounced due to lower length and travel time before sewage reach to the main sewer and for large cities this variation will be less.

For estimating design discharge following relation can be considered:

Maximum daily flow = Two times the annual average daily flow (representing seasonal variations)

Maximum hourly flow = 1.5 times the maximum daily flow (accounting hourly variations)

= Three times the annual average daily flow

As the tributary area increases, peak hourly flow will decrease. For smaller population served (less than 50000) the peak factor can be 2.5, and as the population served increases its value reduces. For large cities it can be considered about 1.5 to 2.0. Therefore, for outfall sewer the peak flow can be considered as 1.5 times the annual average daily flow. Even for design of the treatment facility, the peak factor is considered as 1.5 times the annual average daily flow. The minimum flow passing through sewers is also important to develop self cleansing velocity to avoid silting in sewers. This flow will generate in the sewers during late night hours. The effect of this flow is more pronounced on lateral sewers than the main sewers.

Sewers must be checked for minimum velocity as follows:

Minimum daily flow = 2/3 Annual average daily flow

Minimum hourly flow = ½ minimum daily flow

= 1/3 Annual average daily flow

The overall variation between the maximum and minimum flow is more in the laterals and less in the main or trunk sewers. This ratio may be more than 6 for laterals and about 2 to 3 in case of main sewers.

DESIGN PERIOD

The branches and main sewers are designed for the population which may occur at the end of one generation of 30 years. This period of 30 years is called the design period. However, the pumping plants etc. are designed for a design period of 5 to 10 years only. The treatment units are designed for 10 to 30 year period. The design period depends upon the following:

1. Ease and difficulty in expansion,
2. Amount and availability of investment,
3. Anticipated rate of population growth, including shifts in communities, industries and commercial investments,
4. Hydraulic constraints of the systems designed, and
5. Life of the material and equipment.

Following design period can be considered for different components of sewerage scheme.

Table 9.1 Period can be considered for different components

1	Laterals less than 15 cm diameter	Full development
2	Trunk or main sewers	40 to 50 years
3	Treatment Units	40 to 50 years
4	Pumping plant	5 to 10 years

DESIGN DISCHARGE OF SANITARY SEWAGE

The total quantity of sewage generated per day is estimated as product of forecasted population at the end of design period considering per capita sewage generation and appropriate peak factor. The per capita sewage generation can be considered as 75 to 80% of the per capita water supplied per day. The increase in population also result in increase in per capita water demand and hence, per capita production of sewage. This increase in water demand occurs due to increase in living standards, betterment in economical condition, changes in habit of people, and enhanced demand for public utilities.

Problem 9.1

A city has a projected population of 60,000 spread over area of 50 hectare. Find the design discharge for the separate sewer line by assuming rate of water supply of 250 lpcd and out of this total supply only 75 % reaches in sewer as wastewater. Make necessary assumption whenever necessary.

Solution:

Given data

Rate of supply	= 250 lit/capita/day
Sewage flow	= 75% of water supply
	= 0.75 X 250
	= 187.5 lpcd

Total sewage generated	$= 187.5 \text{ X } \dfrac{60000}{24X60X60}$
	= 130.21 lit/sec
	$= 0.13 \text{ m}^3/\text{s}$
Assume peak factor	= 2
Total design discharge	=2 x 0.13 = **0.26 m^3/s.**

STORM WATER FLOW

When rainfall takes place, a part of it infiltrates or percolates into the ground surface. While the remaining flows over the land depending upon permeability of the ground, its surface slope and many other factors. The amount of water flowing over the ground surface, pavements, house roofs etc. is commonly known as runoff or the storm water. This stormwater is ultimately drained through the sewers; otherwise the streets, roads etc. would be flooded. The amount of peak storm water flow may be several times more than dry weather flow. In the case of combined system, the sewers are normally not designed for the peak storm water flow; otherwise the size of the sewers would be alarmingly large. The storm water flow is also known as wet weather flow and is abbreviated as WWF.

Factors Affecting the Quantity of Stormwater flow

The surface run-off resulting after precipitation contributes to the stormwater. The quantity of stormwater reaching to the sewers or drains is very large as compared with sanitary sewage. The factors affecting the quantity of stormwater flow are as below:

1. Area of the catchment
2. Slope and shape of the catchment area
3. Porosity of the soil
4. Obstruction in the flow of water as trees, fields, gardens, etc.
5. Initial state of catchment area with respect to wetness.

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Factors Affecting the Quantity of Stormwater flow

6. Intensity and duration of rainfall
7. Atmospheric temperature and humidity
8. Number and size of ditches present in the area
9. Condition of ground prior to the rainfall
10. Concentration or compactness of catchment area

MEASUREMENT OF RAINFALL

The rainfall intensity could be measured by using rain gauges and recording the amount of rain falling in unit time. The rainfall intensity is usually expressed as mm/hour or cm/hour. The rain gauges used can be manual recording type or automatic recording rain gauges.

METHODS FOR ESTIMATION OF QUANTITY OF STORM WATER

The purpose of estimating stormwater flow for sewer design, the following two methods are commonly followed.

1. Rational Method
2. Empirical formulae method

In both the above methods, the quantity of storm water is considered as function of intensity of rainfall, coefficient of runoff and area of catchment.

Time of Concentration

The time of concentration is defined as the longest time, without unreasonable delay, that will be required for a drop of water to flow from the farther point of the drainage area to the point of concentration.

Time of Concentration

When rainfall just starts all over the catchment area simultaneously, the rain drops falling just near the sewer inlet will enter the sewer first. The rate of flow at this stage will be very small, but it will gradually increase as more and more area contributes to the flow. Finally when the whole area is contributing, maximum rate of run off will be obtained, which will be equal to rate of precipitation over the whole of the impervious area. The time required from the beginning of rainfall to the one corresponding to the achievement of maximum rate of runoff is called the time of concentration. This maximum rate of runoff will continue till the rainfall stops. After that, the runoff will gradually decrease.

Time of Concentration

- The rainfall with duration lesser than the time of concentration will not produce maximum discharge.
- The runoff may not be maximum even when the duration of the rain is more than the time of concentration. This is because in such cases the intensity of rain reduces with the increase in its duration.
- The runoff will be maximum when the duration of rainfall is equal to the time of concentration and is called as *critical rainfall duration*. The time of concentration is equal to sum of inlet time and time of travel.

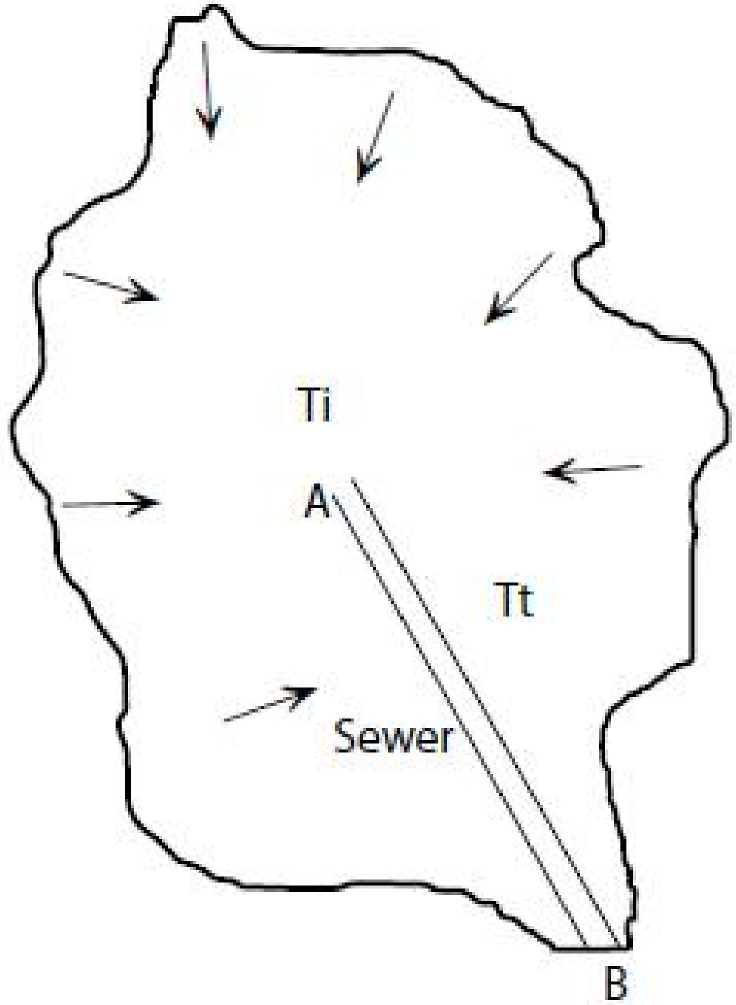

Figure 2.2 Runoff from a given catchment

Inlet Time: The time required for the rain in falling on the most remote point of the tributary area to flow across the ground surface along the natural drains or gutters up to inlet of sewer is called inlet time (Figure 9.2). The inlet time 'Ti' can be estimated using relationships similar to following. These coefficients will have different values for different catchments.

$$Ti = \left[0.885 \frac{L3}{H}\right]^{0.385}$$

Where,

Ti = Time of inlet, minute

L = Length of overland flow in Kilometer from critical point to mouth of drain

H = Total fall of level from the critical point to mouth of drain, meter

Time of Travel: The time required by the water to flow in the drain channel from the mouth to the point under consideration or the point of concentration is called as time of travel.

Time of Travel (Tt) = Length of drain/ velocity in drain

Runoff Coefficient: The total precipitation falling on any area is dispersed as percolation, evaporation, storage in ponds or reservoir and surface runoff. The runoff coefficient can be defined as a fraction, which is multiplied with the quantity of total rainfall to determine the quantity of rain water, which will reach the sewers. The runoff coefficient depends upon the porosity of soil cover, wetness and ground cover. The overall runoff coefficient for the catchment area can be worked out as follows:

Overall runoff coefficient, $C = [A_1.C_1 + A_2.C_2 + \ldots. + An.Cn] / [A_1 + A_2 + \ldots + An]$

Where, A_1, A_2, ….An are types of area with C_1, C_2, …Cn as their coefficient of runoff, respectively.

The typical runoff coefficient for the different ground cover is provided in the Table 9.2.

Table 9.2 Runoff coefficient for different type of cover in catchment

Type of Cover	Coefficient of runoff
Business areas	0.70 – 0.90
Apartment areas	0.50 – 0.70
Single family area	0.30 – 0.50
Parks, Playgrounds, Lawns	0.10 – 0.25
Paved Streets	0.80 –0.90
Water tight roofs	0.70 – 0.95

Rational method

The rational formula is commonly used for design of storm drains, in taken into account of the following factors:

1. Catchment area (A)
2. Impermeability factor (C) of the catchment area
3. Intensity of rainfall (I)

i) Catchment area

The catchment area served by a given storm water can be found directly from the map of the town showing the position of streets, houses, play grounds, sewers etc. The factor I depend up on type of the surface.

ii) Impermeability factor

The storm water flow depends upon the imperviousness of the surface over which rainfall takes place. If the ground is relatively impervious, more runoff takes place. The percentage of rain water that is available in the form of runoff is known as impermeability factor or runoff coefficient.

iii) Intensity of rainfall

The value of factor I, ie intensity of rainfall in mm/hour can be worked out from the rainfall records of the area. I also depend upon (a) storm frequency and (b) duration of the storm..

Storm Frequency: Storm frequency of storm for which the sewers are to be designed depends upon the importance of the drainage area. Commercial and highly priced areas should be subject to less frequent flooding than the residential area while the other unimportant areas can be subject to more frequent flooding.

Duration of storm: The duration of storm is taken equal to the time of concentration. It is the time required for the runoff to be contributed to the point of concentration from the entire area.

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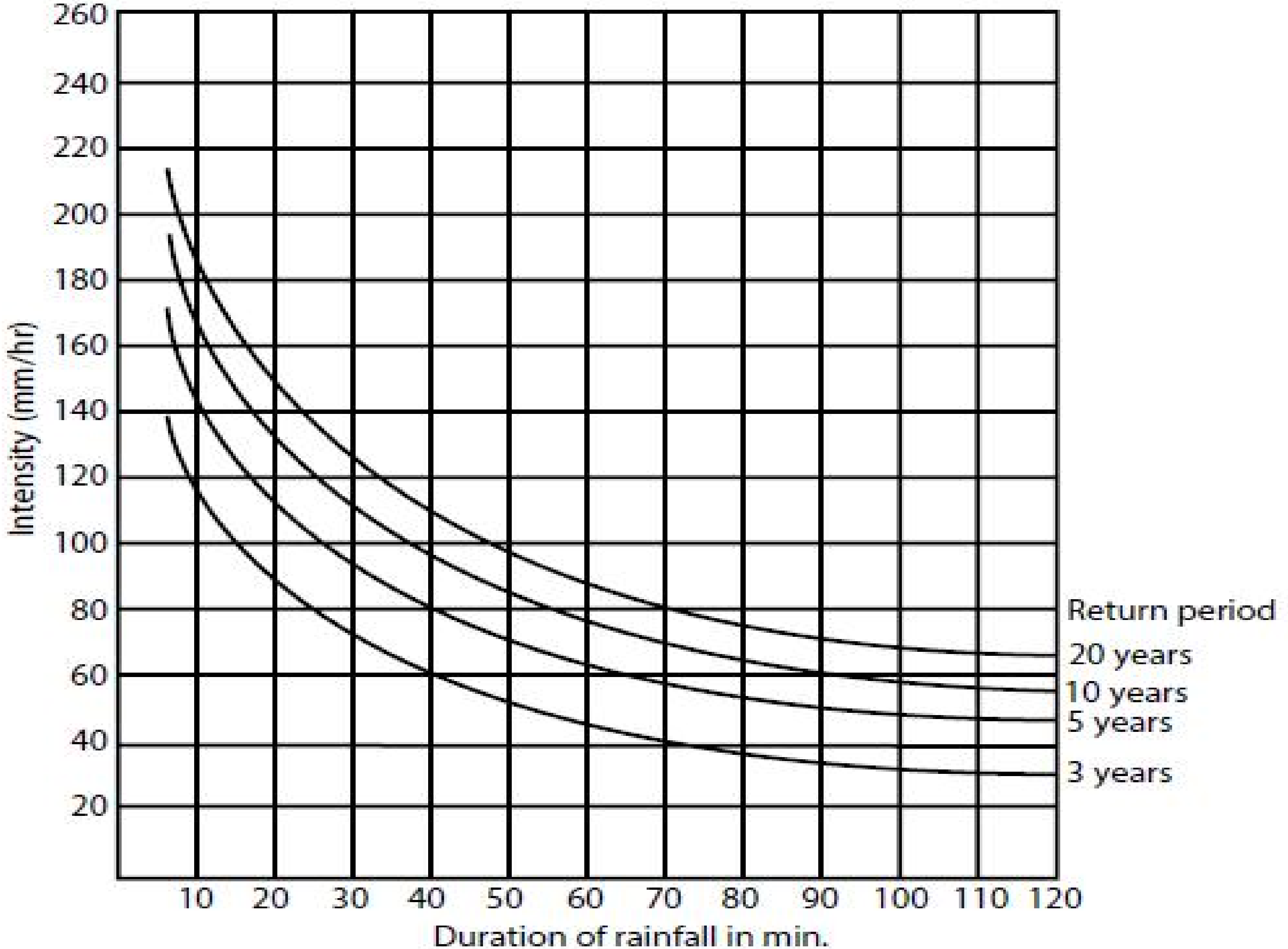


Figure 2.3 Rain fall intensity curve

Figure shows typical rainfall intensity curves for various frequencies of storm. The value of the rainfall intensity (I) can be determined for any given time of concentration (Tc), using these curves. Where rainfall records are not available, the following empirical formula may be used for computing I in mm/Hr. In all formulae I is inversely proportional to t, ie shorter the duration of rainfall grater will be the intensity during that period. In general, the empirical relationship has the following forms:

$$I = a/ (t + b)$$

British Ministry of Health formula

$$I = 760 / (t + 10) \text{ (for storm duration of 5 to 20 minutes)}$$

$$I = 1020 / (t + 10) \text{ (for storm duration of 20 to 100 minutes)}$$

Where, I is intensity of rainfall, mm/h and t is duration of storm, minutes.

The rational formula can be expressed as follows in its generalised form

$$\text{Storm water quantity, } Q = \frac{C.I.A}{360}$$

Where,

Q = Quantity of storm water, m^3/sec

C = Coefficient of runoff

I = intensity of rainfall (mm/hour) for the duration equal to time of concentration, and

A = Drainage area in hectares

OR

$$Q = 0.278 \, C.I.A$$

Where, Q is m^3/sec; I is mm/hour, and A is area in square kilometer

9.8.2 Empirical Formulae

Empirical formulae are used for determination of runoff from very large area. Various empirical relationships are developed based on the past observations on specific site conditions suiting a particular region. These empirical formulae can be used for prediction of storm water runoff for that particular catchment.

a) Burkli – Zeiglar formula

$$Q = \frac{C.I.A}{141.58}\sqrt[4]{\frac{S}{A}}$$

b) Mc Math formula (used in USA)

$$Q = \frac{C.I.A}{148.35} \sqrt[5]{\frac{S}{A}}$$

c) Fuller's formula

$$Q = \frac{CM^{0.8}}{13.23}$$

(Where, S- Slope of the area in meter per thousand meter, M- drainage area in sq. km., A – drainage area in hectare)

Example 9.2

Determine designed discharge for a combined system serving population of 50000 with rate of water supply of 135 LPCD. The catchment area is 100 hectares and the average coefficient of runoff is 0.60. The time of concentration for the design rainfall is 30 min and the relation between intensity of rainfall and duration is I = 1000/(t + 20).

Solution

Estimation of sewage quantity

Considering 80% of the water supplied will result in wastewater generation,

The quantity of sanitary sewage = 50000 x 135 x 0.80

= 5400 m^3/day = 0.0625 m^3/sec

Considering peak factor of 2.5,

The design discharge for sanitary sewage = 0.0625 x 2.5

= 0.156 m^3/sec

Estimation of storm water discharge

Intensity of rainfall, I $= 1000/(t + 20)$

Therefore, I $= 1000/(30 + 20)$

$= 20$ mm/h

Hence, storm water runoff, Q $= \dfrac{C.I.A}{360}$

$= \dfrac{0.6 \times 20 \times 100}{360}$

$= 3.33\ m^3/sec$

$\therefore$ design discharge for combined sewer $= 3.33 + 0.156$

$=$ **3.49** $m^3/sec.$

Example 9.3

The catchment area is of 300 hectares. The surface cover in the catchment can be classified as given below:

Type of cover	Coefficient of runoff	Percentage
Roofs	0.90	15
Pavements and yards	0.80	15
Lawns and gardens	0.15	25
Roads	0.40	20
Open ground	0.10	15
Single family dwelling	0.50	15

Calculate the runoff coefficient and quantity of storm water runoff, if intensity of rainfall is 30 mm/h for rain with duration equal to time of concentration. If population density in the area is 350 persons per hectare and rate of water supply is 200 LPCD, calculate design discharge for separate system, partially separate system, and combined system.

Solution

Estimation of storm water discharge for storm water drain of separate system

$$\text{Overall runoff coefficient } C = \frac{[A1.C1 + A2.C2 + \ldots + An.Cn]}{[A1 + A2 + \ldots + An]}$$

$$= \frac{(0.15 \times 0.90 + 0.15 \times 0.80 + 0.25 \times 0.15 + 0.20 \times 0.4 + 0.15 \times 0.1 + 0.15 \times 0.5)}{0.15 + 0.15 + 0.25 + 0.20 + 0.15 + 0.15}$$

$$= 0.44$$

$$\therefore \text{ quantity of storm water, } Q = \frac{C.I.A}{360}$$

$$= \frac{0.44 \times 30 \times 300}{360}$$

$$= 11 \text{ m}^3/\text{sec}$$

Estimation of sewage discharge for sanitary sewer of separate system

Quantity of sanitary sewage $= 300 \times 350 \times 200 \times 0.80 = 16800\,000$ ltrs/day

$= 16800\ m^3/day$

$= 0.194$ m3/sec

Considering peak factor of 2, the design discharge for sanitary sewers

$= 0.194 \times 2$

$= 0.389\ m^3/sec$

Estimation of discharge for partially separate system

Storm water discharge falling on roofs and paved courtyards will be added to the sanitary sewer.

This quantity can be estimated as:

$$\text{Average coefficient of runoff} = \frac{(0.90 \times 45 + 0.80 \times 45)}{90} = 0.85$$

$$\text{Discharge} = \frac{0.85 \times 30 \times 90}{360}$$

$= 6.375\ m^3/sec$

Therefore total discharge in the sanitary sewer of partially separate system

$= 6.375 + 0.389$

$= 6.764$ m3/sec, and

The discharge in storm water drains $= 11 – 6.375$

$=$ **4.625** m^3/sec

HYDRAULIC DESIGN OF SEWERS

After the determination of the quantity of sewage, the next step is to design the sewer system. In a separate sewage system, which is mostly adopted in modern days, the circular sewer pipes are laid below the ground level, sloping continuously at sufficiently steeper gradients towards the outfall point; and the storm water drains are the separately constructed rectangular or trapezoidal surface drains constructed at suitable gradients, and may be covered or kept open. The sewer pipes are designed to carry the maximum quantity of sanitary sewage likely to be produced from the area contributing to the particular sewer; and the storm water drains are designed to carry the maximum drainage discharge. The combined sewers are designed to carry the sewage as well as the drainage.

TYPES OF SEWERS

Following are types of sewer according to material:

1. Asbestos Cement (AC) sewer
2. Brick sewer
3. Cement concrete sewer
4. Cast iron (CI) sewer
5. Steel sewers
6. Plastic sewers
7. Vertified clay or stoneware sewer
8. High density poly Polyethylene sewer
9. Lead sewer

1. Asbestos Cement (AC) Sewer

Types of sewer like Asbestos Cement (AC) Sewers are manufactured from a mixture of cement and asbestos fiber. Asbestos Cement (AC) Sewers are suitable for carrying domestic sanitary sewage. Asbestos cement sewer is best as vertical pipe for carrying sullage from upper floors of multistory buildings (in two pipe system of plumbing).

Advantages of Asbestos Cement (AC) Sewer

1. Smooth
2. Light in weight
3. Can easily be cut, fitted and drilled
4. Durable against soil corrosion

Disadvantages of Asbestos Cement (AC) Sewer

1. Brittle cannot withstand heavy loads
2. They are easily broken in handling and transport.

2. Brick Sewers

Brick Sewers are made at site and used for construction large size sewer. Brick Sewers are very useful for construction of storm sewer or combined sewer. Nowadays brick sewers are replaced by concrete sewer. Brick sewers my get deformed and leakage may take place. A lot of labor work is required. In order to avoid leakage the brick sewer should be plastered.

3. Plain or reinforced cement Concrete

Cement concrete pipes can either be plain or reinforced. Plain cement concrete pipes are used from 80 mm to 450 mm, with their thickness varying from 25 to 35 mm. For bigger diameters, these are reinforced, consisting of longitudinal and transverse reinforcement in the form of rings or elliptical cage. They may be cast in situ or precast, resistant to heavy loads, corrosion and high pressure. These are very heavy and difficult to transport.

Advantages of concrete pipes

1. Strong in tension as well as compression.
2. Resistant to erosion and abrasion.
3. They can be made of any desired strength.
4. Easily molded, and can be in situ or precast pipes.
5. Economical for medium and large sizes.
6. These pipes are available in wide range of size and the trench can be opened.
7. Back filled rapidly during maintenance of sewers

Disadvantages

These pipes can get corroded and pitted by the action of H_2SO_4.

The carrying capacity of the pipe reduces with time because of corrosion.

The pipes are susceptible to erosion by sewage containing silt and grit.

4. Cast Iron (CI) Sewers

These types of sewer are high strength, durable and water tight. Cast Iron sewers can withstand high internal pressure and can bear external load. Cast Iron sewers are suitable for the following conditions;

1. When the sewage is conveyed under high pressure
2. When the sewer line is subject to heavy external load e.g. under railway line, foundation wall etc, below highways
3. When there is considerable difference in temperature.

5. Steel Sewers

steel sewers are Impervious; light, resistant to high pressure, flexible, suitable when;

1. The sewage is carried under pressure.
2. The sewage has to be carried across a river under water.
3. The sewer has to cross under a railway track.
4. They are generally used for outfall and trunk sewers.

6. Plastic Sewers

Now a days PVC sewers are used for carrying sewage. Plastic sewers are resistant to corrosion. Such types of sewer are light in weight, smooth and can be bent easily. But Plastic sewers are having high co-efficient of thermal expansion and cannot be used in very hot areas.

7. Vitrified Clay or Stoneware Sewers

These pipes are used for house connections as well as lateral sewers. The size of the pipe available is 5 cm to 30 cm internal diameter with length 0.9 to 1.2 m. These pipes are rarely manufactured for diameter greater than 90 cm. These are joined by bell and spigot flexible compression joints.

Advantages

1. Resistant to corrosion, hence fit for carrying polluted water such as sewage.
2. Interior surface is smooth and is hydraulically efficient.
3. The pipes are highly impervious.
4. Strong in compression.
5. These pipes are durable and economical for small diameters.
6. The pipe material does not absorb water more than 5% of their own weight, when immersed in water for 24 hrs.

Disadvantages

1. Heavy, bulky and brittle and hence, difficult to transport.
2. These pipes cannot be used as pressure pipes, because they are weak in tension.
3. These require large number of joints as the individual pipe length is small.

8. High Density Polythylene (HDPE) Pipes

Use of these pipes for sewers is recent development. They are not brittle like AC pipes and other pipes and hence hard fall during loading, unloading and handling do not cause any damage to the pipes. They can be joined by welding or can be jointed with detachable joints up to 630 mm diameter (IS:4984-1987). These are commonly used for conveyance of industrial wastewater. They offer all the advantages offered by PVC pipes. PVC pipes offer very little flexibility and normally considered rigid; whereas, HDPE pipes are flexible hence best suited for lying in hilly and uneven terrain. Flexibility allows simple handling and installation of HDPE pipes.

8. High Density Polythylene (HDPE) Pipes

Because of low density, these pipes are very light in weight. Due to light in weight, they are easy for handling, this reduces transportation and installation cost. HDPE pipes are non corrosive and offer very smooth inside surface due to which pressure losses are minimal and also this material resist scale formation.

9. Lead Sewers

- They are smooth, soft and can take odd shapes.
- This pipe has an ability to resist sulphide corrosion.
- However, these pipes are very costly.
- These are used in house connection

10.3 SHAPES OF SEWER PIPES

Sewers are generally circular pipes laid below ground level, slopping continuously towards the outfall. These are designed to flow under gravity. Shapes other than circular are also used. Other shapes used for sewers are (Figure 10.1 a through i):

a. Standard Egg-shaped sewer

b. New egg-shaped sewer

c. Horse shoe shaped sewer

d. Parabolic shaped sewer

e. Semi-elliptical section

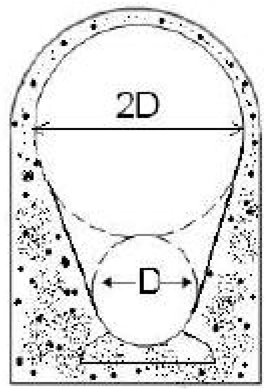
f. Rectangular shape section

g. U-shaped section

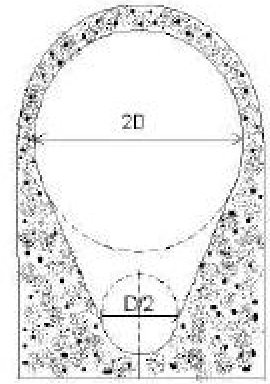
h. Semi-circular shaped sewer

i. Basket handled shape sewer

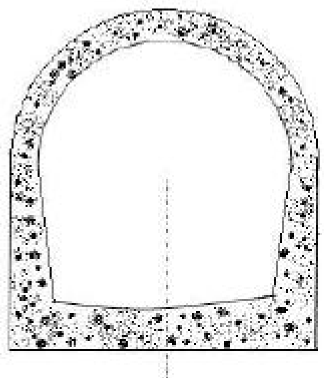
Standard egg-shaped sewers, also called as ovoid shaped sewer, and new or modified egg-shaped sewers are used in combined sewers. These sewers can generate self cleansing velocity during dry weather flow. Horse shoe shaped sewers and semi-circular sections are used for large sewers with heavy discharge such as trunk and outfall sewers. Rectangular or trapezoidal section is used for conveying storm water. U-shaped section is used for larger sewers and especially in open cuts. Other sections of the sewers have become absolute due to difficulty in construction on site and non availability of these shapes readily in market.



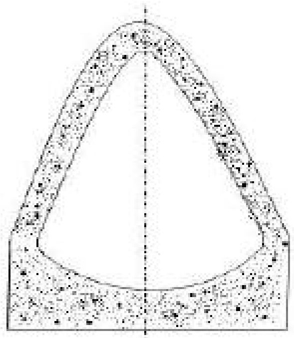
(a) Standard Egg Shaped Sewer



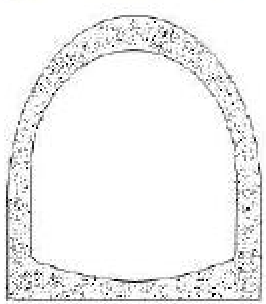
(b) New/ Modified Egg shaped Sewe



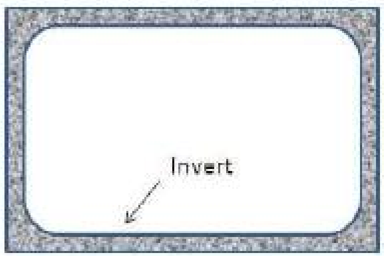
(c) Horse shoe sewer section



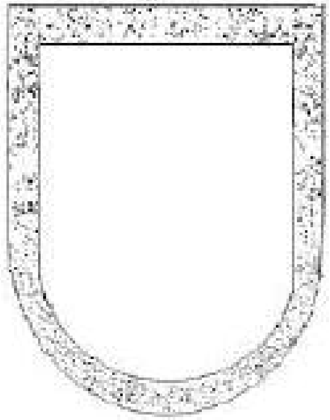
(d) Parabolic section



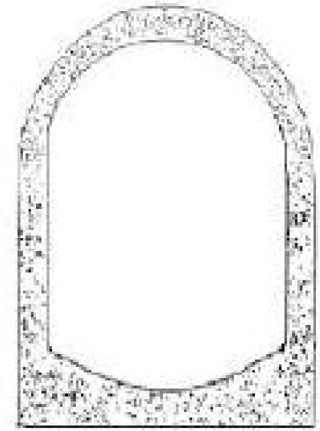
(e) Semi-elliptical section



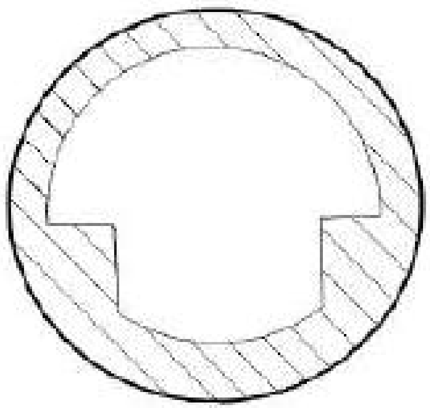
(f) Rectangular Sewer



(g) U-shaped section



(h) Semi-circular Section



(i) Basket-Handle Section

Figure 10.1 Different shapes used for constructionof sewer other than circular shape

10.4 FORMULAS FOR DETERMINING FLOW VELOCITIES IN SEWERS AND DRAINS

The sewers and drains are generally designed as open channels except when it is especially required to design them as flowing under pressure as in the case of inverted siphons; and discharge lines from sewage pumping stations, which always flow under pressure. Various empirical formulas, which have been suggested for determining the gradients necessary to obtain design velocities of flow in sewers, are given below.

1) Chezy's formula

$$V = C\sqrt{R \times S}$$

Where,

V = Velocity of flow in the channel in m/sec.

$$R = \text{Hydraulic mean depth of channel} = \frac{A}{P} = \frac{D}{4}$$

(For circular sewer running full)

A= Area of channel
P = wetted perimeter
S = hydraulic gradient
C = Chezy's constant

The Chezy's constant C depends up on various factors such as size, shape, roughness etc. of the channel. The value of C can be obtained by using either the Kutter's formula or the Bazin's formula. By knowing the velocity of flow V, the channel section can be designed from the equation:

$$Q = A \times V$$

a) Kutter's formula for constant C

$$C = \frac{\left(23 + \frac{0.00155}{S}\right) + \frac{1}{N}}{1 + \left(23 + \frac{0.00155}{S}\right)\frac{N}{\sqrt{R}}}$$

Where,

R = Hydrsulic mean radius = $\frac{A}{P}$

S = Slope

N= Rugosity coefficient, the value depends upon the nature of inside surface of the sewer.

Various values of N are given below.

Table 10.1 Values of Rugosity Coefficient 'N'

Sl. No	Pipe material	Values of N	
		Good interior surface condition	Fair interior surface condition
1	Salt glazed stone ware pipes	0.012	0.014
2	Cement concrete pipes	0.013	0.015
3	Castiron pipes	0.012	0.013
4	Brick, unglazed sewer/drains	0.013	0.015
5	Asbestos cement pipes	0.011	0.011
6	Plastic pipes (Smooth)	0.011	0.011

b) Bazin's formula for constant C

$$C = \frac{157.6\,K}{1.81 + \frac{K}{\sqrt{R}}}$$

Where, K = Bazin's constant as given below

Table 10.2 Bazin's constant 'K'

Sl. No	Type or nature of inside surface of sewer or drain	Values of K
1	Very smooth surface	0.11
2	Smooth brick and concrete surface	0.29
3	Rough brick and concrete surfaces	0.50
4	Smooth rubble and masonarysurface	0.83
5	Good earthen channels	1.54
6	Rough earthen channels	3.17

2. Mannings formula

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

Where N = Rugosity coefficient and it is same as were given by Kutter.

3. Hazen – William's formula

$$V = 0.85\ C\ R^{0.65}\ S^{0.54}$$

The value of C is given below.

Table 10.3 Hazen andwilliam's coefficient 'C'

Sl. No	Type of pipe material	Values of C	
		New pipes	Design purpose
1	Concrete RCC pipes	140	110
2	Cast iron pipes	130	100
3	Galvanised iron pipes	120	100
4	Steel pipe with welded joints	140	100
5	Steel pipe with rivetted joints	110	95
6	Steel pipe with welded joints lined with cement for bituminous enamel	140	110
7	Asbestos cement pipes	150	120
8	Plastic pipes	150	120

10.5 MAXIMUM AND MINIMUM VELOCITIES TO BE GENERATED IN SEWERS

Flow velocities in the sewers should be such that neither the suspended materials in sewage get silted up nor the sewage pipe material gets scoured out.

Minimum velocities

The silting of sewers can be avoided by generating such high velocities that would not permit the solids to settle down; ie. the velocity should be such as to cause automatic self-cleansing effect. The generation of such a minimum self-cleansing velocity in the sewer, atleast, once day is important. Self cleansing velocity for various types of suspended solids present in the sewage and various diameter of the sewer pipes are given in the tables given below:

Table 10.4 Self-cleansing velocities

Nature of the solid particles present in the sewage	Self-clinsing velocity (m/sec.)
Angular stones	1.0
Round pebbles	0.5 to 0.60
Fine gravel	0.3
Course sand	0.2
Fine sand and clay	0.15
Fine clay and silt	0.075

Table 10.5 Self-cleansing velocities of various diameters of the pipes

Diameter of the sewer	Self-clinsing velocity (m/sec.)
15 to 25 cm	1.0
30 to 60 cm	0.75
Above 60 cm	0.6

Shield's expression for self-cleansing velocity

Self cleansing velocity, which is necessary to cause scouring and suspension of solid particles (heavier than water) can be determined by Shield's formula.

$$V = \sqrt{\frac{8K}{f}\left(\frac{\delta_s - \delta}{\delta}\right) g.\, ds}$$

Where,

V = Velocity of flow

f = Darcy's coefficient of friction (0.03)

δ_s = Specific gravity of the solids flowing in the sewage (1.2 to 2.65)

δ = Specific gravity of the liquid

g = Accelaration due to gravity

ds = diameter of the solids to be carried by the liqid in m.

Maximum velocities

The smooth interior surface of a sewer pipe gets scoured due to the continuous abrasion caused by the suspended solids present in sewage. This scouring, wear and tear of the pipe interior is much more pronounced at velocities higher than what can be tolerated by the pipe materials. In order to avoid these complications, it is there fore, necessary to limit the maximum velocity that will be produced in the sewer pipe at any time. The non-scouring velocity will mainly depend upon the material of the sewer, and its values are given in table shown below

Table 10.6 Non scouring velocities of various diameters of the pipes

Material of sewer	Non scoring velocity (m/sec.)
Earthern channels	0.6 to 1.2
Ordinary brick lined sewer	1.5 to 2.5
Cement concrete sewer	2.5 to 3.0
Stone ware sewer	3 to 4.5
Cast iron sewer pipes	3.5 to 4.5
Vertified tiles and glased bricks	4.5 to 5.0

EFFECT OF VARIATIONS OF DISCHARGE ON VELOCITY IN SEWERS

The sewage discharge flowing through a sewer does not remain constant all the time, but varies considerably from time to time due to this variation in discharge, the depth of flow varies and hence the hydraulic mean depth (HMD) varies. Due to the change in hydraulic mean depth, the flow velocity affected from time to time.

Since the velocity developed in a sewer of a given section and grade will be less as and when the flow reduces, it is necessary to check the sewer for maintaining a minimum velocity of about 0.45 m/sec. at the time of minimum flow. The designer should also ensure that a velocity of about 0.9 m/sec. is developed at least at the time of maximum flow and preferably during the average flow periods also.

On the other hand, if the available ground slopes are neither too steep nor too flat, the condition of developing velocities about 0.9 m/sec. at average flow may be practically possible and economical and hence may be insisted.

10.7 HYDRAULIC ELEMENTS OF CIRCULAR SEWERS

Sewers of circular section are more commonly used. How ever some times egg shaped or horse shoe shaped or rectangular shape are also be used.

The circular sewers may some times runfull or may run partially full. Both the cases their hydraulic properties are given below.

i) When runfull

Area of cross section, $A = \frac{\pi}{4} D^2$

Where, D = Dia of the pipe\

Wetted perimeter, $P = \pi D$

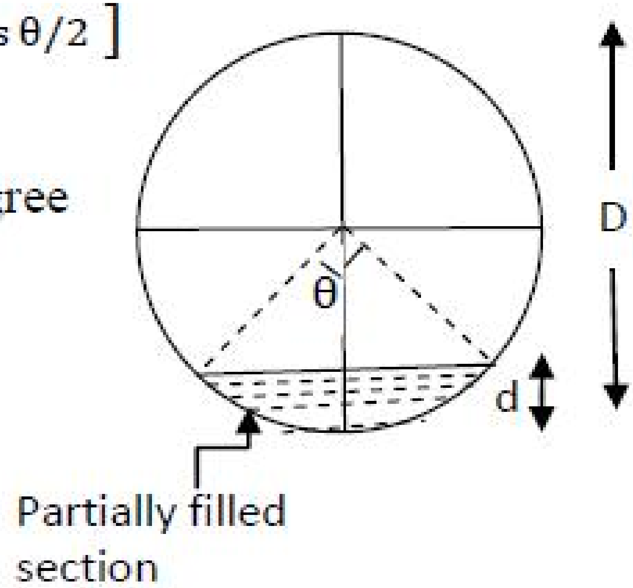
Hydraulic mean depth $R = \frac{A}{P} = \frac{\frac{\pi D^2}{4}}{\pi D} = \frac{D}{4}$

i) When the sewer partially full

$$\text{The depth at partial flow d } = \left[\frac{D}{2} - \frac{D}{2}\cos\theta/2\right]$$

$$= \frac{D}{2}\left[1 - \cos\theta/2\right]$$

Where, θ = Cental angle in degree



$$\therefore \text{Proportanate depth} = \frac{d}{D}$$

$$= \frac{1}{2}\left[1 - \cos\theta/2\right]$$

Area of c/s while running partially full

$$a = \frac{\pi D^2}{4}\frac{\theta}{360} - \frac{D}{2}\cos\frac{\theta}{2} \cdot \frac{D}{2}\sin\frac{\theta}{2}$$

$$= \frac{\pi D^2}{4}\left[\frac{\theta}{360} - \frac{\sin\theta}{2\pi}\right]$$

[Since $\sin\theta - 2\sin\theta/2.\cos\theta/2$]

$$\therefore \text{Proportionate area} = \frac{a}{A} = \left[\frac{\theta}{360} - \frac{\sin\theta}{2\pi}\right]$$

Wettted permeter $p = \pi D \frac{\theta}{360}$

$$\therefore \text{Proportionate perimeter} = \frac{p}{P} = \frac{\pi D \frac{\theta}{360}}{\pi D} = \frac{\theta}{360}$$

$$\text{Hydraulic mean depth } r = \frac{a}{p}$$

$$= \frac{D}{4}\left[1 - \frac{360^\circ \sin\theta}{2\pi\theta}\right]$$

$$\therefore \text{Proportionate HMD} = \frac{r}{R}$$

$$= \left[1 - \frac{360^\circ \sin\theta}{2\pi\theta}\right]$$

Velocity of flow

Velocity of flow is given by Manning's formul as

υ = Velocity at partial flow

$$\therefore \upsilon = \frac{1}{n} r^{2/3} \sqrt{S_o}$$

(where $S = S_o$ = bed slope)

V = velocity, when running full

$$\therefore V = \frac{1}{N} R^{2/3} \sqrt{S_o}$$

(Bed slope So = S remaining same whether pipe run full or partally full)

$$\therefore \text{Proportionate velocity} = \frac{\upsilon}{V} = \frac{\frac{1}{n} r^{2/3} \sqrt{S_o}}{\frac{1}{N} R^{2/3} \sqrt{S_o}}$$

$$= \frac{N}{n} \frac{r^{2/3}}{R^{2/3}}$$

Assuming that roughnes coefficient N is not vary with depth, where, n = N

$\therefore$ Proportionate velocity $= \frac{r^{2/3}}{R^{2/3}}$

$$= \left[1 - \frac{360^o \ \mathrm{Sin}\theta}{2\pi\theta}\right]^{2/3}$$

$\therefore$ Discharge when pipe is running partially full,

$$q = a\ \upsilon$$

Discharge when pipe is running full

$$Q = A\ V$$

$\therefore$ Proportionate Discharge

$$= \frac{Q}{q} = \frac{a\upsilon}{AV} = \frac{a}{A} \frac{\upsilon}{V}$$

$$= \left[\frac{\theta}{360} - \frac{\mathrm{Sin}\ \theta}{2\pi}\right] \mathrm{X} \left[1 - \frac{360^o \ \mathrm{Sin}\theta}{2\pi\theta}\right]^{2/3}$$

In all the above derived equations, except θ, every thing is constant, and hence by giving values of θ, all the six proportionate elements can be easily calculated.

By taking proportionate depth (d/D) as reference, values of other elements can be found out from the ready made computed values, shown in table 10.7.

Table 10.7 Proportionate values of hydraulic elements for circular sewers when flowing partially full (Without corrected for variations of roughness with depth)

Proportionate Depth d/D (1)	*Proportionate Area a/A (2)*	*Proportionate Wetted perimeter p/P (3)*	*Proportionate HMD r/R (4)*	*Proportionate Velocity v/V (5)*	*Proportionate Discharge q/Q (6)*
1.00	1.00	1.000	1.000	1.000	1.000
0.90	0.949	0.857	1.192	1.124	1.066
0.80	0.858	0.705	1.217	1.140	0.988
0.70	0.748	0.631	1.185	1.120	0.838
0.60	0.626	0.564	1.110	1.072	0.671
0.50	0.500	0.500	1.000	1.000	0.500
0.40	0.373	0.444	0.857	0.902	0.337
0.30	0.252	0.369	0.684	0.776	0.196
0.20	0.143	0.296	0.482	0.615	0.088
0.10	0.052	0.205	0.254	0.401	0.021
0.00	0.000	0.000	0.000	0.000	0.000

Table 10.8 Hydraulic particulars of circular sewers accounting for variations of roughness with depth

Proportionate Depth d/D (1)	*Proportionate roughness n/N* (2)	*Proportionate Velocity v/V* (3)	*Proportionate Discharge q/Q* (4)
1.0	1.00	1.000	1.000
0.9	1.07	1.056	1.020
0.8	1.14	1.003	0.890
0.7	1.18	0.952	0.712
0.6	1.21	0.890	0.557
0.5	1.24	0.810	0.405
0.4	1.27	0.713	0.266
0.3	1.28	0.605	0.153
0.2	1.27	0.486	0.070
0.1	1.22	0.329	0.017

From the table, it can be seen the velocities in partially filled circular sections, equal or exceed those in full sections. Maximum velocity is obtained not when the sewer is running full but when the depth of flow is 0.81 times the full depth and is about 12.5 % grater that when running full.

Similarly the maximum discharge is obtained not when the sewer is running full, but when the depth is about 0.95 times the full depth, and is about 7% grater than that when running full.

The above statements are correct only; the roughness (n) is independent of depth. Sundin has demonstrated that the value of n is not constant, but varies more than 20% as shown in the table 10.8.

The effect of the variation of n reduce the proportionate velocities and discharge at lower depths of flow, because roughness (n) increases with lower depths. If these variations of n are also considered, more precise values of proportionate velocities and discharges can be computed out as shown in table 10.8

These precise proportionate velocities and discharges have been plotted by firm lines given in figure 10.2, to obtain a standard chart, which is very useful in obtaining different elements, by knowing any one of them. Values of table 10.7 are also plotted by doted curves as to obtain proportionate elements when variations of n are plotted. This plotted diagram is known as partial flow diagram.

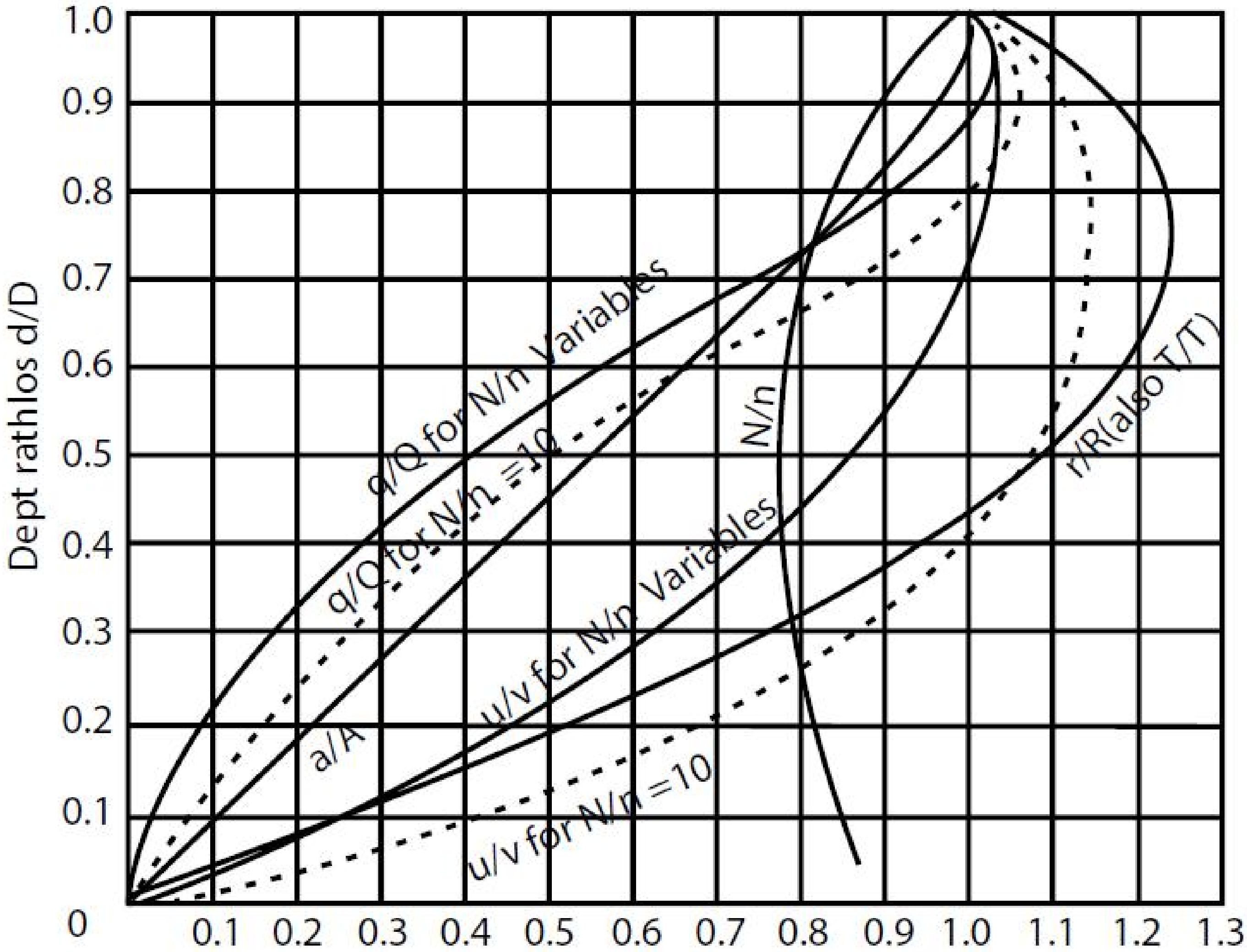


Figure 3.2 Standard chart for proportionate elements for circular sewer

Sewers flowing with depth between 50% and 80% full need not be placed on steeper gradients to be as self-cleansing as sewers flowing full. The self clinsing velocity can be computed with the help of equation;

$$\upsilon_s = \frac{N}{n} \left(\frac{r}{R}\right)^{1/6} \mathrm{V}$$

Problem 10.1

Design a sewer for a discharge of 800 ltrs/Sec. running half full. Assume s = 0.0001.

Solution:

It is given q $= 800$ ltrs/Sec.

$= 0.8\ m^3/Sec$

Since it is running half full $\frac{d}{D} = 1/2$

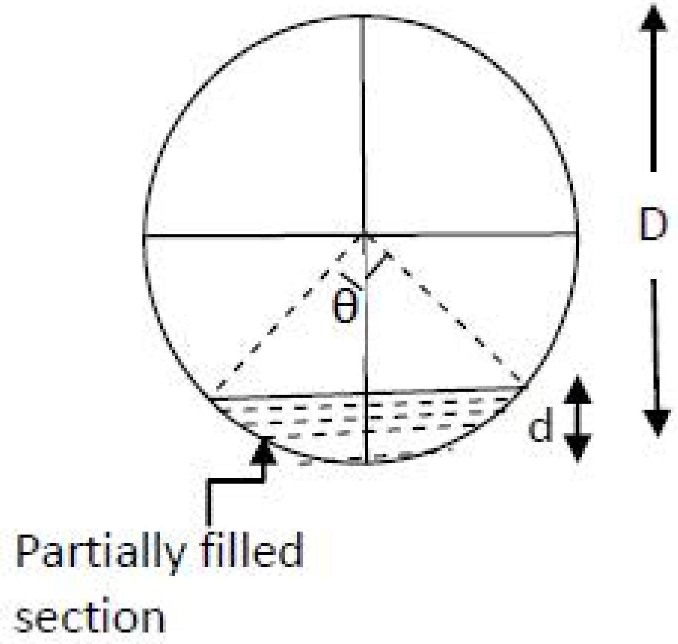
$$\therefore \cos \theta/2 = \frac{\left(\frac{D}{2} - d\right)}{D/2}$$

$$= 2\left[\frac{D}{2} - \frac{d}{D}\right]$$

$$= 1 - \frac{2d}{D} = 0$$

$$\therefore \frac{\theta}{2} = 90°$$

$$\therefore \theta = 180° \text{ and } \sin \theta = 0$$



We know

$$\text{Area, } a = \frac{\pi D^2}{4}\left[\frac{\theta}{360} - \frac{\text{Sin }\theta}{2\pi}\right]$$

$$= \frac{\pi D^2}{4}\left[\frac{180}{360}\right] = \frac{\pi D^2}{8}$$

$$\text{Perimeter, } p = \pi D \frac{\theta}{360} = \pi D \frac{180}{360} = \frac{\pi D}{2}$$

$$\text{Hydraulic mean depth } r = \frac{a}{P}$$

$$= \frac{\frac{\pi D^2}{8}}{\frac{\pi D}{2}} = \frac{D}{4}$$

$$\text{Velocity, } \upsilon = \frac{1}{n} r^{2/3} \sqrt{S_o}$$

$$\therefore \upsilon = \frac{1}{n} \left(\frac{D}{4}\right)^{2/3} \sqrt{0.0001}$$

We know, q= a x v and assume n= 0.012

$$\text{ie, } 0.8 = \frac{\pi D^2}{8} \times \frac{1}{0.012} \left(\frac{D}{4}\right)^{2/3} \sqrt{0.0001}$$

D = 1.98 mtr.
Say 200 cm

Problem 10.2

Design a sewer for half flow condition. Population = 50000, water supplied 150 lpcd. Maximum flow 5 times the DWF, slope 1 in 625, mannings coefficient n = 0.013.

Solution:

Population = 50000

Rate of water supply = 150 lpcd

∴ Total quantity of water supplied = 50000 x 150

$= 7.5 \times 10^6$ ltrs/day

Assume average discharge of sewage is 80% of the water supplied.

$\therefore$ Average discharge of sewage $= 0.80 \times 7.5 \times 10^6$ ltrs/day

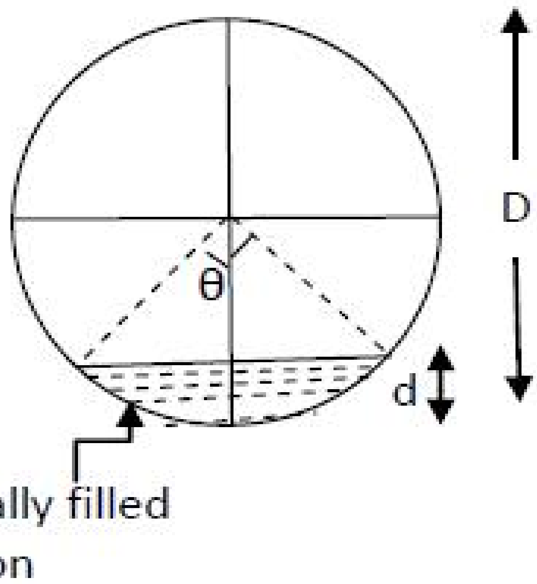
$= 6 \times 10^6$ ltrs/day

Maximum discharge $= 5 \times DWF$

$= 5 \times 6 \times 10^6$

$= 30 \times 10^6$ ltrs/day

$\mathbf{= 30 \times 10^3\ m^3/day}$



Since it is running half full $\frac{d}{D} = 1/2$

$$\therefore \cos \theta/2 = \frac{(\frac{D}{2} - d)}{D/2}$$

$$= 2[\frac{D}{2} - \frac{d}{D}]$$

$$= 1 - \frac{2d}{D} = 0$$

$$\therefore \frac{\theta}{2} = 90°$$

$$\therefore \theta = 180° \text{ and } \sin\theta = 0$$

We know

$$\text{Area, } a = \frac{\pi D^2}{4}\left[\frac{\theta}{360} - \frac{\sin\theta}{2\pi}\right]$$

$$= \frac{\pi D^2}{4}\left[\frac{180}{360}\right] = \frac{\pi D^2}{8}$$

$$\text{Perimeter, } p = \pi D \frac{\theta}{360} = \pi D \frac{180}{360} = \frac{\pi D}{2}$$

$$\text{Hydraulic mean depth } r = \frac{a}{P}$$

$$= \frac{\frac{\pi D^2}{8}}{\frac{\pi D}{2}} = \frac{D}{4}$$

$$\text{Velocity, } \upsilon = \frac{1}{n} r^{2/3} \sqrt{S_o}$$

$$\therefore \upsilon = \frac{1}{n}\left(\frac{D}{4}\right)^{\frac{2}{3}} \left(\frac{1}{625}\right)^{1/2}$$

$$= \frac{1}{n}\left(\frac{D}{4}\right)^{\frac{2}{3}} \text{ X } 0.04$$

We know, $q = a \times V$ and $n = 0.013$

$$\text{ie, } \frac{30 \times 10^3}{24\ X\ 60 X\ 60} = \frac{\pi D^2}{8} \text{ X } \frac{1}{0.013} \left(\frac{D}{4}\right)^{2/3} \text{x } 0.04$$

$$D^{8/3} = 0.73$$

D = 0.88 mtr.

Say 90 cm

Check for self cleaning velocity at maximum discharge.

$$r = \frac{D}{4} = \frac{0.9}{4} = 0.225$$

$$\therefore \upsilon = \frac{1}{0.013}(0.225)^{\frac{2}{3}}\left(\frac{1}{625}\right)^{1/2}$$

$$= 1.14 \text{ m/sec}$$

This is much more than the self clinsing velocity of 0.9 m/sec.

Problem 10.3

A 40 cm diameter sewer is to flow at 0.4 depth on a grade ensuring a degree of self- cleansing equivalent to that obtained at a velocity of 80 cm/sec. find

(i) The required grade

(ii) Associted velocity

(iii) The rate of discharge at this depth

Given

(i) Manning's rugocity coefficient = 0.014

(ii) Prportionate area = 0.252

(iii) Prportionate HMD = 0.684

Solution:

(i)Finding the gradient

It is given V = 0.8 m/sec, D = 0.40 mtr and N = 0.014

At 0.4 depth $\frac{d}{D} = 0.4, \ \frac{a}{A} = 0.252, \ \frac{r}{R} = 0.684$

At full depth,

$$\text{Velocity } \upsilon = \frac{1}{n} r^{2/3} \sqrt{S_o}$$

$$\therefore 0.8 = \frac{1}{0.014} \left(\frac{0.4}{4}\right)^{\frac{2}{3}} \sqrt{S_o}$$

$$\therefore S_o = 0.002704 = \frac{1}{369.822}$$

Now, for a sewer to bee the same self-clinsing at 0.4 depth as it will be at full depth, we have the gradient (Ss) required from the following equation.

$$Ss.\ R = R\ .\ S$$

$$\therefore Ss = \left(\frac{R}{r}\right) \text{x}\ S$$

$$= \left(\frac{1}{0.684}\right) \text{X}\ 0.002704$$

$= 0.00395$

$= \frac{\mathbf{1}}{\mathbf{252.959}}$

Hence the required gradient is 1 in 253

(ii)Finding the velocity

The velocity generated at this gradient at 0.4 depth is given by the equation.

$$\upsilon_s \quad = \frac{N}{n} \left(\frac{r}{R}\right)^{1/6} \mathrm{V}$$

$$= 1 \times (0.684)^{1/6} \times 0.8$$

$$\boldsymbol{\upsilon_s} \quad = \mathbf{0.7512\ m/sec}$$

(ii)Finding the discharge

The discharge $q_s = a \times \upsilon_s$

$$= 0.252 \times \frac{\pi(0.4)^2}{4} \times 0.7512$$

$$= \mathbf{0.0238\ m^3/sec.}$$

Where, $\frac{a}{A} = 0.252$

$\therefore a = 0.252 \times \frac{\pi D^2}{4}$

SEWER APPURTENANCES

In order to make the process of construction easy and have efficient working and maintenance, the sewer system requires various other additional structures. So the structures, which are constructed at suitable intervals along the sewerage system to help its efficient operation and maintenance, are called as sewer appurtenances. Following are the important sewer appurtenance:

SEWER APPURTENANCES

Following are the important sewer appurtenance:

(1) Manholes,

(2) Drop manholes,

(3) Lamp holes,

(4) Clean-outs,

(5) Street inlets called Gullies,

(6) Catch basins,

(7) Flushing Tanks,

(8) Grease & Oil traps,

(9) Inverted Siphons, and

(10) Storm Regulators.

MANHOLES

The manhole is masonry or R.C.C. chamber constructed at suitable intervals along the sewer lines, for providing access into them. Thus, the manhole helps in inspection, cleaning and maintenance of sewer. These are provided at every bend, junction, change of gradient or change of diameter of the sewer. The sewer line between the two manholes is laid straight with even gradient. For straight sewer line manholes are provided at regular interval depending upon the diameter of the sewer.

MANHOLES

The spacing of manhole is recommended in IS 1742-1960. For sewer up to 0.3 m diameter or sewers which cannot be entered for cleaning or inspection the maximum spacing between the manholes recommended is 30 m, and 300 m spacing for pipe greater than 2.0 m diameter. The minimum width of the manhole should not be less than internal diameter of the sewer pipe plus 150 mm benching on both the sides.

MANHOLES

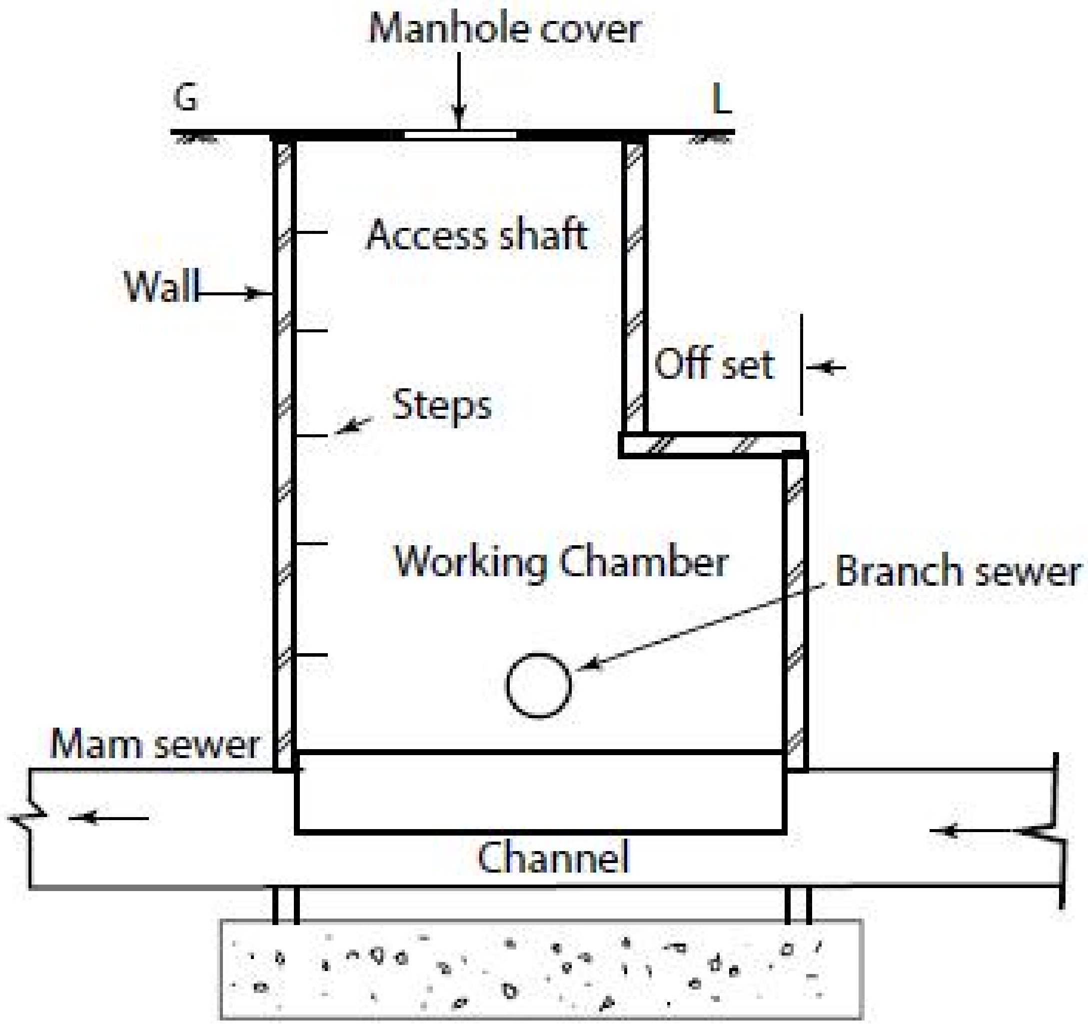


Figure 5.1Manhole

MANHOLES

A manhole typically consists of following component parts:

- Access shaft
- Working chamber
- Bottom or invert
- Side walls
- Steps or ladder
- Top cover

DROP MANHOLE

A drop manhole is a special type of manhole on a sewer line which is constructed to provide a connection between a high level branch sewer to a low level main sewer. When a branch sewer enters a manhole by more than 0.5 to 0.6 mtr above the main sewer, the sewage is not allowed to fall directly in to the manhole, to avoid the possibilities of sewage being thrown on person entering the working chamber of manhole

DROP MANHOLE

Instead, the sewage of the branch sewer is brought into the manhole of the main sewer through a down pipe. The construction of drop manhole avoids unnecessary steep gradient of branch sewer thus reducing the amount of earth work.

DROP MANHOLE

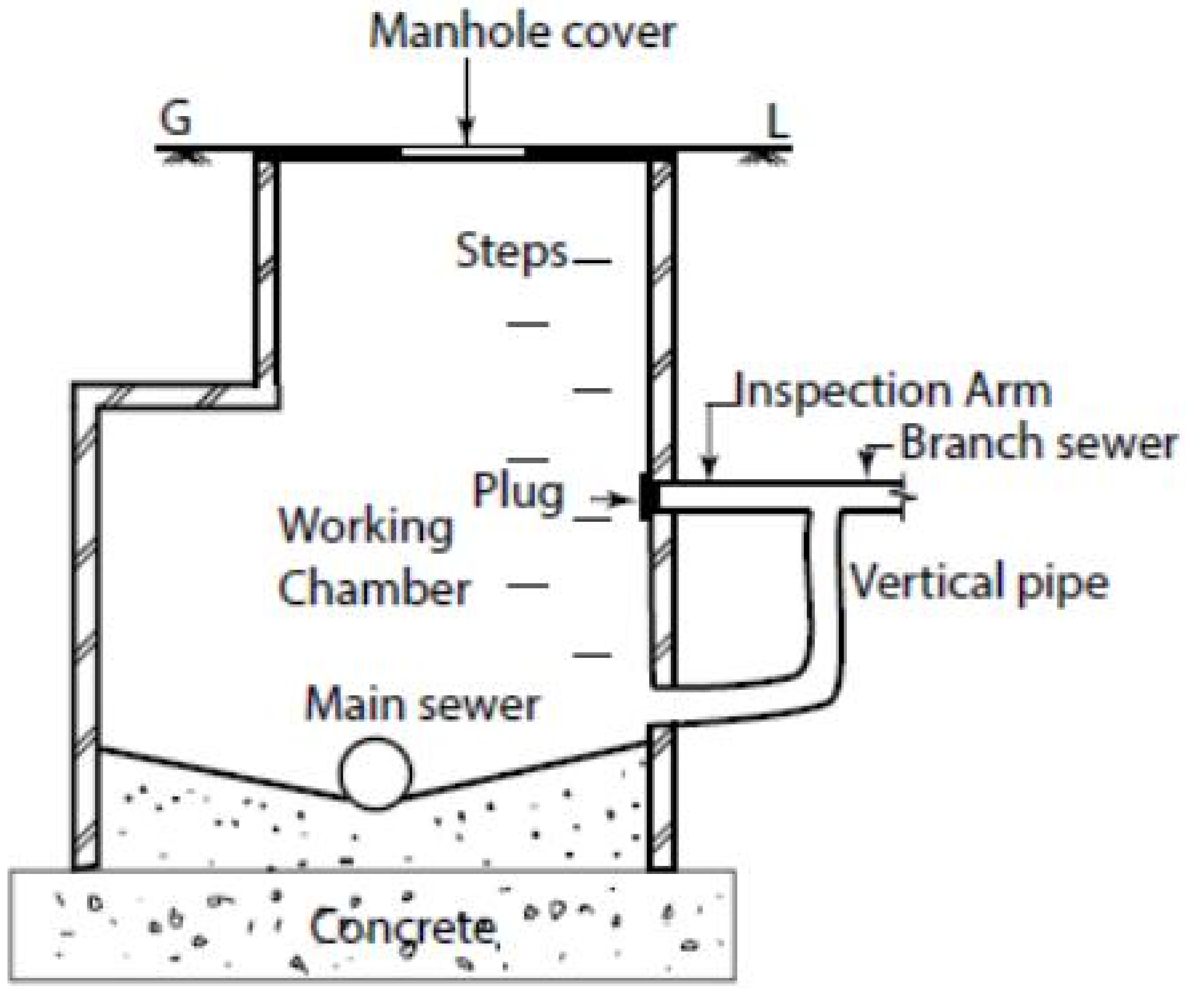


Figure 5.2 Drop manhole

DROP MANHOLE

The branch sewer is joined to the manhole of main sewer through a vertical pipe. The sewage coming from the branch sewer trickles down in the vertical pipe and emerges out through a horizontal pipe just above the benching. A plug is provided at the point where the branch sewer, if prolonged straight, intersects with the vertical wall of the manhole. The prolonged length of branch sewer, beyond the vertical pipe, serves as the inspection arm; after opening the plug, it can be used for inspecting or cleaning the branch sewer.

LAMP HOLES

It is an opening or hole constructed in a sewer for purpose of lowering a lamp inside it. It consists of stoneware or concrete pipe, which is connected to sewer line through a T-junction as shown in the Figure. The pipe is covered with concrete to make it stable. Manhole cover of sufficient strength is provided at ground level to take the load of traffic.

LAMP HOLES

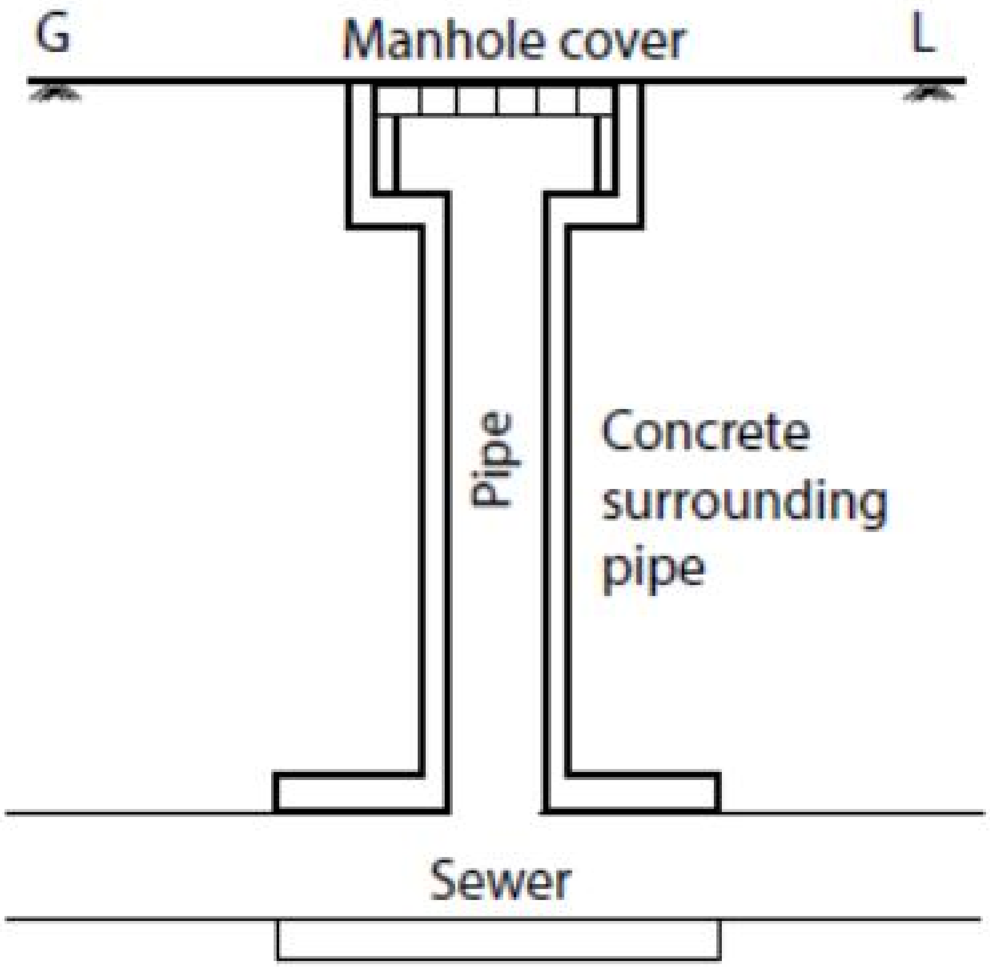


Figure 5.3 Lamp hole

LAMP HOLES

An electric lamp is inserted in the lamp hole and the light of lamp is observed from manholes. If the sewer length is unobstructed, the light of lamp will be seen. It is constructed when construction of manhole is difficult in that location. This lamp hole can also be used for flushing the sewers. If the top cover is perforated it will also help in ventilating the sewer, such lamp hole is known as fresh air inlet. In present practice as far as possible the use of lamp hole is avoided.

CLEAN-OUTS

It is a pipe which is connected to the underground sewer. The other end of the clean-out pipe is brought up to ground level and a cover is placed at ground level (Figure 3.4). A clean-out is generally provided at the upper end of lateral sewers in place of manholes. During blockage of pipe, the cover is taken out and water is forced through the clean-out pipe to lateral sewers to remove obstacles in the sewer line. For large obstacles, flexible rod may be inserted through the clean-out pipe and moved forward and backward to remove such obstacle.

CLEAN-OUTS

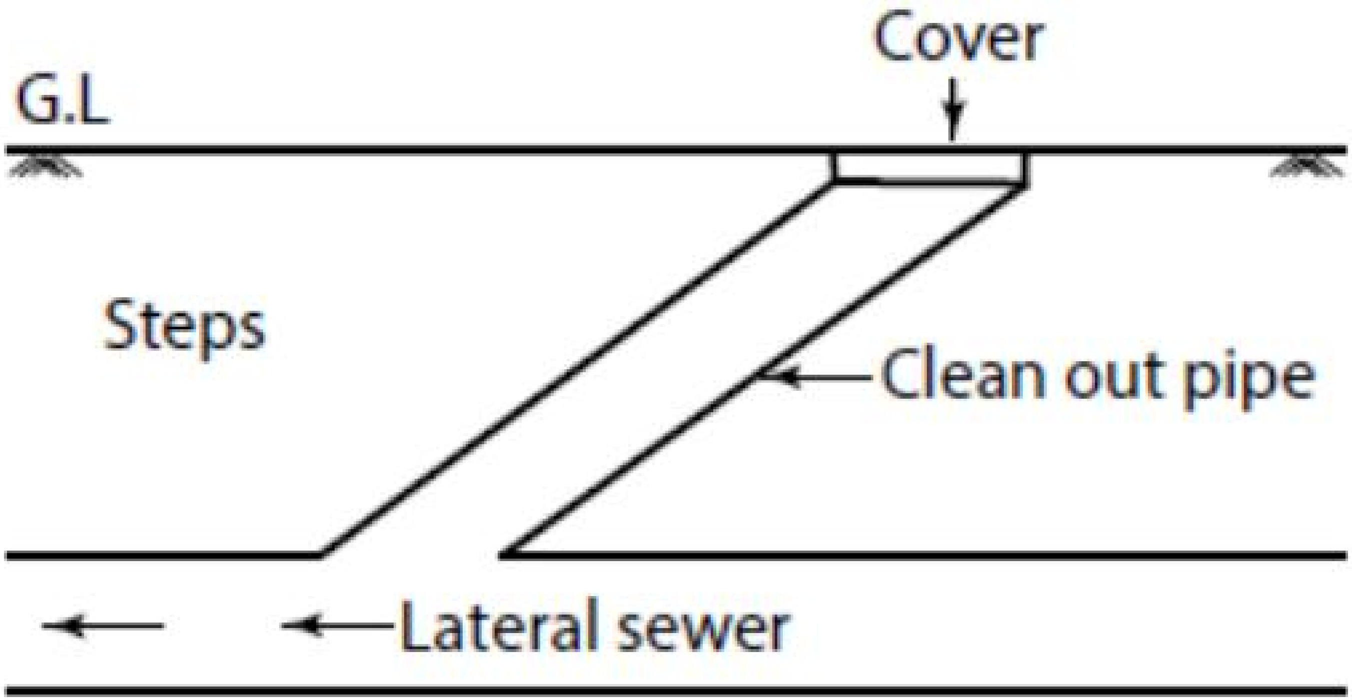


Figure 5.4 Clean-out

CLEAN-OUTS

It is a pipe which is connected to the underground sewer. The other end of the clean-out pipe is brought up to ground level and a cover is placed at ground level (Figure 3.4). A clean-out is generally provided at the upper end of lateral sewers in place of manholes. During blockage of pipe, the cover is taken out and water is forced through the clean-out pipe to lateral sewers to remove obstacles in the sewer line. For large obstacles, flexible rod may be inserted through the clean-out pipe and moved forward and backward to remove such obstacle.

STREET INLETS

Storm water inlets are provided to admit the surface runoff to the sewers. These are classified in three major groups viz. curb inlets, gutter inlets, and combined inlets.

STREET INLETS

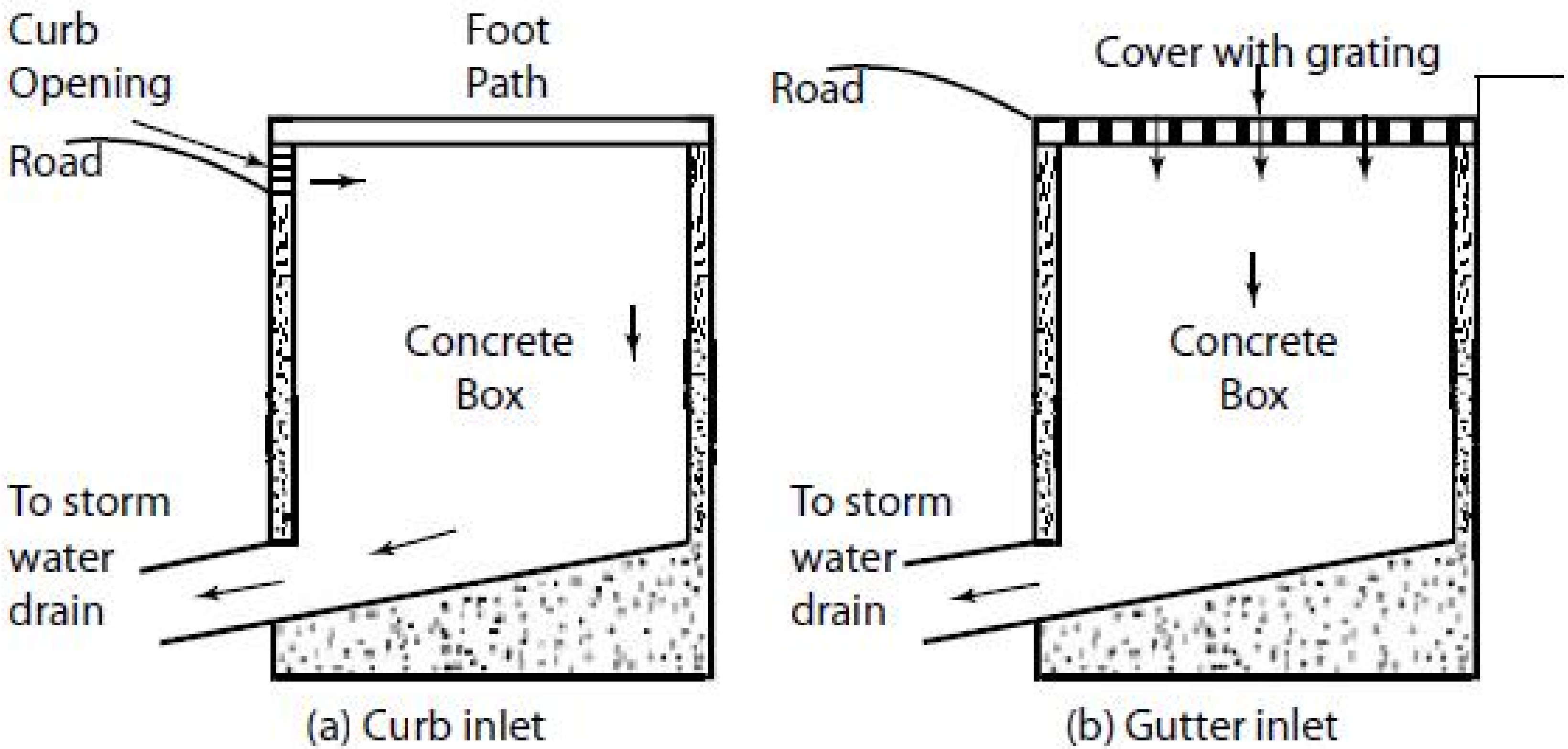


Figure 5.5 (a) Curb inlet and (b) Gutter inlet

STREET INLETS

The Inlets are located by the sides of roads at a distance of about 30 m to 60 m. The inlets are so located that storm water is collected in a short period and there is no flooding on roads. The inlets are connected to nearby manholes by pipe line. The structure of the inlet is constructed with brickwork with cast iron grating at the opening confirming to IS 5961. Where the traffic load is not expected, fabricated steel grating can be used. The clear opening shall not be more than 25 mm. The connecting pipe from the street inlet to the sewer should be minimum of 200 mm diameter and laid with sufficient slope.

CATCH BASINS

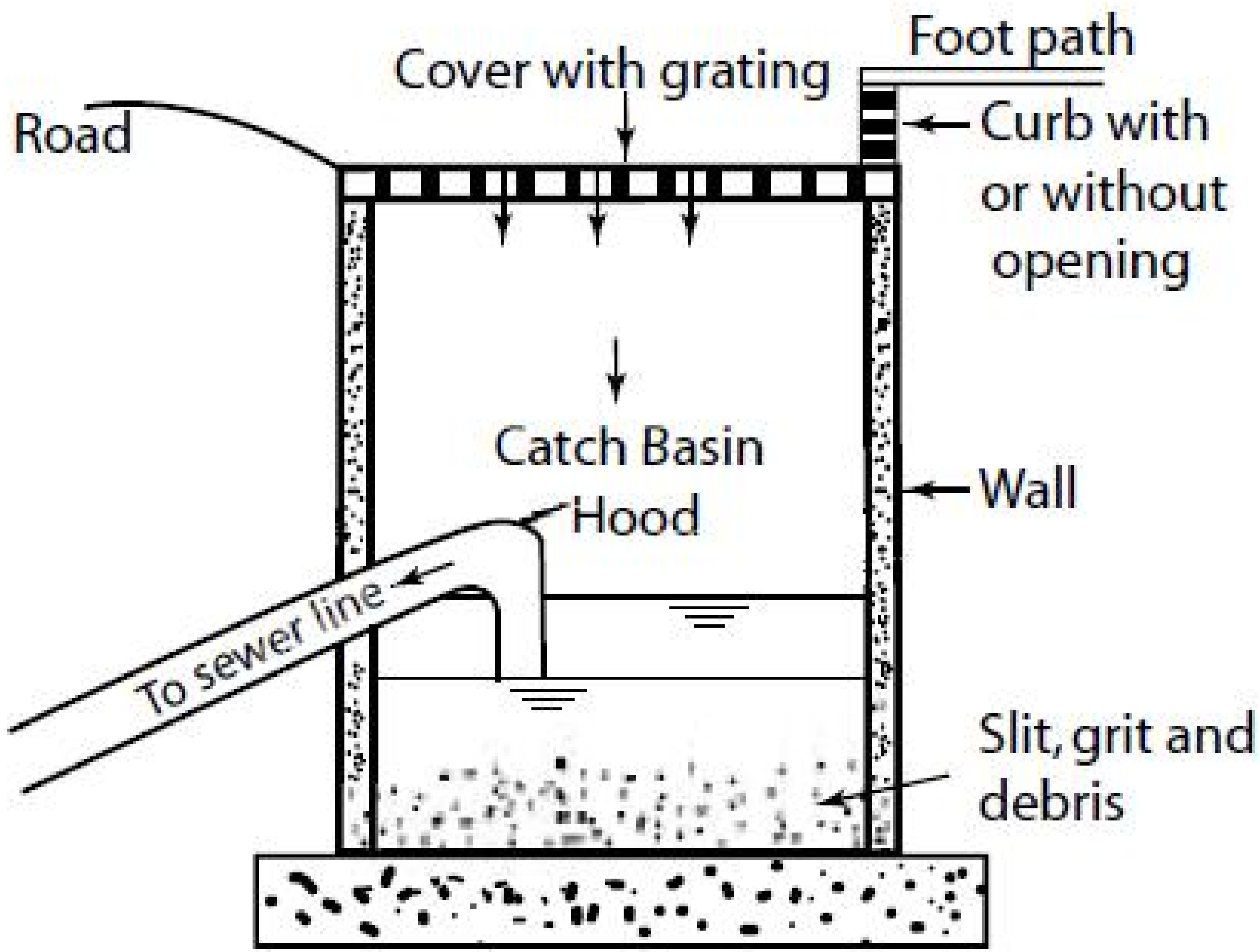


Figure 5.6 Catch basins

CATCH BASINS

A catch basin or catch pit is a special type of inlet, in which a basin is provided which allows grit, sand and debris etc. flowing in with storm water, settle out. The outlet is usually trapped to prevent escape of odours from the sewers and to retain floating matter. The settled matter is taken out periodically. Perforated cover is provided at the top of the basin to admit rain water into the basin. A hood is provided to prevent escape of sewer gas.

FLUSHING TANKS

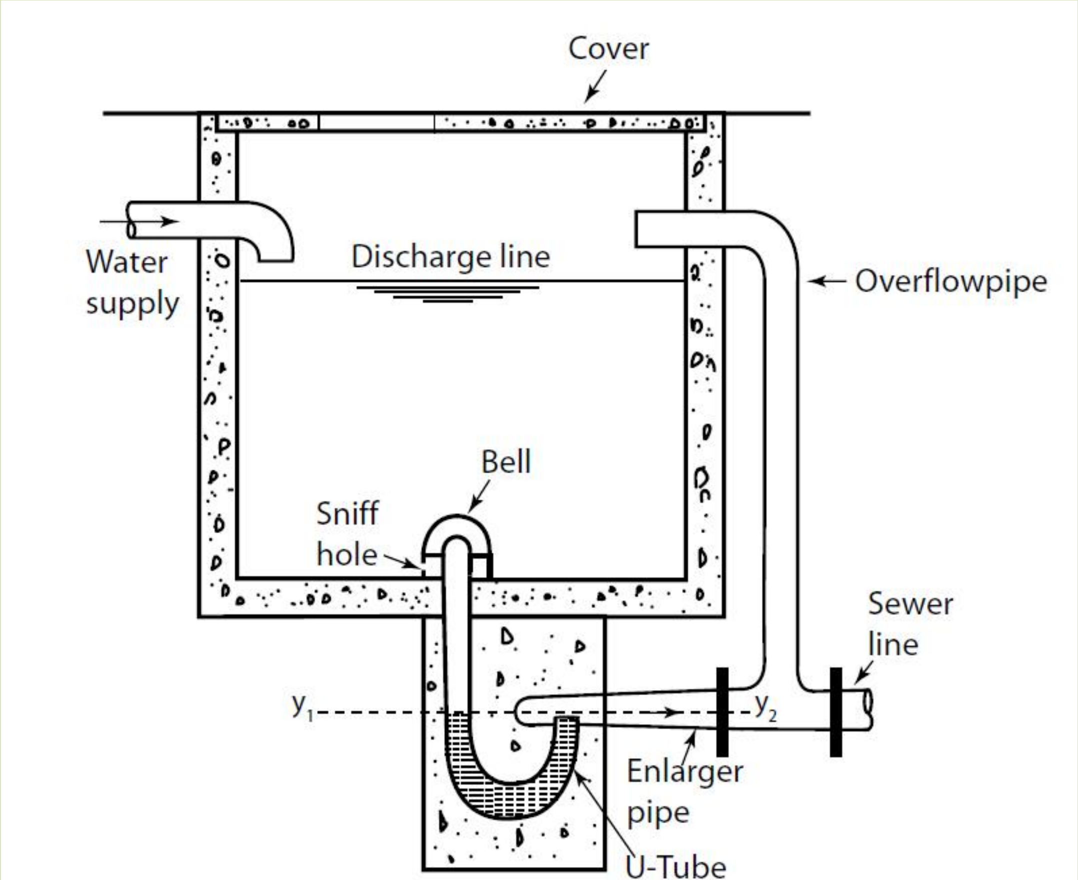
A flushing tank is a device which holds water and then throws it into the sewer for the purpose of flushing it. It can be operated either manually or automatically. Sewer laid on flat gradients may not produce sef-cleansing velocity and may get blocked frequently.They can be cleaned with the help flushing tanks. Flushing may achieved by two methods.

1. Hand operated flushing operation
2. Use of automatic flushing tanks

FLUSHING TANKS

Hand operated flushing operation

One end of hose pipe is connected to the nearest fire hydrants and another end is placed in the manhole to achieve flushing operation.



Use of automatic flushing tanks

Flushing action may be achieved with flushing tanks, automatically at regular intervals. The entry of water is so regulated that the tank is just filled upto the discharge point, in a period equal to flushing interval. An over flow pipe is also provided to drain away excess water in case the tank overflows before the flushing action starts.

Use of automatic flushing tanks

The tank consists of a masonry or concrete chamber fitted with a water tap for filling the tank with water. A U-tube in the bell cap at its one end connects the chamber with the enlarged end of the sewer pipe. The bel cap has sniff hole, as shown. When the water level in the tank is below the sniff hole, the water level in the u-tube is at level Y1 – Y2 . As the water level in the tank rises above the sniffhole, further entry of air into the bell is sealed, and the air caught up in the bell is compressed. This compressed air excerts pressure on the water surface at Y1 , due to which water level in the long arm of the 'U' tube is depressed. As the water level in the tank increases, the water level in the longer arm of 'U'-tube goes on depressing more and more, till a stage is reached when water level in the tank just reaches the discharge level. At this stage some compressed air gets released through the shorter arm and the siphonic action so caused starts releasing water from the tank into the sewer through the enlarge pipe. The siphonic action continues till the water level in the tank just falls to the level of the sniff hole.

SEWAGE CHARACTERISTICS

Characterization of wastes is essential for an effective and economical waste management programme. It helps in the choice of treatment methods deciding the extent of treatment, assessing the beneficial uses of wastes and utilizing the waste purification capacity of natural bodies of water in a planned and controlled manner. While analysis of wastewater in each particular case is advisable, data from the other cities may be utilized during initial stage of planning.

SEWAGE CHARACTERISTICS

Domestic sewage comprises spent water from kitchen, bathroom, lavatory, etc. The factors which contribute to variations in characteristics of the domestic sewage are daily per capita use of water, quality of water supply and the type, condition and extent of sewerage system, and habits of the people. Municipal sewage, which contains both domestic and industrial wastewater, may differ from place to place depending upon the type of industries and industrial establishment. The important characteristics of sewage are discussed here.

TEMPERATURE

The observations of temperature of sewage are useful in indicating solubility of oxygen, which affects transfer capacity of aeration equipment in aerobic systems, and rate of biological activity. Extremely low temperature affects adversely on the efficiency of biological treatment systems and on efficiency of sedimentation. In general, under Indian conditions the temperature of the raw sewage is observed to be between 15 and 35^{o}C at various places in different seasons.

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THE pH

The hydrogen ion concentration expressed as pH, is a valuable parameter in the operation of biological units. The pH of the fresh sewage is slightly more than the water supplied to the community. However, decomposition of organic matter may lower the pH, while the presence of industrial wastewater may produce extreme fluctuations. Generally the pH of raw sewage is in the range 5.5 to 8.0.

COLOUR AND ODOUR

Fresh domestic sewage has a slightly soapy and cloudy appearance depending upon its concentration. As time passes the sewage becomes stale, darkening in colour with a pronounced smell due to microbial activity.

SOLIDS

Though sewage generally contains less than 0.5 percent solids, the rest being water, still the nuisance caused by the solids cannot be overlooked, as these solids are highly degradable and therefore need proper disposal. The sewage solids may be classified into dissolved solids, suspended solids and volatile suspended solids.

SOLIDS

Knowledge of the volatile or organic fraction of solid, which decomposes, becomes necessary, as this constitutes the load on biological treatment units or oxygen resources of a stream when sewage is disposed off by dilution. The estimation of suspended solids, both organic and inorganic, gives a general picture of the load on sedimentation and grit removal system during sewage treatment. Dissolved inorganic fraction is to be considered when sewage is used for land irrigation or any other reuse is planned.

NITROGEN AND PHOSPHORUS

The principal nitrogen compounds in domestic sewage are proteins, amines, amino acids, and urea. Ammonia nitrogen in sewage results from the bacterial decomposition of these organic constituents. Nitrogen being an essential component of biological protoplasm, its concentration is important for proper functioning of biological treatment systems and disposal on land. Generally, the domestic sewage contains sufficient nitrogen, to take care of the needs of the biological treatment.

NITROGEN AND PHOSPHORUS

For industrial wastewater if sufficient nitrogen is not present, it is required to be added externally. Generally nitrogen content in the untreated sewage is observed to be in the range of 20 to 50 mg/L measured as TKN.

NITROGEN AND PHOSPHORUS

Phosphorus is contributing to domestic sewage from food residues containing phosphorus and their breakdown products. The use of increased quantities of synthetic detergents adds substantially to the phosphorus content of sewage. Phosphorus is also an essential nutrient for the biological processes. The concentration of phosphorus in domestic sewage is generally adequate to support aerobic biological wastewater treatment. However, it will be matter of concerned when the treated effluent is to be reused. The concentration of PO_4 in raw sewage is generally observed in the range of 5 to 10 mg/L.

CHLORIDES

Concentration of chlorides in sewage is greater than the normal chloride content of water supply. The chloride concentration in excess than the water supplied can be used as an index of the strength of the sewage. The daily contribution of chloride averages to about 8 grm per person. Based on an average sewage flow of 150 LPCD, this would result in the chloride content of sewage being 50 mg/L higher than that of the water supplied. Any abnormal increase should indicate discharge of chloride bearing wastes or saline groundwater infiltration, the latter adding to the sulphates as well, which may lead to excessive generation of hydrogen sulphide.

FATS GREASE AND OILS

Fats and oil are mainly contributed from kitchen wastes, because they are major components of food stuffs such as butter, lard, margarine, and vegetable oils and fats are also commonly found in meats, seeds, nuts and some fruits. Grease and oils are also discharged from industries like garages, workshops, factories etc. Fats and oils are compounds of alcohol or glycerol with faty acids. Such matters float on the top of sedimentation tanks, often choke pipes in the winter, and clog filters. They thus interfere with functioning of normal treatment plants. The particles interfere with biological action and cause maintenance problems.

SURFACTANTS

Surfactants come primarily from synthetic detergents. These are discharged from bathrooms, kitchens, washing machines etc. Surfactants are large organic molecules which cause foaming in wastewater treatment. Due to this, aeration of wastewater is hindered. The content of surfactants in wastewater is determined by measuring the colour change in standared solution of methylene blue dye.

TOXIC COMPOUNDS

Copper, lead, silver, chromium, arsenic and boron are some of the cations which are toxic to micro-organisms resulting in the malfunctioning of the biological treatment plants. These are results from industrial wastewaters. Some toxic anions, including cyanides and chromates, present in some industrial waste also hinder the waste water treatment facilities. Hence their presence should be taken into consideration in the design of biological treatment plants.

OTHER GASES

Following are the gases that are commonly found in untreated wastewater:

Nitrogen

Oxygen

Carbon dioxide

Amonia

Methane

OTHER GASES

The first three are the gases of atmosphere which are found in all waters exposed to air, the later three are as a result of decomposition of organic matter present in the waste water. Methane gas is the principal by-product of the anaerobic decomposition of the organic matter in wastewater. This gas is colourless and odourless and is highly combustible. Since its explosion hazard is high, manholes, sewer junctions, chambers etc. should be kept well ventilated.

ORGANIC MATERIALS

Organic compounds present in sewage are of particular interest for environmental engineering. A large variety of microorganisms (that may be present in the sewage or in the receiving water body) interact with the organic material by using it as an energy or material source. The utilization of the organic material by microorganisms is called metabolism. The conversion of organic material by microorganism to obtain energy is called catabolism and the incorporation of organic material in the cellular material is called anabolism.

ORGANIC MATERIALS

To describe the metabolism of microorganisms and oxidation of organic material, it is necessary to characterize quantitatively concentration of organic matter in different forms. In view of the enormous variety of organic compounds in sewage it is totally unpractical to determine these individually. Thus a parameter must be used that characterizes a property that all these have in common. In practice two properties of almost all organic compounds can be used: (1) organic compound can be oxidized; and (2) organic compounds contain organic carbon.

ORGANIC MATERIALS

In environmental engineering there are two standard tests based on the oxidation of organic material: 1) the Biochemical Oxygen Demand (BOD) and 2) the Chemical Oxygen Demand (COD) tests. In both tests, the organic material concentration is measured during the test. The essential differences between the COD and the BOD tests are in the oxidant utilized and the operational conditions imposed during the test such as biochemical oxidation and chemical oxidation. The other method for measuring organic material is the development of the Total Organic Carbon (TOC) test as an alternative to quantify the concentration of the organic material.

ORGANIC MATERIALS

Oxygen in a sample of wastewater is reported in the following three ways:

Oxygen consumed

Dissolved oxygen

Oxygen demand

Oxygen consumed: Oxygen consumed is the oxygen required for the oxidation of carbonaceous matter. This quantity of oxygen is determined by adding standard amount of pottassium permanganate with dilute sulphuric acid to a sample of wastewater. This test is made to determine the relative strength of sewage instead of BOD test. But this test does not give the total oxygen needed for the biological oxidation of all or the bulk of the organic matter.

Dissolved oxygen: Dissolved oxygen (DO) is the amount of oxygen in the dissolved state in the wastewater. Though wastewater generally does not have DO, its presence in untreated wastewater indicates that the wastewater is fresh. Similarly, its presence in treated wastewater/effluent indicates that considerable oxidation has been accomplished during the treatment stages. While discharging the treated waste water into receiving waters, it is essential to ensure that atleast 4 ppm of DO is present in it. If DO is less, the aquatic animals like fish etc. are likely to be killed near the vicinity of disposal. The presence of DO in waste water is desirable because it prevents the formation of noxious odours.

for the livelihood of organisms. The aerobic action continues only till the oxygen is present in wastewater, and after that anaerobic action begins resulting in putrefaction. Thus, oxygen is demanded in wastewater for the oxidation of both inorganic as well as organic matter. Thus demand of oxygen may be expressed in the following ways:

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1. Biochemical oxygen demand (BOD)
2. Chemical oxygen demand (COD)

BIOCHEMICAL OXYGEN DEMAND (BOD)

The biochemical oxygen demand (BOD) is a measure of the oxygen requred to oxidize the organic matter present in a sample, through the action of micro-organisms contained in a sample of waste water. It is the most widely used parameter of organic pollution applied to both wastewater as well as surface water. The BOD may be defined as the oxygen required for the micro-organisms to carry out biological decomposition of dissolved solids or organic matter in the wastewater under aerobic conditions at standard temperature.

BIOCHEMICAL OXYGEN DEMAND (BOD)

The BOD test results are used for the following purposes:

1. Determination of approximate quantity of oxygen required for the biological stabilization of organic matter present in the wastewater.
2. Determination of size of wastewater treatment facilities
3. Measurement of efficiency of some treatment process.
4. Determination of strength of sewage.
5. Determination of amount of clear water required for the efficient disposal of wastewater by dilution

Tests for BOD

The BOD can be worked out by the following two mehods.

Direct method

Dilution method

Direct method: The test consists of keeping the sample of wastewater in contact with a definite air or oxygen, in a specially prepared vessel. The BOD is then measured manometrically.

Dilution method: The dilution method is very popular and the procedure for carrying out test by this method is as follows.

1. The sample of sewage is diluted with water of known content of dissolved oxygen. Water should be free from any appreciable oxygen demand and it should not contain any organic matter.
2. The diluted sewage is then kept for 5 days at 20^{o}C in air tight glass vessels after find its initial DO.

Dilution method:

3. The DO is again worked out after this period.
4. The loss of oxygen in the sample of sewage is then found out by calculating the difference between the initial DO and the final content of DO. BOD is then computed from the reation given below:

BOD_5 = [Oxygen consumed] X Dilution ratio

Where, dilution ratio =

$$\frac{\text{Volume of diluted sample}}{\text{Volume of undiluted sewage sample}}$$

Relative stability: Relative stability of wastewater is defined as the ratio of available oxygen to the reqired oxygen satisfying first stage of BOD.

Population Equivalent: The wastewater carried by a sewer consists mainly of domestic sewage and the industrial wastewater. Since the contribution of solids to sewage should be nearly constant on a per capita basis, the BOD contribution should also be constant. Generally, BOD contribution per capita per day may be taken as 80grm/day. Industrial wastewaters are generally compared with per capita domestic sewage, through the concept of population equivalent (P_E) per capita BOD value as the basis.

Thus we have,

Population Equivalent:

$$P_E = \frac{\text{Total BOD}_5 \text{ of the industrial wastewater (kg/day)}}{\text{BOD}_5 \text{ value per capita/day}}$$

For example, if the total BOD_5 of an industrial wastewater is 800 kg/day and BOD_5 value are 0.08 kg/capita/day.

$\therefore$ Population equvalent, $P_E = \frac{800}{0.08}$

= **10,000** persons

Problem: 1

5 ml of raw sewage was diluted by specially prepared water, in a 300ml capacity BOD bottle. The DO concentration of the diluted sample at the beginning of the test was 8 mg/L and 5 mg/L after 5 day incubation at 20°C. Find the BOD of raw sewage.

Solution

$$\text{Dilution ratio} = \frac{\text{Volume of diluted sample}}{\text{Volume of undiluted sewage sample}}$$

$$= \frac{300}{5} = 60$$

$$\text{Loss of DO} = 8 - 5 = 3 \text{ mg/L}$$

$$\therefore \ BOD_5 = 3 \times 60 = \mathbf{180\ mg/L}$$

2.13.2 Theory of BOD reaction

At a given temperature, the rate at which BOD issatisfied at any time (ie rate of deoxygenation) may be assumed to be directly proportional to the amount of organic matter present in sewage. In other words, the exertion BOD is considered to be first order reaction defined by;

$$\frac{dLt}{dt} = -K'.Lt \qquad \text{....1}$$

Where, Lt =Amount of first stage BOD remaining in the sample at any time 't expressed in mg/L.

K' = Rate constant

T = Time in days

Integrating eq.1 between time t=0 to t=t

We get,

$$\int_{L_o}^{L_t} \frac{dL_t}{dt} = -K' \int_0^t dt$$

$$\log_e \frac{L_t}{L_o} = -K't$$

$$\frac{L_t}{L_o} = e^{-K't} = 10^{-Kt} \qquad \text{....2}$$

Where the rate constant K $= \frac{K'}{2.303}$

= Deoxygenation constant

The amount of BOD remaining at any time t is

$$Lt = Lo\,(10^{-kt}) \qquad \ldots.3$$

Where, Lt = BOD at any time t

Lo = Ultimate BOD

K = Reaction constant

T = time in days

Simillerly Y_t the amount of BOD that has been exerted at any time t can be found out from the following equation,

$$Yt = Lo - Lt$$

$$= Lo - Lo\,(10^{-kt}) \qquad \text{(Since } Lt = Lo\,(10^{-kt})\text{)}$$

$$= Lo[1 - 10^{-kt}] \qquad \ldots.4$$

In the above equation Y_t is the BOD of t days.

For instance, 5 day BOD equal to

$$Y_5 = Lo - L_5$$

$$= Lo[1 - 10^{-5K}] \qquad \ldots.5$$

The ultimate first stage BOD, Yu will be obtained by substituting t = α in Eq. 4

ie. $Yu = Lo[1 - 10^{-K\alpha}] = Lo$

In the above equation Y_t is the BOD of t days.

For instance, 5 day BOD equal to

$$Y_5 = Lo - L_5$$

$$= Lo[1 - 10^{-5K}] \qquad \text{....5}$$

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$$Y_5 = Lo - L_5$$

$$= Lo[1 - 10^{-5K}] \quad \text{....5}$$

The ultimate first stage BOD, Yu will be obtained by substituting t = α in Eq. 4

ie. $Yu = Lo[1 - 10^{-K\alpha}] = Lo$

Problem :2

Determine ultimate BOD and I day BOD for a sewage having 5 day BOD at $20^{\circ}C$ as 200 ppm, assume K_{20}=0.1.

Problem :2

Determine ultimate BOD and I day BOD for a sewage having 5 day BOD at 20°C as 200 ppm, assume K_{20}=0.1.

Solution

$$Y5 = Lo - L_5$$

$$= Lo[1 - 10^{-5K}]$$

$$200 = Lo\ [1 - 10^{-5X0.1}]$$

$$\therefore Lo = 292.5\ \text{ppm}$$

First day BOD

$$L_1 = Lo(10^{-k\ 1})$$

$$= 292.50\ X\ 10^{-0.1X1}$$

$$= 232.4\ \text{ppm}$$

$$Y_1 = 292.50 - 232.40$$

$$= \mathbf{60.16\ ppm}$$

Problem: 3

The values of R for three samples of sewage are 0.12, 0.18 and 0.25 per day respectively. If 5 day BOD of each sample is 250 mg/L, Calculate the ultimate BOD of each sample.

Solution

Five day BOD = 250 mg/L

$Y5 = Lo - L_5 = Lo[1 - 10^{-5K}]$

Where,

Lo - ultimate BOD

K = R and

$Y_5 = 250$ mg/L

ultimate BOD of sample 1

$$250 = Lo[1\text{-}10^{-5X0.12}]$$

$\therefore$Lo = **333.9** mg/L

Ultimate BOD of sample 2

$$250 = Lo[1\text{-}10^{-5X0.18}]$$

$\therefore$Lo = **286** mg/L

Ultimate BOD of sample 3

$$250 = Lo[1\text{-}10^{-5X0.25}]$$

$\therefore$Lo = **264.9** mg/L

Problem: 4

If the contribution of suspende solids and BOD is 100 and 60 g/c/d, estimate the polution equivalent of:

i) Combined system serving 1000 persons and having 125 gram per capita daily BOD.

ii) 45000 lts daily contribution of industrial waste containing 1700 mg/L of suspended solids.

Solution

i) In the first case polution equivalent is based on the criterian of BOD.

Per capita BOD = 60 grm.

Total daily BOD load $= 1000 \text{ X } 125$

$= 12.5 \text{ x } 10^4$ grm.

$\therefore$ Population equivalent $= \dfrac{12.5 \, X \, 10^4}{60}$

$= 2084$

ii) In the second case, population equivalent is based on the criterian of suspended solids per capita suspende solids = 100 gram.

$$\text{Total daily load of suspende solids} = \frac{45000 \; X \; 1700}{10^3}$$

$$= 76500 \text{ grm.}$$

$$\therefore \text{Population equivalent} = \frac{76500}{100}$$

$$= 765$$

CHEMICAL OXYGEN DEMAND (COD)

The BOD test takes a minimum of 5 days time, and due to this, it is not useful in the control of treatment processes. An alternative test is the COD test. COD can be determined only in 3 hours in contrast to 5 days BOD test. In COD test, a strong chemical oxidising agent is used in acidic medium to measure the oxygen equivalent of organic matter that can be oxidised. The COD test involes an acidic oxidation with pottassium dicromate.

CHEMICAL OXYGEN DEMAND (COD)

A measured amount of pottassium dicromate is added to the sample. The acidified sample is then boiled for 2 hours, cooled and the amount of dicromate remaining is measured by titration with ferrous ammonium sulphate. To accelerate the oxidation of certain types of organic compounds a catalyst, usually silver sulphate, is required to aid the oxidation.

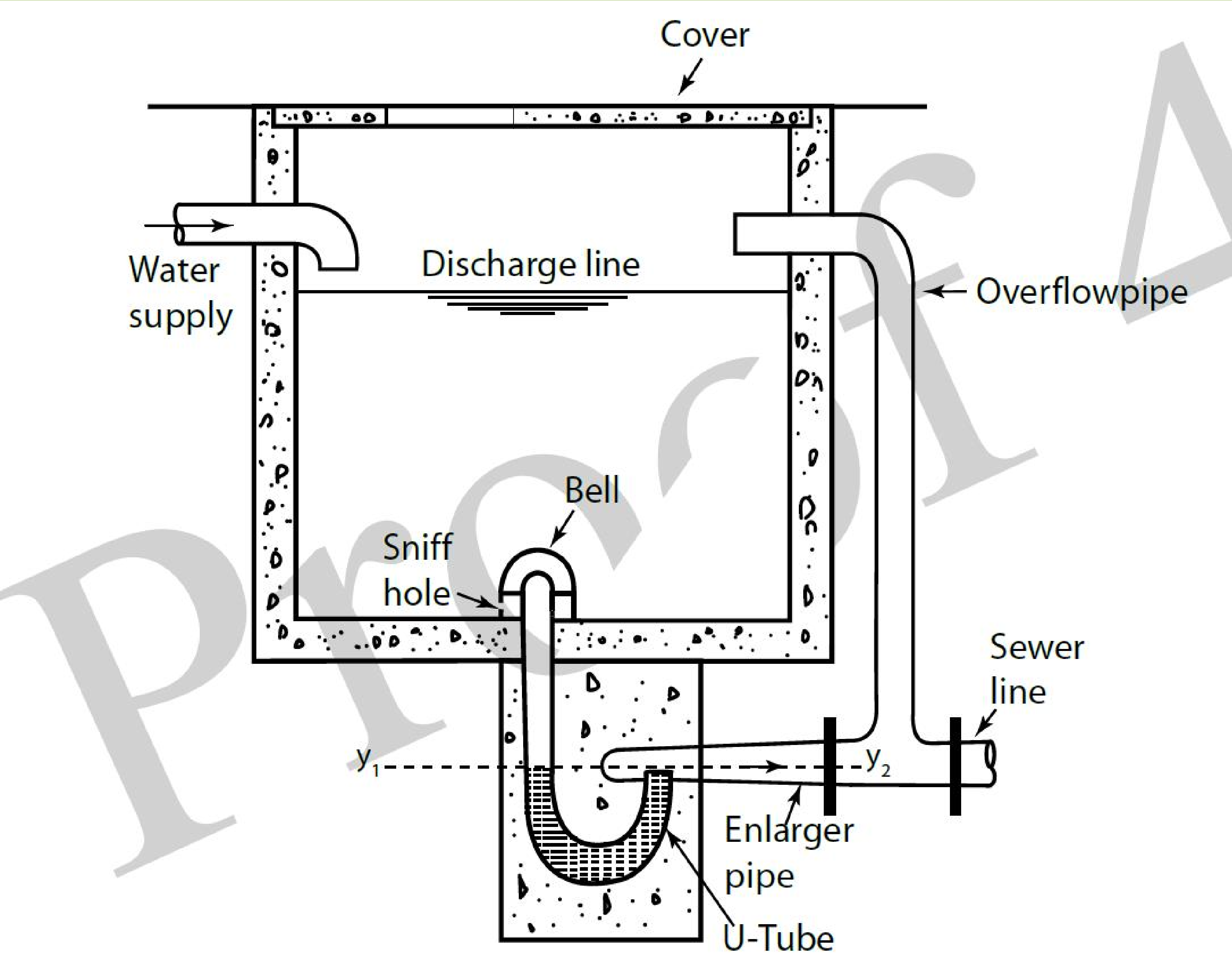
BACTERIOLOGICAL TEST

In sewage analysis, bacteriological tests are of not much importance and hence they are rarely made. Presence of bacteria in a sample of sewage has no effect on the choice or selection of the sewage treatment method. Presence of biological life is essential in sewage for the efficient working of the treatment units. Absence of bacteria in a sample of sewage indicates the presence of industrial waste which is harmful to the bacterial life in sewage.

Use of automatic flushing tanks

The tank consists of a masonary or concrete chamber fitted with a water tap for filling the tank with water. A U-tube in the bell cap at its one end connects the chamber with the enlarged end of the sewer pipe. The bel cap has sniff hole, as shown. When the water level in the tank is below the sniff hole, the water level in the u-tube is at level $Y_1 - Y_2$. As the water level in the tank rises above the sniffhole, further entry of air into the bell is sealed, and the air caught up in the bell is compressed. This compressed air exerts pressure on the water surface at Y_1, due to which water level in the long arm of the 'U' tube is depressed.

FLUSHING TANKS



Use of automatic flushing tanks

As the water level in the tank increases, the water level in the longer arm of ‘U’-tube goes on depressing more and more, till a stage is reached when water level in the tank just reaches the discharge level. At this stage some compressed air gets released through the shorter arm and the siphonic action so caused starts releasing water from the tank into the sewer through the enlarge pipe. The siphonic action continues till the water level in the tank just falls to the levell of the sniff hole.

GREASE & OIL TRAPS

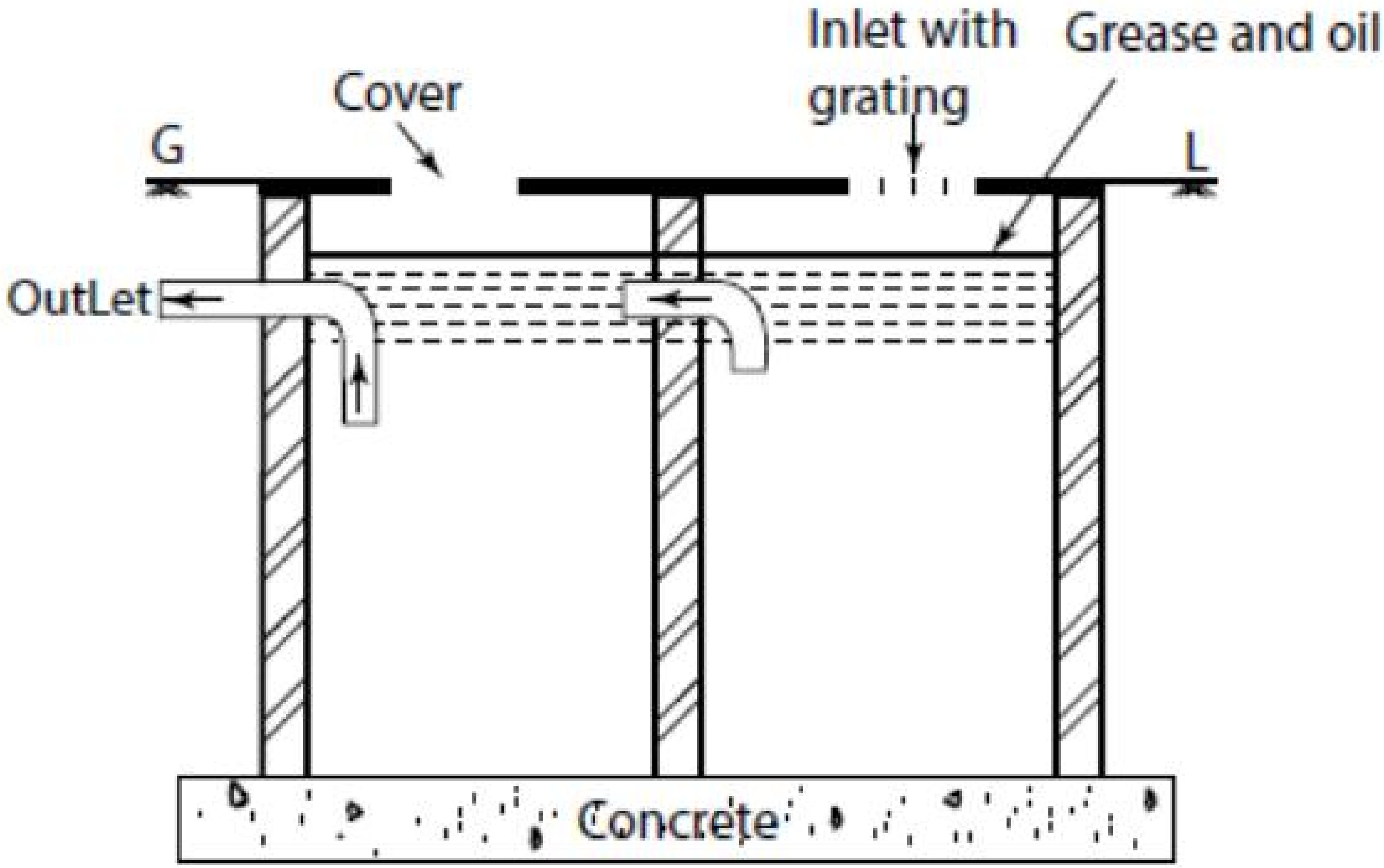


Figure 5.8 Grease and oil traps

GREASE & OIL TRAPS

These are traps or chambers which are provided on the sewer line to exclude grease and oil from sewage before they enter the sewer line.

The grease and oil being light in weight float on the surface of sewage. Hence if outlet draws the sewage from lover level, grease and oil are excluded. Thus the outlet level is near the bottom of chamber and inlet level is near the top of chamber.

Figure 3.8 shows a typical grease and oil trap. It consists of two chambers. The inlet with gratings is provided just near the top and end of outlet is located at a height of about 60cm from the bottom of trap.

INVERTED SIPHONS

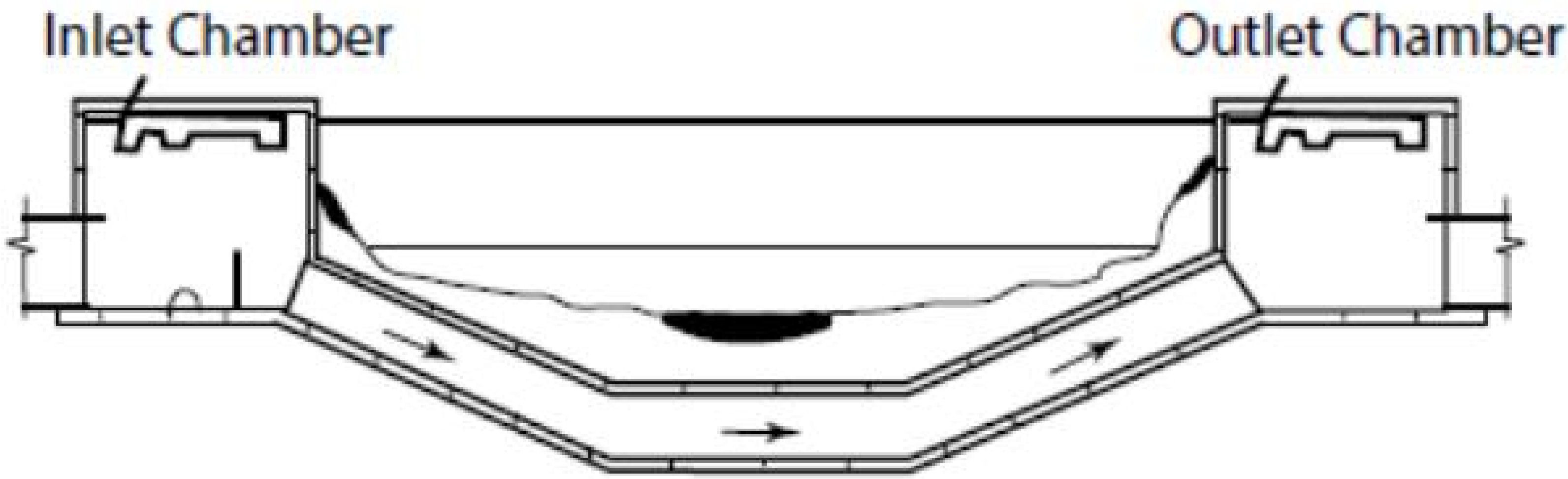


Figure 5.9 Inverted siphons

INVERTED SIPHONS

An inverted siphon or depressed sewer is a sewer that runs full under gravity flow at a pressure above atmosphere in the sewer. Inverted siphons are used to pass under obstacles such as buried pipes, subways, etc (Figure 3.9). This terminology 'siphon' is misnamed as there is no siphon action in the depressed sewer. As the inverted siphon requires considerable attention for maintenance, it should be used only where other means of passing an obstacle in line of the sewer are impracticable. It consists of inlet chamber, outlet chamber and three pipes laid parellel to each other.

STORM REGULATORS

These are used for preventing overloading of sewers, pumping stations, treatment plants or disposal arrangement, by diverting the excess flow to relief sewer. The main object of providing a storm water regulator is to divert the excess storm water to natual stream or river to reduce the load on the treatment units or pumping stations. Following are the three types of storm regulators.

1. Leaping weir
2. Overflow weir and side flow weir
3. Siphon spillway

Leaping weir

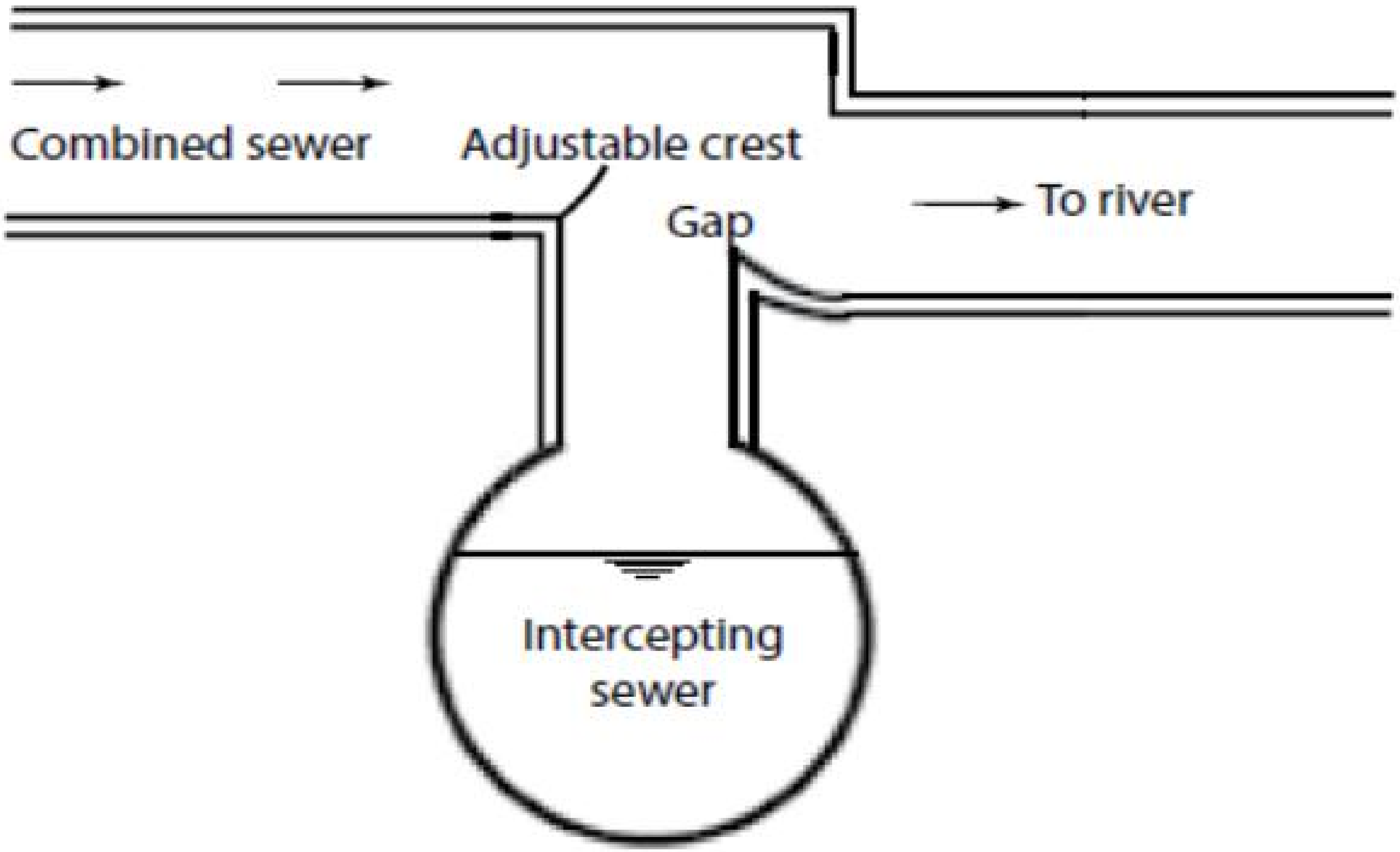


Figure 5.10 Leaping weir with adjustable crest

Leaping weir

The term leaping weir is used to indicate the gap or opening in the invert of a combined sewer. The intercepting sewer runs at right angles to the combined sewer. The leaping weir is formed by a gap in the invert of a sewer through which the dry weather flow falls and over which a portion of the entire storm leaps. When discharge is small, the sewage falls directly into the intercepting sewer through the opening. But when the discharge exceeds a certain limit, the excess sewage leaps or jumps across the weir and it is carried to natural stream or river. This arrangement is shown in the Figure 310. This has an advantage of operating as regulator without involving moving parts.

Leaping weir

However, the disadvantage of this weir is that, the grit material gets concentrated in the lower flow channel. From practical consideration, it is desirable to have moving crests to make the opening adjustable.

Overflow weir and side flow weir

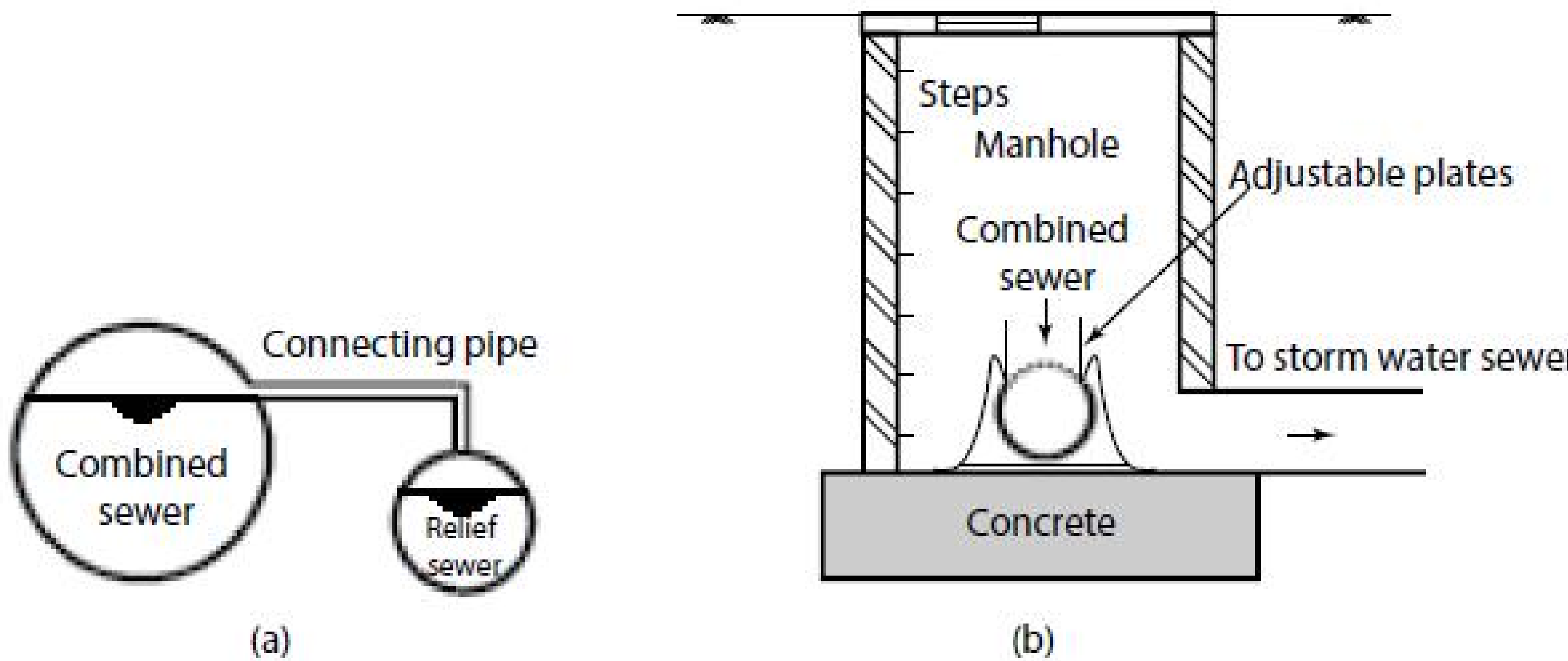


Figure 5.11 (a) Side flow weir (b) Overflow weir arrangement

Overflow weir and side flow weir

Overflow weir: In this type of arrangement, the excess sewage is allowed to overflow in the channel made in the manhole as shown in figure 3.11(b). When the quantity of sewage exceeds the capacity of the combined sewer, it overflows and falls into the channels and it is conveyed to storm water sewer. In order to prevent the escape of floating matter from the combined sewer channel, adjustable plates are sometimes provided.

Overflow weir and side flow weir

Side flow weir: It is constructed along one or both sides of the combined sewer and delivers the excess flow during storm period to relief sewers or natural drainage courses (Figure 3.11. (a)). The crest of the weir is set at an elevation corresponding to the desired depth of flow in the sewer. The weir length must be sufficient long for effective regulation of the flow.

Siphon spillway

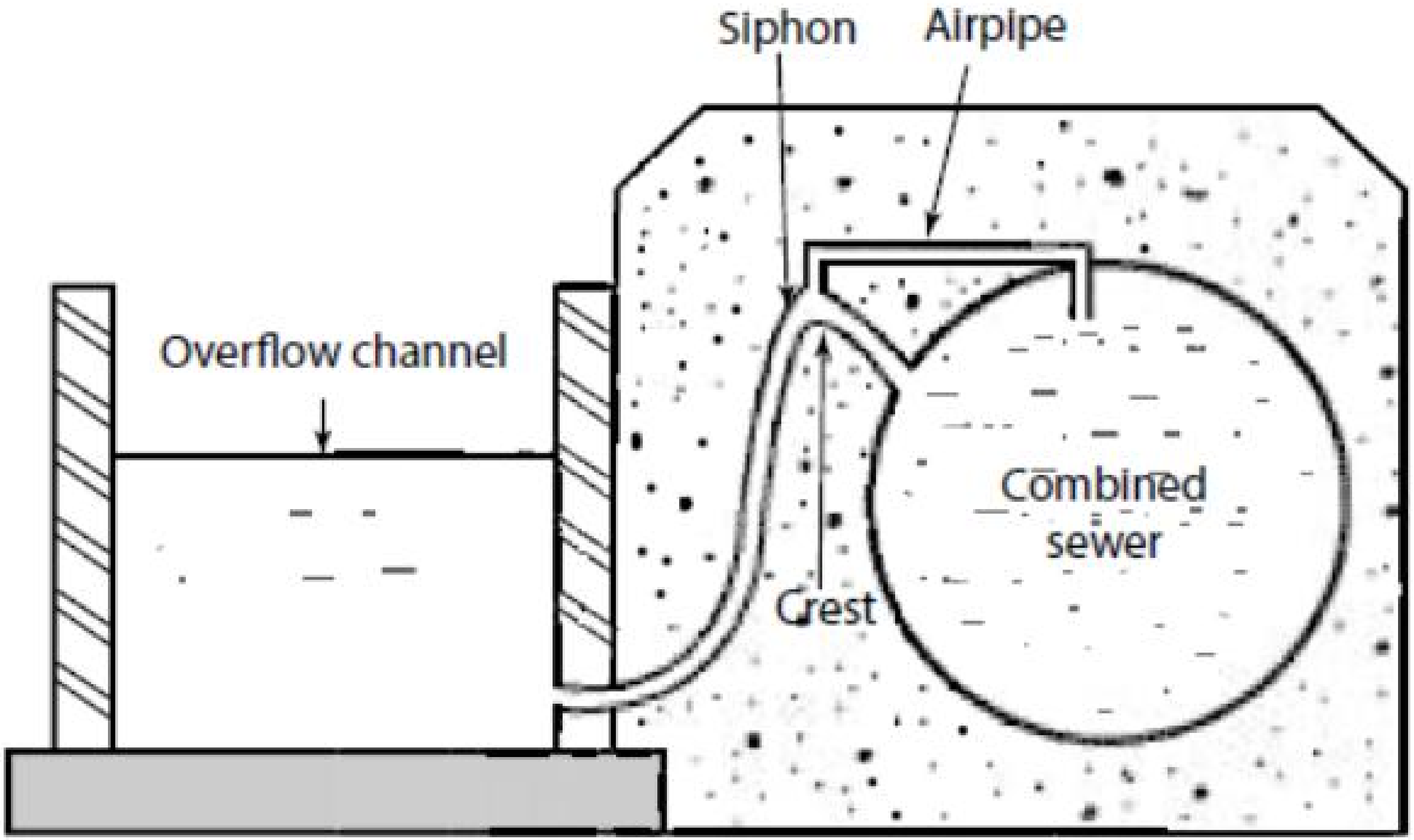


Figure 5.12 Siphon spillway

Siphon spillway

It consists of a siphon connected to the main combined sewer. The lower end of the siphon is connected to an overflow channel which leads excess water to a natural drain or water course. An air pipe connects the throat of the siphon to the combined sewer.

Siphon spillway

The level of the crest of siphon is kept at the predetermined desired level. As soon as the flow in the combined sewer exceeds this level, the mouth of air pipe is closed and the air contained in the siphon pipe is suddenly carried away by the flow, causing the priming action. The suction thus developed starts the siphonic action and flow is immediately established in the siphon. When the flow reduces and when water level falls below the crest, mouth of air pipe gets exposed, air enters and depriming takes place resulting in breaking of the siphonic action.

VENTILATION OF SEWERS

Various gases are produced in sewers due to purification of organic materials of sewage. These gases are very foul in nature, cause harmful to human health and corrode the sewers. The gases so produced are highly explosive and may cause accidents. Due to the above difficulties, the sewers must be properly ventilated

VENTILATION OF SEWERS

The following methods are adopted for ventilating the sewers:

1) Use of ventilating columns:

The ventilating shaft is provided along the sewer line at an interval of 150 m to 300 m. They are also provided at the upper end of every branch sewer and at every points where sewer dia changes. Ventilating shaft helps to remove the foul and explosive gases produced in the sewer. They provide fresh air to the workers working in the manholes. They also help to prevent the formation of air locks in the sewage and there by ensure the continuous flow of sewage inside the sewer.

VENTILATION OF SEWERS

2) Use of ventilating manhole covers:

The manhole covers are sometimes provided with perforations, through which the sewer gets exposed to the atmosphere. This will help in achieving some ventilation, but will cause more nuisances.

3) Proper design of sewers:

The proper design of sewers also ensures enough ventilation.

VENTILATION OF SEWERS

4) Use of mechanical devices:

Forced draught is sometimes provided by exhaust fans to expel out the foul gases from the sewers.

5) House vent and soil pipes:

They may directly help in ventilating house drains and public sewers.

6) Artificial ventilation:

Sometimes artificial ventilation is arranged at site before entering a sewer, by blowing fresh air supply into the sewer though mechanical means.

VENTILATION OF SEWERS

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MODULE III

WASTEWATER DISPOSAL SYSTEMS

After conveying the wastewater through sewers, the next step is its disposal, either after treatment or even before treatment. There are two general methods of disposing of the sewage effluents:

i. Disposal by dilution

i. Disposal on land (Sewage farming)

Condition favorable for disposal by dilution

The dilution method for disposing of the sewage can favorably by adopt under the following conditions:

i) When sewage is comparatively fresh and free from floating and settleable solids.

ii) When the diluting water has high dissolved oxygen content.

iii) Where diluting waters are not used for the purpose of navigation or water supply for at least some reasonable distance on the downstream from the point of sewage disposal.

Condition favorable for disposal by dilution

iv) Where the flow currents of the diluting waters are favorable, causing no deposition, nuisance or destruction of aquatic life. It means that swift forward currents are helpful, as they easily carry away the sewage to the points of unlimited dilution. On the other hand, slow back currents tend to cause sedimentation, resulting in large sludge deposits.

v) When the outfall sewer of the city or the treatment plant is situated near some natural waters having large volumes.

SELF PURIFICATION OF STREAMS

When the wastewater or the effluent is discharged into a natural stream, the organic matter is broken down by bacteria to ammonia, nitrates, sulphates, carbon dioxide etc. In this process of oxidation, the dissolved oxygen content of natural water is utilised. Due to this, deficiency of dissolved oxygen is created. As the excess organic matter is stabilized, the normal cycle will be reestablished in a process known as self-purification where in the oxygen in replenished by its reaeration by wind.

SELF PURIFICATION OF STREAMS

Also, the stable byproducts of oxidation mentioned above are utilized by plants and algae to produce carbohydrates and oxygen. Water quality standards are often based upon maintenance of some minimum dissolved oxygen concentration which will protect the natural cycle in the stream while taking advantage of its natural assimilative capacity.

Actions involved in self purification.

The following are various factors which affect the process of self purification:

1. Dilution: When sufficient dilution water is available in the receiving water body, where the wastewater is discharged, the DO level in the receiving stream may not reach to zero or critical DO due to availability of sufficient DO initially in the river water before receiving discharge of wastewater.

If Cs and C_R are the concentrations of any impurity such as organic content, BOD, suspended solids in the sewage and river having discharge rates Qs and Q_R respectively, the resulting concentration C of the mixture is given by

$$CsQs + C_R Q_R = C(Qs + Q_R)$$

or

$$C = \frac{CsQs + C_R Q_R}{Qs + Q_R}$$

When the dilution ratio is quite high, large quantities of DO are always available which will reduce the chance of putrification and pollutional effects. Aerobic conditions will always exist because of dilution. This will however, not be there if dilution ratio is small, when large quantities of effluent are discharged into a small stream.

2. Current: When strong water current is available, the discharged wastewater will be thoroughly mixed with stream water preventing deposition of solids. In small current, the solid matter from the wastewater will get deposited at the bed following decomposition and reduction in DO.

3. Temperature: The quantity of DO available in stream water is more in cold temperature than in hot temperature. Also, as the activity of microorganisms is more at the higher temperature, hence, the self-purification will take less time at hot temperature than in winter.

4. Sunlight: Algae produces oxygen in presence of sunlight due to photosynthesis. Therefore, sunlight helps in purification of stream by adding oxygen through photosynthesis.

5. Rate of Oxidation: Due to oxidation of organic matter discharged in the river DO depletion occurs. This rate is faster at higher temperature and low at lower temperature. The rate of oxidation of organic matter depends on the chemical composition of organic matter.

6. Sedimentation: If the stream velocity is lesser than the scour velocity of particles, sedimentation will take place, which will have two effects: (i) the suspended solids, which is contribute largely to oxygen demand will be removed by settling and hence water quality to the downstream will be increased, (ii) due to settled solids, anaerobic decomposition may take place.

7. Reduction: The reduction occurs in the streams due to hydrolysis of the organic matter biologically. Anaerobic bacteria will split the organic matter into liquids and gases, thus paving way for their ultimate stabilization by oxidation.

Zones of pollution in the stream

The self purification process of stream polluted by the wastewater or effluent discharged into it can be divided into the following four zones:

(i) Zone of degradation,

(ii) Zone of active decomposition,

(iii) Zone of recovery,

(iv) Clear water zone.

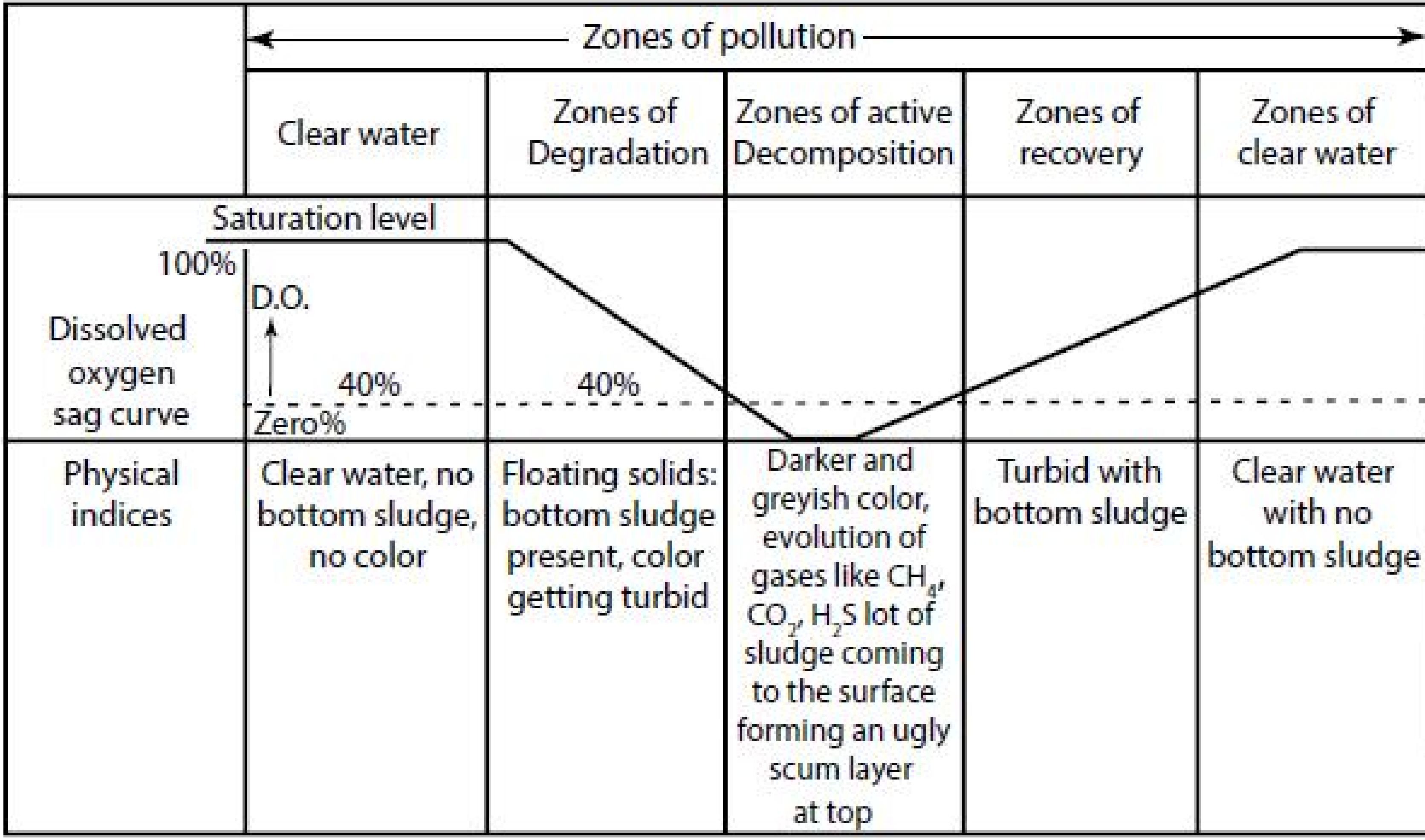

Figure 6.1 Zones of pollution in the stream

(i) Zone of degradation:

This zone is situated just below the outfall sewer when discharging its contents into the stream. In this zone, water is dark and turbid, having the formation of sludge deposits at the bottom. This zone is found for a certain length just ahead of the point where sewage is discharged into the river. Bacteria will start oxidation of the organic matter. The DO is reduced to about 40 % of the saturation value. There is an increase in carbon dioxide content. Oxygenation occurs but is slower than deoxygenating. These conditions are unfavorable to the development of aquatic life; and as such, algae dies out, but certain type of fish may be present feeding on fresh organic matter. The decomposition of solid matter takes place in this zone and anaerobic decomposition prevails.

(ii) Zone of active decomposition: This zone is just after the degradation zone and is marked by heavy pollution. It is characterized by water becoming grayish and darker than in the previous zone. DO concentration falls down to zero, an anaerobic conditions may set in with the evolution of gases like methane, carbon dioxide, hydrogen sulphide, etc. bubbling to the surface, with masses of sludge forming ugly scum layer at the surface.

(ii) Zone of active decomposition: Cont…..

As the organic decomposition slackens due to stabilization of organic matter, rate of deoxygenating becomes lesser than the re-aeration rate and DO again rises to the original level (i.e. above 40 %). In this region, the bacteria, flora will flourish. At the upper end, anaerobic bacteria will replace aerobic bacteria, while at the lower end, the position will be reversed. Protozoa and fungi first disappear then reappear. Fish life will be absent.

(iii) Zone of recovery:

In this zone, the river stream tries to recover from its degraded condition to its former appearance. The water becomes clearer, and so the algae reappear while fungi decreases. BOD decreases and DO content rises above 40 % of the saturation value. The organic material will be mineralised to form nitrate, phosphate, carbonates, etc. Near the end of the zone, microscopic aquatic life reappears, fungi decrease and algae reappears.

OXYGEN SAG ANALYSIS

The oxygen sag or oxygen deficit in the stream at any point of time during self purification process is the difference between the saturation DO content and actual DO content at that time.

Oxygen deficit, D = Saturation DO – Actual DO

OXYGEN SAG ANALYSIS

The saturation DO value for fresh water depends upon the temperature and total dissolved salts present in it; and its value varies from 14.62 mg/L at 0^oC to 7.63 mg/L at 30^oC, and lower DO at higher temperatures.

OXYGEN SAG ANALYSIS

The DO in the stream may not be at saturation level and there may be initial oxygen deficit 'Do'. At this stage, when the effluent with initial BOD load Lo, is discharged in to stream, the DO content of the stream starts depleting and the oxygen deficit (D) increases. The variation of oxygen deficit (D) with the distance along the stream, and hence with the time of flow from the point of pollution is depicted by the 'Oxygen Sag Curve' (Figure 4.2). The major point in sag analysis is point of minimum DO, i.e., maximum deficit. The maximum or critical deficit (Dc) occurs at the inflexion points of the oxygen sag curve.

OXYGEN SAG ANALYSIS

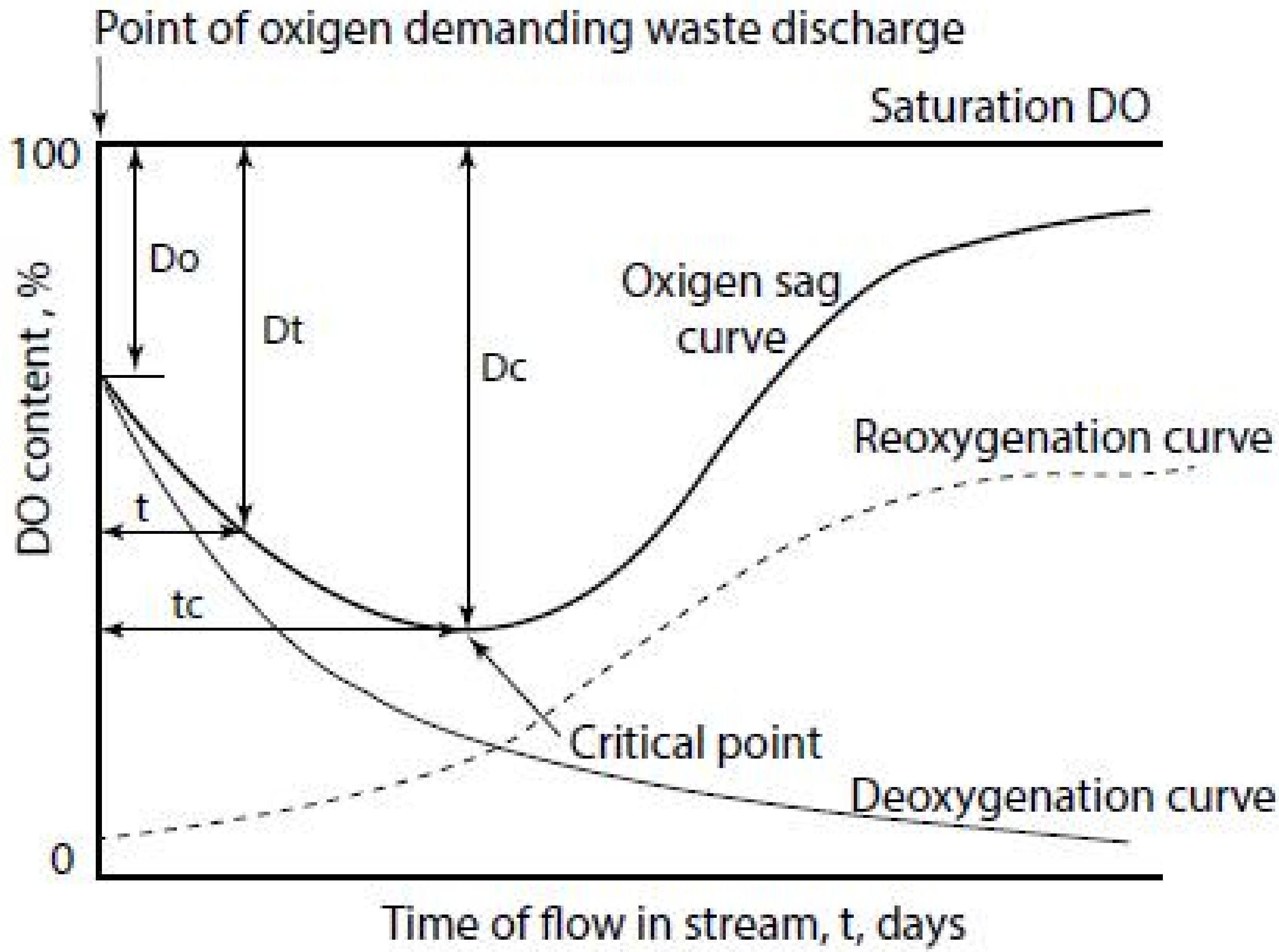

Figure 6.2 Deoxygenation, reoxygenation and oxygen sag curve

Deoxygenation and Reoxygenation Curves

Deoxygenation Curve: When wastewater is discharged into the stream, the DO level in the stream goes on depleting. This depletion of DO content is known as deoxygenation. The rate of deoxygenation depends upon the amount of organic matter remaining (Lt) to be oxidized at any time t, as well as temperature (T) at which reaction occurs. The variation of depletion of DO content of the stream with time is depicted by the deoxygenation curve in the absence of aeration. The ordinates below the deoxygenation curve indicate the oxygen remaining in the natural stream after satisfying the bio-chemical oxygen demand of oxidizable matter.

Deoxygenation and Reoxygenation Curves

***Reoxygenation Curve**:* When the DO content of the stream is gradually consumed due to BOD load, atmosphere supplies oxygen continuously to the water, through the process of re-aeration or reoxygenation, i.e., along with deoxygenation, re-aeration is continuous process.

The rate of reoxygenation depends upon:

1. Depth of water in the stream: more for shallow depth.
2. Velocity of flow in the stream: less for stagnant water.
3. Oxygen deficit below saturation DO: since solubility rate depends on difference between saturation concentration and existing concentration of DO.
4. Temperature of water: solubility of oxygen is lower at higher temperature and also saturation concentration is less at higher temperature.

OXYGEN SAG CURVE

In a running polluted stream exposed to the atmosphere, the de-oxygenation as well as the re-oxygenation goes hand in hand. If de-oxygenation is more rapid than the re-oxygenation an oxygen deficit is results.

The amount of resultant oxygen deficit can be obtained by algebraically adding the de-oxygenation and re-oxygenation curves. The resultant curve so obtained is called the oxygen sag curve or the oxygen deficit curve. From this curve, the oxygen deficit and oxygen balance present in a stream after a certain lapse, can be found out.

4.5 MATHEMATICAL ANALYSIS OF OXYGEN SAG CURVE: STREETER – PHELPS EQUATION

The analysis of oxygen sag curve can be easily done by superimposing the rates of deoxygenation and reoxygenation as suggested by the Streeter – Phelps analysis. The rate of change in the DO deficit is the sum of the two reactions as explained below:

$$\frac{dDt}{dt} = f\text{ (deoxygenation and reoxygenation)}$$

or
$$\frac{dDt}{dt} = K^1 Lt - R^1 Dt \qquad \text{....(1)}$$

Where,

Dt = DO deficit at any time t,

Lt = amount of first stage BOD remaining at any time t

K^1 = BOD reaction rate constant or deoxygenation constant (to the base e)

R^1 = Reoxygenation constant (to the base e)

t = time (in days)

$\frac{dDt}{dt}$ = rate of change of DO deficit

Now,

Where, Lo = BOD remaining at time t = 0 i.e. ultimate first stage BOD

Hence,

$$\frac{dDt}{dt} = K\,Loe^{-k^1 t} - R^1\,Dt \qquad \text{....(2)}$$

or

$$\frac{dDt}{dt} + R^1\,Dt = K\,Loe^{-K^1 t} \qquad \text{....(3)}$$

This is first order first degree differential equation and solution of this equation is as under.

$$Dt = \frac{K^1 Lo}{R^1 - K^1}\left[e^{-K^1 t} - e^{-R^1 t}\right] + Do.\,e^{-R^1 t} \qquad \text{....(4)}$$

Changing base of natural log to 10 the equation can be expressed as:

$$Dt = \frac{KLo}{R-K}\left[10^{-kt} - 10^{-Rt}\right] + Do.\,10^{-R^1 t} \qquad \text{....(5)}$$

Where,

K = BOD reaction rate constant, to the base 10

R = Reoxygenation constant to the base 10

Do = Initial oxygen deficit at the point of waste discharge at time t = o

t = time of travel in the stream from the point of discharge

= x/u

x = distance along the stream

u = stream velocity

Now,

Where, Lo = BOD remaining at time t = 0 i.e. ultimate first stage BOD

Hence,

$$\frac{dDt}{dt} = K\,Loe^{-k^1 t} - R^1\,Dt \qquad ....(2)$$

or

$$\frac{dDt}{dt} + R^1\,Dt = K\,Loe^{-K^1 t} \qquad ....(3)$$

This is first order first degree differential equation and solution of this equation is as under.

$$Dt = \frac{K^1 Lo}{R^1 - K^1}\left[e^{-K^1 t} - e^{-R^1 t}\right] + Do.\,e^{-R^1 t} \qquad ....(4)$$

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Do = Initial oxygen deficit at the point of waste discharge at time t = o

t = time of travel in the stream from the point of discharge

= x/u

x = distance along the stream

u = stream velocity

This is Streeter-Phelps oxygen sag equation. The graphical representation of this equation is shown in Figure 4.3.

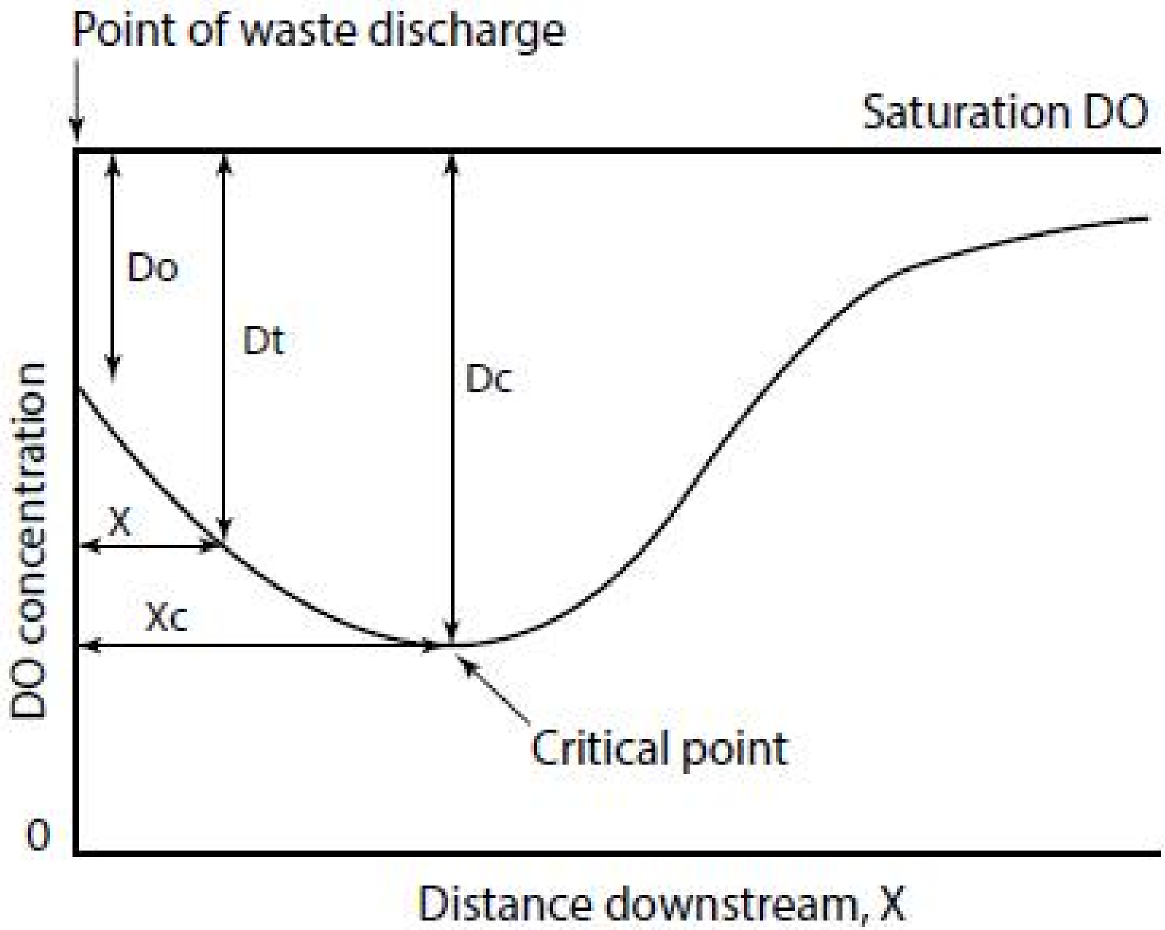

Figure 6.3 **Oxygen sag curve of Streeter-Phelps equation**

Determination of Critical DO deficit (Dc) and distance (Xc)

The value of Dc can be obtained by putting dDt/dt = 0 in equation 3,

Hence,

$$Dc = \frac{K'}{R'}\ Lo.\, e^{-K'tc} \qquad ....(6)$$

or

$$Dc = \frac{K'}{R'}\ Lo.\, 10^{-K'tc} \qquad ....(7)$$

Where,' t_c' is time required to reach the critical point.

The value of 't_c' can be obtained by differentiating equation 4 (or 5) with respect to 't' and setting dDt/dt = 0

Therefore,

$$t_c = \frac{1}{R'-K'} \log_e \frac{R'}{K'} \left[1 - \frac{Do\,(R'-K')}{K'Lo}\right] \qquad ....(8)$$

or

$$t_c = \frac{1}{R-K} \log_{10} \frac{R}{K} \left[1 - \frac{Do\,(R-K)}{K.Lo}\right] \qquad ....(9)$$

The distance Xc is given by Xc = tc . u

Where, u = velocity of flow in the stream

The deoxygenation constant K, is obtained by laboratory test or field tests, and varies with temperature as given below:

$$K_T = K_{20}(\theta)^{(T-20)} \qquad \text{....(10)}$$

Where, θ varies with the temperature = 1.056 in general or 1.047 for 20°C to 30°C temperature, and 1.135 for 4°C to 20°C

K = 0.1 to 0.3 for municipal sewage, base 10, (0.23 to 0.70 for base e)

The reoxygenation constant R also varies with the temperature and can be expressed as:

$$R_T = R_{20}(1.024)^{(T-20)} \qquad \text{....(11)}$$

Where, R'/R = 2.303

R = 0.15 to 0.20 for low velocity large stream

= 0.20 to 0.30 for normal velocity large stream

= 0.10 to 0.15 for lakes and sluggish stream

$$\boldsymbol{R_T = R_{20}(1.016)^{(T-20)}} \qquad \textit{(Peavy et al., 1985)}$$

The ratio of R/K (or R'/K') is called the self purification constant fs and it is equal to 0.50 to 5.0.

Problem: 1

A city discharges 20000 m^3/day of sewage into a river whose rate of flow is 0.7 m^3/sec. Determine D.O. deficit profile for 100 km from the following data:

River	Sewage effluent from STP
5 day B.O.D. at 20°C = 3.4 mg/l	5 day B.O.D. at 20°C = 45 mg/l
Temperature 23°C	Temperature 26°C
D.O. = 8.2 mg/l	D.O. = 2.0 mg/l

Velocity of mix = 0.25 m/sec, R'=0.4, K' = 0.23

Solution:

River discharge $= 0.7\ m^3/sec$,

Sewage discharge $= 20000/(24 \times 3600)$

$= 0.231\ m3/sec$

$$\text{BOD of mix} = \frac{0.7\ X3.4+0.231X45}{0.7+0.231} = 13.72\ mg/L$$

$$\text{Do of mix} = \frac{0.7\ X8.2+0.231X2.0}{0.7+0.231} = 6.6\ mg/L$$

$$\text{Temp. of mix} = \frac{0.7\ X23+0.231X26}{0.7+0.231} = 23.74^{o}C$$

Saturation value of D.O. at $23.74^{o}C$ is 8.57 mg/l

$$\text{Ultimate B.O.D. } Lt = L0\ (1 - e^{-kXt})$$

$$13.72 = L0\ (1 - e^{-0.23X5})$$

$$L0 = 20.08\ mg/L$$

Solution:

River discharge $= 0.7\ m^3/sec,$

Sewage discharge $= 20000/(24 \times 3600)$

$= 0.231\ m3/sec$

$$\text{BOD of mix} = \frac{0.7\ X3.4+0.231X45}{0.7+0.231} = 13.72\ mg/L$$

$$\text{Do of mix} = \frac{0.7\ X8.2+0.231X2.0}{0.7+0.231} = 6.6\ mg/L$$

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Ultimate B.O.D. $Lt = L0\ (1 - e^{-kXt})$

$$13.72 = L0\ (1 - e^{-0.23X5})$$

$$L0 = 20.08\ mg/L$$

Initial D.O. deficit (D0) = 8.57 - 6.66 = 1.91 mg/L

Deoxygenation and reoxygenation coefficients at 23.74°C temperatu

$$K_T = K_{20}\,(\theta)^{(T-20)}$$

Hence, $K_{23.74} = 0.23\,(1.047)^{(23.74-20)} = 0.273\ \text{day}^{-1}$

$$R_T = R_{20}\,(\theta)^{(T-20)}$$

Hence, $R_{23.74} = 0.40\,(1.016)^{(23.74-20)} = 0.424\ \text{day}^{-1}$

Critical time $t_c = \dfrac{1}{R'-K'} \log_e \dfrac{R'}{K'} \left[1 - \dfrac{Do\,(R'-K')}{K'Lo}\right]$

$$= \frac{1}{0.424-0.273} \log_e \frac{0.424}{0.273} \left[1 - \frac{1.91(0.424-0.273)}{0.273 \text{ x } 20.08}\right]$$

$$= 2.557 \text{ days}$$

Critical D.O deficit, $Dc = \dfrac{K'}{R'}\,Lo.\,e^{-K'tc}$

$$= \frac{0.273}{0.424}\,20.08.\,e^{-0.273 \text{ X } 2.557}$$

$$= 6.432 \text{ mg/L}$$

Distance at which it occurs = L= velocity x time

$$= (0.25 \text{ m/sec}) \times (2.557 \times 24 \times 60 \times 60 \text{ sec})$$

$$= 55231 \text{ m} = 55.23 \text{ km}$$

Similarly time required for mix to reach at 20 km distance,

$$t_{20} \text{ km} = \frac{20 \text{ X } 1000}{0.25\text{X } 24\text{X}3600}$$

$$= 0.926 \text{ day}$$

And DO deficit at 20 km can be calculated using equation 4

$$Dt = \frac{K'Lo}{R'-K'}\left[e^{-K't} - e^{-R't}\right] + Do.\, e^{-R't}$$

Where, K' = 0.273 d^{-1}, R' = 0.424 d^{-1}, Do = 1.91 mg/L and Lo = 20.08 mg/L and t = 0.926 day

Hence, DO deficit at 20 km = 4.970 mg/L

Similarly DO deficit at 40 km (i.e. t = 1.852 days) = 6.211 mg/L

DO deficit at 80 km (i.e., t = 3.704 days) = 6.056 mg/L

DO deficit at 100 km (i.e., t = 4.63 days) = 5.427 mg/L

The DO deficit at different points along length of river is as below:

Distance in km	Time in days	DO deficit,mg/L	DO,mg/L
0	0	1.91	6.66
20	0.926	4.97	3.6
40	1.852	6.211	2.359
55.23	2.557	6.432	2.138
80	3.704	6.056	2.514
100	4.63	5.427	3.143

DISPOSAL ON LAND

When the wastewater, either raw or partly treated, is applied or spread on the surface of land, the method is called disposal by land treatment. Some part of the wastewater evaporates while other part percolates in the ground leaving behind suspended solids which are partly acted upon by the bacteria and partly oxidised by exposure to atmospheric actions of air, heat and light. The sewage adds to the fertilising value of the land, and crops can be profitably raised on such land. Due to this, the disposal by land treatment is also sometime called as sewage farming.

4.6.1 Conditions favorable for sewage farming

The effluent irrigation method for disposal of sewage can be favorably adopted under the following conditions:

1. When natural rivers or streams are not located in the vicinity, land treatment is the only alternative.
2. When rivers run dry or have a very small flow during summer, discharging sewage into them is out of question.
3. When plentiful land with sandy, loamy or alluvial soil overlying soft Murom, sand or gravel is available, land treatment is favored. Such soils are easily aerated and it is easy to maintain aerobic conditions in them.
4. When climate is arid, land treatment is favored.

Conditions favorable for sewage farming

The effluent irrigation method for disposal of sewage can be favorably adopted under the following conditions:

1. Land treatment is favored when subsoil water table is low even during the wet season.
2. Land treatment is favored when rainfall is low and there is an acute demand for irrigation water.
3. When large open areas in the surround locality are available, broad irrigation by sewage can be easily practiced.
4. Cash crops can be easily grown on sewage farms.

Methods of applying sewage effluents to farms

The sewage effluents can be used for irrigating farms exactly in the same manner as irrigation water is used for farming. The various techniques that are employed for irrigating crops are:

1. Surface irrigation called broad irrigation: In this method, sewage is applied in different ways, on to the surface of the land. Depending upon the mode of application, it can be of different types such as:

1. Free flooding
2. Border flooding
3. Check flooding
4. Basin flooding and
5. Furrow irrigation method

2. Sub-surface irrigation: In this method, sewage is supplied directly to the root the zone of crops, through a system of properly laid open-jointed pipes. Sewage, as it flows through these pipes, percolates through the open joints, and is distributed in the surrounding area by the action of capillarity.

3. Sprinkler or spray irrigation: In this method, sewage is spread over the soil through nozzles, which are fitted at the tips of pipes carrying sewage under pressure. The process, being costly, is not preferred in India, although it gives very good results, like those of a natural rainfall.

Sewage sickness

When sewage is applied continuously on a piece of land, the soil pores or voids may get filled up and clogged with sewage matter retained in them. The time taken for such a clogging will, of course, depending upon the type of soil and the load present in sewage. But when once these voids are clogged, free circulation of air will be prevented, and anaerobic conditions will develop within the pores. Due to this, the aerobic decomposition of organic matter will stop, and anaerobic decomposition will start. The organic matter will thus, of course, be mineralised, but with the evolution of foul gases like hydrogen sulphide, carbon dioxide and methane. The land is unable to take any further load of sewage. This phenomenon of soil is known as sewage sickness of land.

Sewage sickness

Sewage sickness can be prevented by adopting the following measures.

1. Pretreatment of sewage: By giving primary treatment to the sewage, the suspended solids are removed. Due to this measure, the pores of the soil will not get clogged quickly. Also, BOD load will be reduced by about 30%.

2. Provision of extra land: Extra land, as reserve or stand by should be available so that the land with sewage sickness can be given the desired rest. During the rest period, the sick land should be properly ploughed so that it is broken up and aerated..

3. Under drainage of soil: Subsoil drains should be provided to collect the percolating effluent. This will minimise the possibility of sewage sickness.

4. Proper choice of land: The land chosen for this purpose should be sandy or loamy, having higher permeability. Clayey soil should be avoided.

5. Rotation of crops: Rotation of crops minimise the chances of sewage sickness.

6. Shallow depth application: Sewage should be applied in shallow depths. If sewage is applied in greater depths, chances of sewage sickness are increase.

Comparison of disposal methods

1. For disposal by dilution, large volumes of natural waters, either in the form of stream or lake or sea, are required, while for disposal by land application, large area of pervious land is required.

2. In urban areas, the cost of land is quite high and hence disposal by land application may be costly. If such areas are situated in close vicinity of natural waters, disposal by dilution may be preferred.

3. In rural areas, the cost of land is relatively less. At the same time, the rate of water supply is generally less, resulting in concentrated sewage. For such cases, therefore, disposal through land application may be preferred.

Comparison of disposal methods

4. Disposal by land application, in general, requires efficient land management, Dilution method, however, does not require such efficient management.

5. In disposal by land application, sewage can be applied either raw or after primary treatment, while in disposal by dilution, efficient pretreatment, to meet the efficient quality requirements, is must.

6. In disposal by land application, natural gradients may not be available, necessitating high head pumping, while in dilution methods, such pumping may not be required because rivers/streams flow through lowest contours of the area.

Comparison of disposal methods

7. In hot climates, DO content of natural waters is low and the quantity of water flow is also less, resulting in pollution hazards if high degree of pretreatment is not applied to the effluents before discharging them. In such cases, disposal by land application may be preferred.

8. If the receiving waters are used as source for water supply at the downstream side, disposal by dilution cannot be adopted unless high degree of pretreatment is applied to the effluents.

9. In disposal by dilution, the recreational use of natural waters will vanish completely.

Comparison of disposal methods

10. Disposal by land application especially through sewage farming will increase the yield of crops etc., but it should be practiced with caution since it may cause serious health hazards. Disposal by land application may also pollute ground water.

A comminutor consists of a vertical revolving drum-screen with 6 mm to 10 mm slots. The course material is cut by cutting teeth and the shear bars on the revolving drum as solids are carried past a stationary comb. The small sheared particles then pass through the slots of the drum out and of a bottom opening through an inverted siphon.

Barminutor: A barminutor is another comminuting unit which is used for flows exceeding 50 MLD. It is a combined screening and cutting machine consisting of a bar screen of special design with small openings, along with rotating cutters comprising a comminuting unit which travels up and down the screen cutting the retained solids.

Grit Chambers

Grit is the heavy mineral material found in raw sewage, and it may contain sand, gravel, silt, cinders, broken glass, small fragments of metal and other small inorganic solids, that have subsiding velocities or specific gravities greater than those of organic putrescible solids in wastewater. Grit also includes egg shells, bone chips, coffee grounds, seeds and large organic particles such as food wastes. Grit has a specific gravity ranging from 2.4 to 2.65.

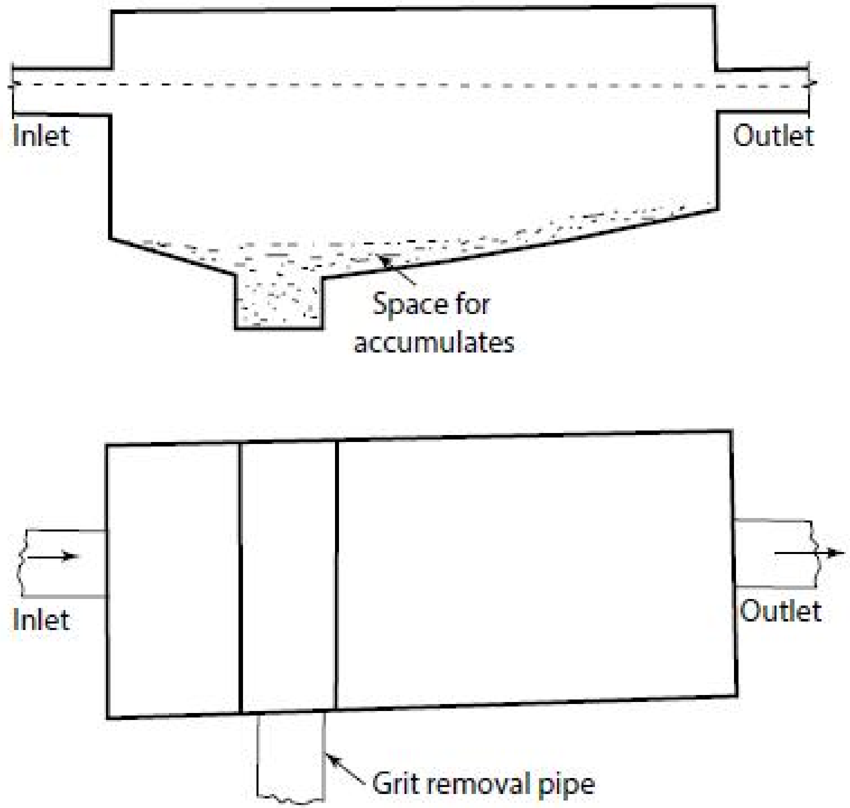


Figure 7.5 Grit Chamber

Figure 5.5 Grit Chamber

Grit chambers are provided to protect moving mechanical equipment from abrasion and accompanying abnormal wear. Grit chamber is an enlarged channel or a long basin, in which the cross-section is increased, so as to reduce the flow velocity of sewage to such that the heavy inorganic materials do settle down by gravity, and the lighter organic materials remain in suspension.

The important point in the design of the grit basin is that the flow velocity should neither be too low as to cause the settling of lighter organic matter, nor should it be so high as not to cause the settlement of the entire silt and grit present in sewage.

The settling velocity is given by stock's law, applicable for particles diameter less than 0.1 mm.

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

$$\upsilon_s = \frac{g}{18}\ \frac{S_s-1}{\upsilon}\ d^2$$

Where,

υ_s	=	Settling velocity
d	=	Size of particle in cm.
μ	=	Dynamic viscosity of liquid in centipoise
υ	=	$\frac{\mu}{\rho}$ = kinamatic viscosity in centistocks
ρ_s	=	Mass density of liquid (gm/cm^3)
ρ	=	Mass density of particle (gm/cm^3)
S_s	=	$\frac{\rho_s}{\rho}$ Specific gravity of particles
g	=	Acceleration due to gravity (cm/sec^2)

The relationship between settling velocity, size and density, of particle, density and temperature of liquid is given by Hazen's modified equation.

$$\upsilon_s = 60.6\,(S_s - 1)\,d\,\left[\frac{3t+70}{100}\right]$$

υ_s = Settling velocity in cm/sec.

t = temperature of liquid in$^{\circ}$C

Critical velocity is given by the modified Shield's formula taking the multiplying factor as 4 is,

$$Vc = 4\sqrt{g(Ss - 1)d}$$

Problem 5.2

A grit chamber is designed to remove particles with a diameter of 0.2mm and specific gravity 2.65, for an average working temperature of 20°C. A flow through velocity of 0.25m/sec. will be maintained by providing a proportional flow weir. Determine the channel dimension for a maximum wastewater flow of 12000m^3/day.

Solution:

Maximum wastewater flow = 12000 m^3/day

$$= \frac{12000}{24X60X60}$$

$$= 0.139 \text{ m}^3/\text{sec.}$$

Flow through velocity $= 0.25$ m/sec.

$\therefore$ Cross sectional velocity $= \dfrac{0.139}{0.25} = 0.556 \text{ m}^2$

Provide a width of 1 mtr.

$\therefore$ Liquid depth $= \dfrac{0.556}{1} = 0.556$ mtr.

Let us provide a free board of 0.3 m, and a depth of 0.2 m for sludge accumulation.

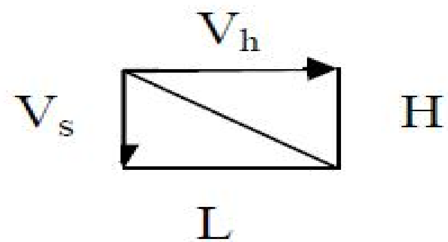
$\therefore$ Total depth $= 0.556+0.3+0.25 = 1.106$ mtr.

Hence keep a depth of 1.15mtr.

Settling velocity, $\upsilon_s = 60.6\,(S_s - 1)\,d\,[\frac{3t+70}{100}]$

$= 60.6\,(2.65 - 1)\,0.02\,[\frac{3X20+70}{100}]$

$= 2.6$ cm/sec.



Now, the ratio

$$\frac{H}{L} = \frac{Vs}{Vh}$$

$$\frac{1.15}{L} = \frac{2.6}{25}$$

$\therefore$ L $= 11.06$ mtr.

This is the theoretical length. Let us make allowance of 25% for inlet and outlet zone.

Hence total length $= 1.25 \text{ X } 11.06$

$= 13.83$ mtr.

Say = 14 mtr.

Hence dimension of grit chamber is **14 x 1.0 x 1.15** mtr.

DETRITUS TANK

A detritus tank is a grit-removal unit which also removes silt as well as some organic matter along with it. This is because the flow through velocity is less and detention time is more in a detritus tank. The main idea of installing a detritus tank is therefore, to remove finer particles than those moved by grit chamber.

A detritus tank is a continuous flow settling tank of rectangular or square shape. The sides of tank are vertical and they are tapered at the bottom so as to form trough for the collection of detritus, which is a mixture of grit, silt and organic solids.

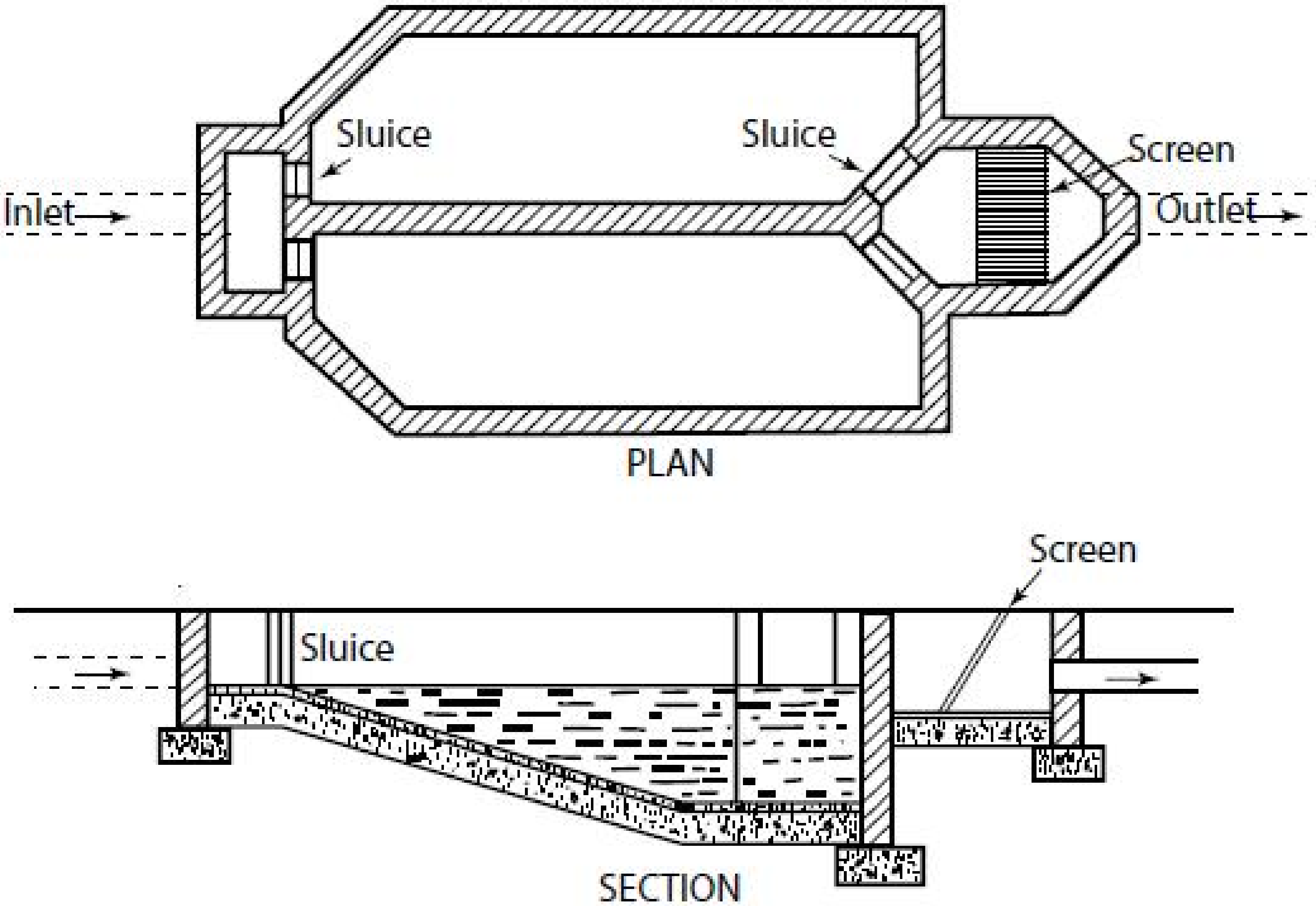


Figure 7.6 Detritus tank

The light organic matter can be washed out of the detritus by the following methods:

1. By passing compressed air through the deposited detritus.
2. By washing it with water.
3. Placing the detritus on a conveyer, and passing the conveyer through water such that the fine organic matter is flushed back into the sewage.

Problem 5.3

Design a suitable detritus tank for a sewage treatment plant receiving a DWF of 500 lps from a separate sewage system. Provide a flow velocity of 0.2 m/sec. and a detention time of 2 minutes. Make other suitable assumptions.

Solution:

Assume maximum flow = 3 times DWF

Hence $Q_{max.}$ = 3 x 500 ltrs/sec.

Let us provide 3 tanks attached and running parallel to each other. Hence design discharge for each tank is

$$Q = 500 \text{ l/s}$$
$$= 0.5 \text{ m}^3/\text{sec.}$$

$$\therefore \text{Cross-section area required } = \frac{Q}{V} = \frac{0.5}{0.2} = 2.5 \text{ m}^2$$

Let us provide a water depth of 1.2 m, in the rechargeable portion.

$$\therefore \text{Width of tank} = \frac{2.5}{1.2} = 2.08\text{m}$$

Say 2.1 mtr.

Also, length of tank = Velocity x detention time

= 0.2 x (2.5 x 60)

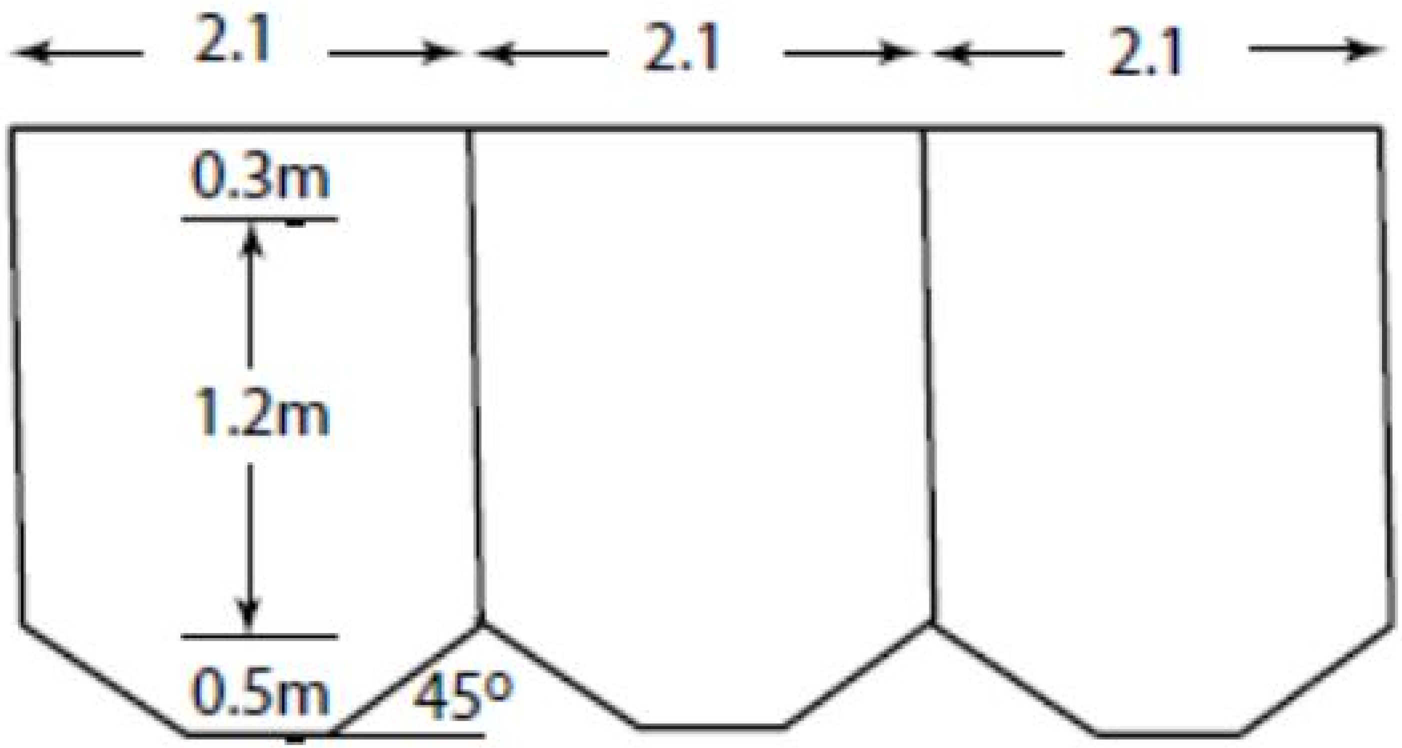
= 30 mtr.

This is the theoretical length; Let us make an allowance of 25% for inlet and outlet zone.

Hence total length = 1.25 x 30

= 37.5 say 38 mtr.

Thus, each unit of the detritus tank will be of 1.5 m width and 38 m length. Provide a free board of 0.3 m; also provide a bottom depth of 0.5 m for the accumulation of detritus and this depth be tapered at an angle of 45°.



Skimming Tanks

Skimming tanks are installed, just ahead of sedimentation tanks to remove floating substances like grease, oil, fats, waxes, soap, free fatty acids, pieces of cork and wood, fruits skins and vegetable debris etc. Much of these oily, greasy substances enter the sewers from kitchens of restaurants and houses, motor garages, oil refineries, soap and candle factories etc. If these are not removed, they seriously affect the working of various treatment units. Skimming tank is a chamber so arranged that floating matter rises and remains on the surface of wastewater unit removed, while the liquid flows out continuously through outlet located at depth, or under portions, curtain walls, deep scum board etc. Most skimming tanks are rectangular or circular, having a detention time of 3 to 5 minutes.

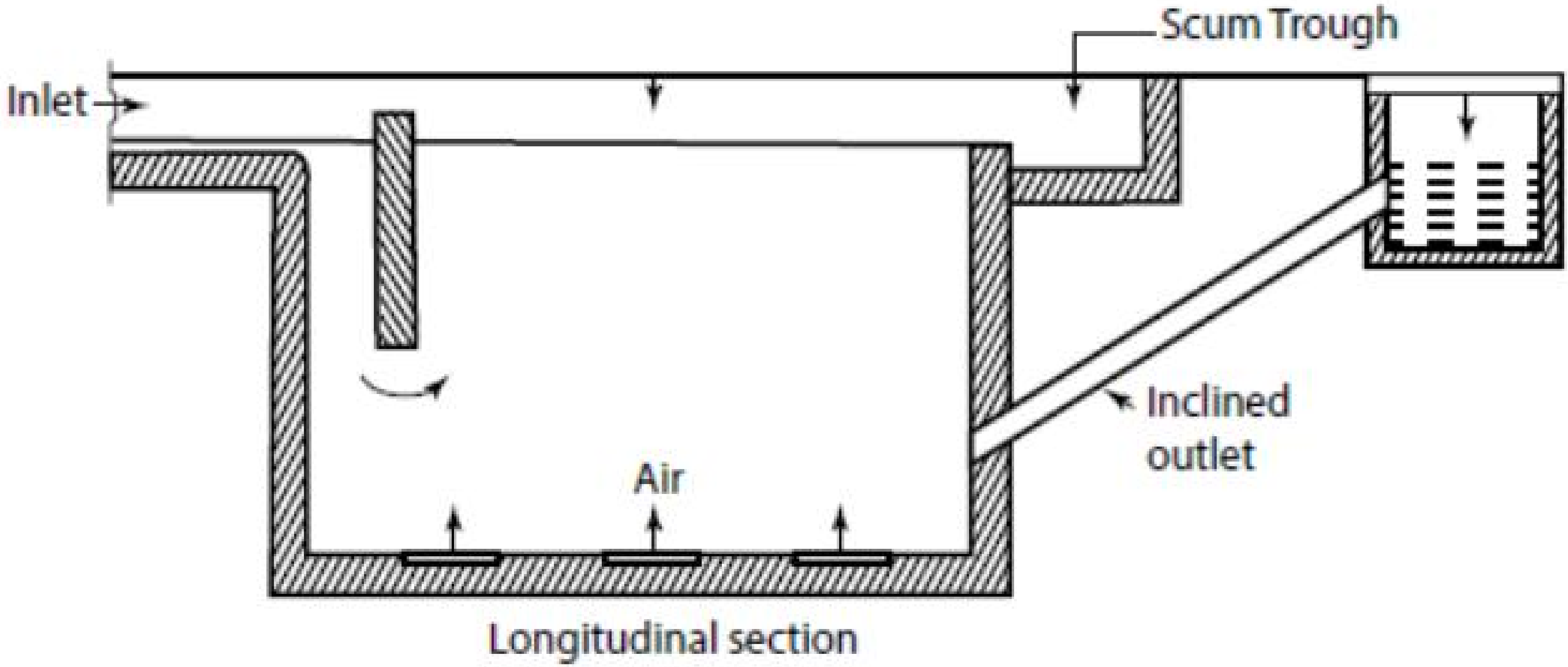


Figure 7.7 Skimming tank

Skimming Tanks

Figure shows a long trough-shaped skimming tank; divide into two or three lateral compartments by vertical baffles having slots for a short distance below the wastewater surface, permitting oil and grease to stilling compartments. For efficient working of skimming tank, air diffusers are provided at the bottom of the tank. The rising air bubbles congeal the greasy and oily material and push it to the side compartment (Stilling chambers). The amount of air required is about 0.2 m^3/m^3 of sewage. The floating matter can be removed either manually or with the help of some mechanical equipment. The volume of skimmings may vary from 0.625 to 37.5 m^3 per million cubic meter of sewage.

Vacuum floatation: Grease, oils etc. can be removed by subjecting the aerated sewage to vacuum pressure of about 0-25cm of mercury for 10 to 15 min. in a vacuator and this processes is known as vacuum floatation. The vacuum pressure cause the air bubbles lift the grease and move upwards. The rising bubbles lift the grease and other lighter matter to the surface.

Disposal of skimming sedimentation: The skimmings should be disposed of by either burial in low laying areas or by burning or by digestion together with sludge. It may also be converted into soap lubricants, pith candles, and other non-edible products. The digestion of skimmings having predominance of vegetable and organic matter produces gases of high fuel value.

SEDIMENTATION TANKS

Effluent of the grit chamber, containing mainly light weight organic matter, is settled in the primary sedimentation tanks. The objective of treatment by sedimentation is to remove readily settleable solids and floating material and thus to reduce the suspended solids content when they are used as preliminary step to biological treatment, their function is to reduce the load on the biological treatment units..

SEDIMENTATION TANKS

The primary sedimentation tanks are usually designed for a flow through velocity of 1 cm/sec at average rate of flow. The detention period in the range of 90 to 150 minutes may be used for design. These tanks may be square, circular, or rectangular in plan with depth varying from 2.3 to 5 m. The diameter of circular tanks may be up to 40 m. The width of rectangular tank may be 10 to 25 m and the length may be up to 100 m..

SEDIMENTATION TANKS

But to avoid water currents due to wind, length is limited up to 40 m. The slope of sludge hoppers in these tanks is generally 2:1 (vertical: horizontal). The slope of 1% is normally provided at the bed for rectangular tanks and 7.5 to 10% for circular tanks. This slope is necessary so that solids may slide to the bottom by gravity.

PLAIN SEDIMENTATION.

When the impurities are separated from suspending fluid by the action of natural forces alone; ie. by gravitation and natural aggregation of the settling particles, the operation is called plain sedimentation.

SEDIMENTATION WITH COAGULATION

Very fine colloidal particles cannot come together and get settle down by the above treatment since they have negative charge associated with them, which impart stability to the colloidal system formed. Hence they are removed by addition of chemicals called coagulants to water before sedimentation.

SEDIMENTATION WITH COAGULATION

These on dissolution in water produce cations which can neutralize the negative charge of the particles and hence from an insoluble gelatinous flocculent precipitate, which adsorb and forming bigger floc and this get settle down easily under the force of gravity. Commonly used coagulants are alum, sodium aluminate and ferrous sulphate. Alum works in alkaline range where as acidic range is suitable to sodium aluminate.

THEORY OF SEDIMENTATION

The process of settlement of a particle is obstructed or opposed by the following three forces.

The velocity of flow

The velocity of flow carries the particle horizontally. The greater the flow area, the lesser is the velocity and hence more easily the particle will settle down.

The viscosity of water in which the particle is travelling

The viscosity varies inversely with temperature. Warm water is less viscous and, therefor, offers less resistance to settlement.

The size, shape and specific gravity of the particle

The greater is the specific gravity, more readily the particle will settle. The size and shape of the particle also affect the settling rate..

TYPES OF SETTLINGS

Particles may settle out of a suspension in the following four ways, depending upon the concentration of the suspension and the flocculating properties of particles (Fig.6.1).

Type I: Discrete particle settling

Type II: Flocculent Particles settling

Type III: Hindered or Zone settling

Type IV: Compression settling

TYPES OF SETTLINGS

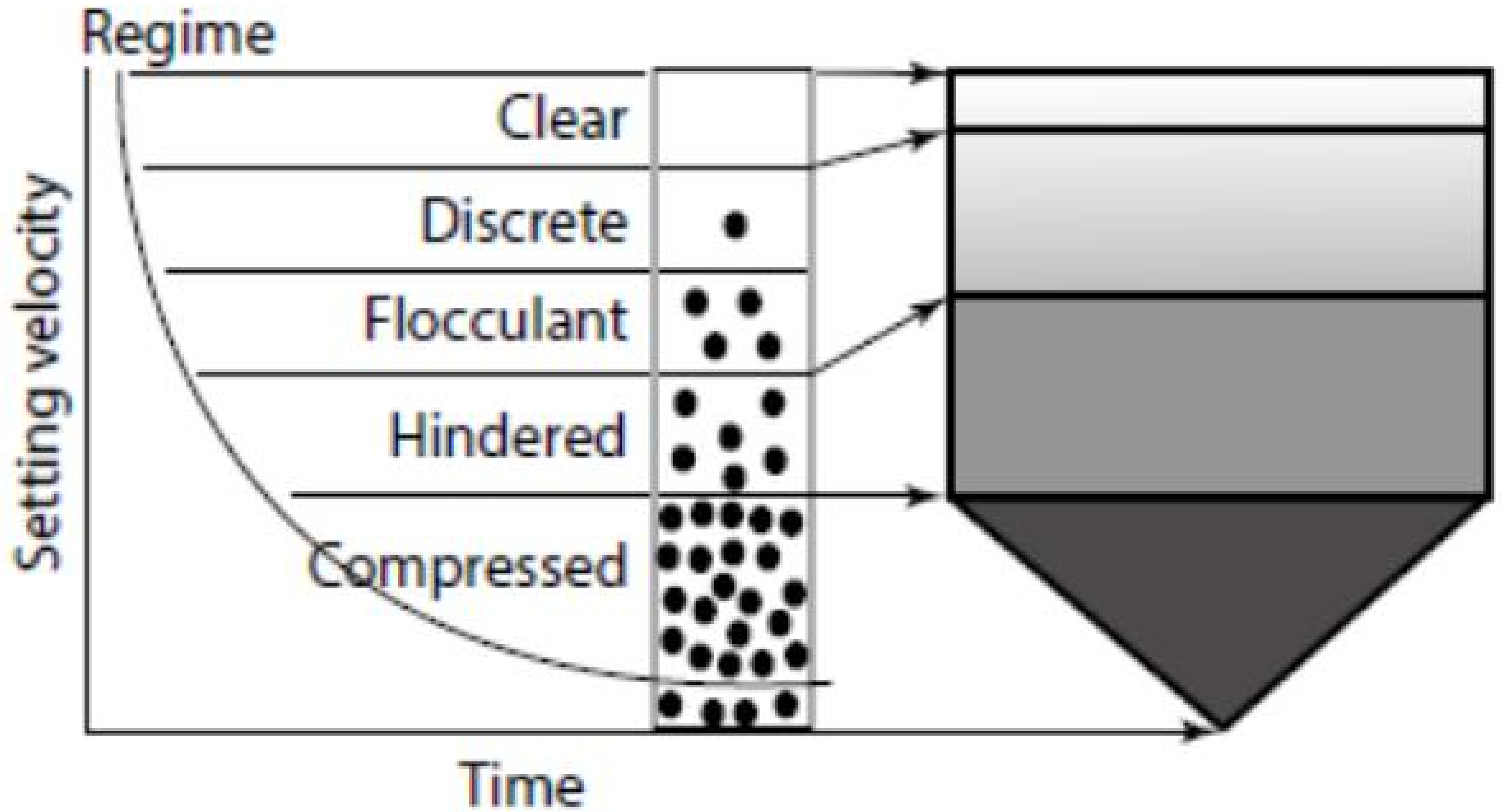


Fig. 8.1 Types of settling

TYPES OF SETTLINGS

1. Type I: ***Discrete particle settling*** - Particles settle individually without interaction with neighboring particles.
2. Type II: ***Flocculent Particles*** – Flocculation causes the particles to increase in mass and settle at a faster rate.
3. Type III: ***Hindered or Zone settling*** –The mass of particles tends to settle as a unit with individual particles remaining in fixed positions with respect to each other.

TYPES OF SETTLINGS

4. Type IV: ***Compression*** – The concentration of particles is so high that sedimentation can only occur through compaction of the structure.

In water treatment, only type I and type II settling are encountered

Type I Settling

Type I sedimentation is concerned with the settling/removal of none flocculating, discrete particles from water. In this type of settling the size, shape and specific gravity of the particles do not change with time and settling velocity remains constant. When a discrete particle is placed in a quiescent fluid, it will accelerate until the frictional resistance F_D of the fluid equals the impelling force F_I acting on the particle. At this stage, the particle attains a uniform or terminal velocity and settles down with this constant velocity known as settling velocity.

The impelling force (F_I) is evidently equal to the effective weight of the particle:

$$F_I = (\rho_s - \rho) g V \qquad ....(1)$$

Where,

ρ_s= Mass density of particle

ρ= Mass density of fluid

g = Acceleration due to gravity

V= Volume of Particle, $\frac{\pi}{6} d^3$, Where d is the diameter of spherical particle

(a) Newton's Law

The drag force F_D depends on

i) Dynamic viscosity μ

ii) Mass density ρ of the fluid and

iii) Shape and size of the particle

The drag force is given by Newton's law for frictional drag in the following form:

$$F_D = C_D . A . \frac{\rho V_s^2}{2} \qquad ....(2)$$

Where, F_D = Drag force

C_D = drag coefficient

A = projected area of the particle = $\frac{\pi}{4} d^2$.

Equating the two, we get an equation for the settling velocity in the form

$$V_s = \sqrt{\frac{4g(\rho_s - \rho)d}{3C_D \rho}} \qquad ....(3)$$

or closely,

$$V_s = \sqrt{\frac{4g(S_s - 1)d}{3\, C_D}} \qquad ....(4)$$

Where, S_S = specific gravity of the particle.

Above equations requires the determination of drag coefficient C_D which is related to Reynolds number R.

(a) Newton's Law

The drag force F_D depends on

i) Dynamic viscosity μ

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Where, S_S = specific gravity of the particle.

Above equations requires the determination of drag coefficient C_D which is related to Reynolds number R.

Type II Settling

Type II settling is the settling of particles that flocculate as they settle. The process flocculation produces larger particles and settles at a faster rate. Settling of particles in coagulation cum sedimentation tank is an example for type II settling. No mathematical relationship exists that can readily be used to analyse flocculent settling..

Type II Settling

In discrete settling overflow rate is an important parameter. But in flocculent settling, in addition to overflow rate, other necessary design parameters are depth, detention time and effluent weir flow rate. If any two of the three parameters of depth, detention time or overflow rates are set, the third is established automatically. The relationship is obvious if the overflow rate is considered as velocity. The effluent weir flow rate is usually limited by standards or design manuals, although there is little evidence that these standards are realistic.

SEDIMENTATION TANKS

The first stage of treatment is the pre-filtration of water and it includes provision of sedimentation tanks or settling tanks or clarifiers. These tanks remove inorganic impurities and make water fit for the next process of filtration.

Classification of sedimentation tanks

Depending upon the nature of working, the sedimentation tanks are of the following two types.

Fill and draw types tanks

Continuous flow type tanks

Fill and draw type tanks

These are also known as intermittent type sedimentation tanks. The water is filled in the tanks and it is then allowed to rest for a certain time. During the period of rest, the particle in suspension will settle down at the bottom of tank. The clear water is then drawn off and tank is cleaned of silt and filled again. Generally, a detention time of 24 hours is allowed. At the end of the period, the clear water drawn off through the outlet valve. The plan and section of fill and draw type tank is given below.

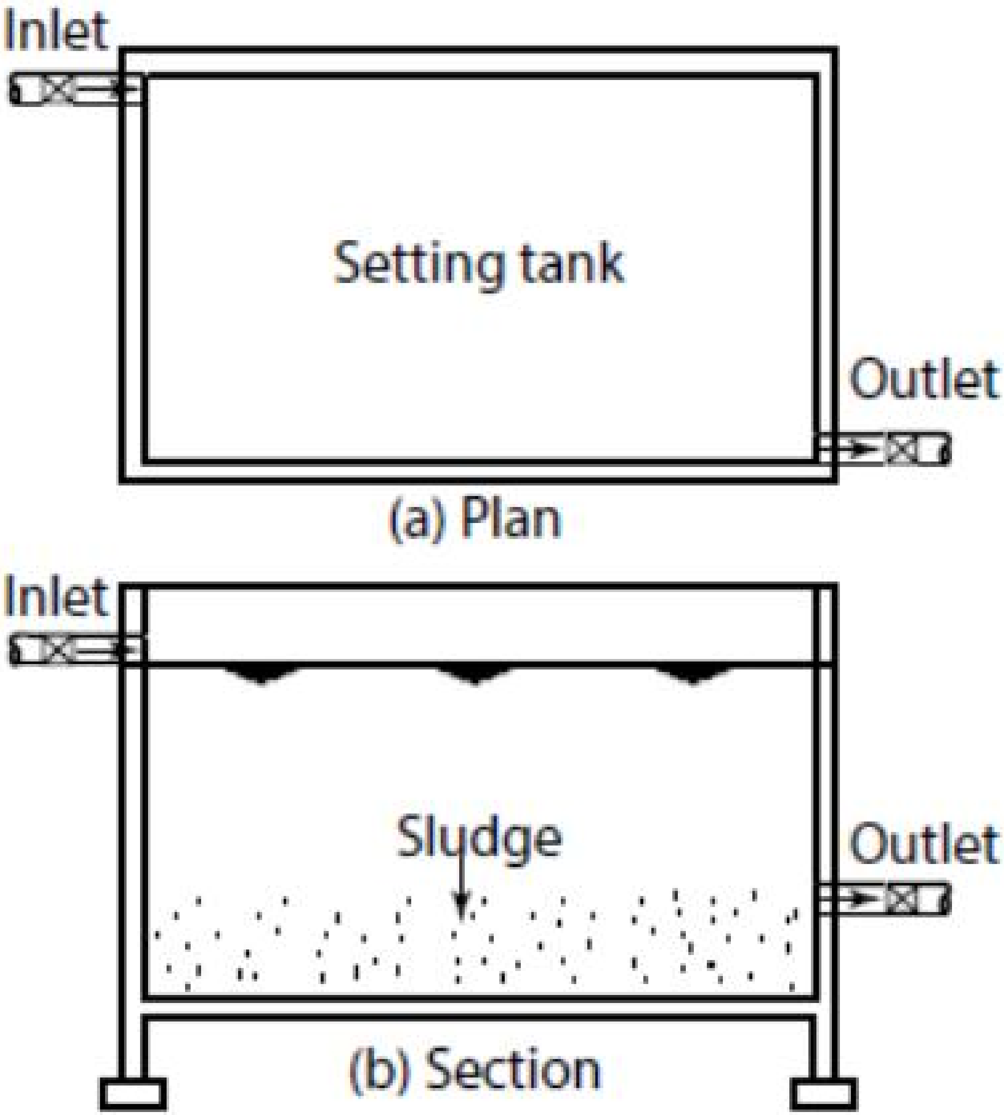


Fig. 8.2 Fill and draw type tank

Continuous flow type tanks

In the continuous flow type, the water continuously keeps on moving in tank, though with a very low velocity during which time the suspended particles settle at the bottom before they reach the outlet. There are two types of continuous flow tanks.

1. Horizontal flow tanks
2. Vertical flow tanks
3.

1.

Horizontal flow tank.

In the horizontal flow type, the tank is generally rectangular in plan having length equal to at least twice the width. The water flows practically in the horizontal direction, with a maximum permissible velocity of 0.3 m/sec. These are further divided in to:

1. Rectangular tanks
2. Circular tanks which are further classified as
3. Radial flow tanks
4. Circumferential flow tanks

Plain sedimentation tanks

1.

Plain sedimentation tanks

Plain sedimentation tanks are usually of the following three types:

1. Rectangular tanks
2. Circular tanks
3. Hopper Bottom tanks

1.

Rectangular sedimentation tank

In this type of tank, its capacity depends upon the volume of water to be treated. The length depends on the velocity of flow and detention period. The detention period may vary from 4-6 hours. The width of the tank varies from 10 m – 12 m, and the depth of the tank varies from 2 m to 4 m.

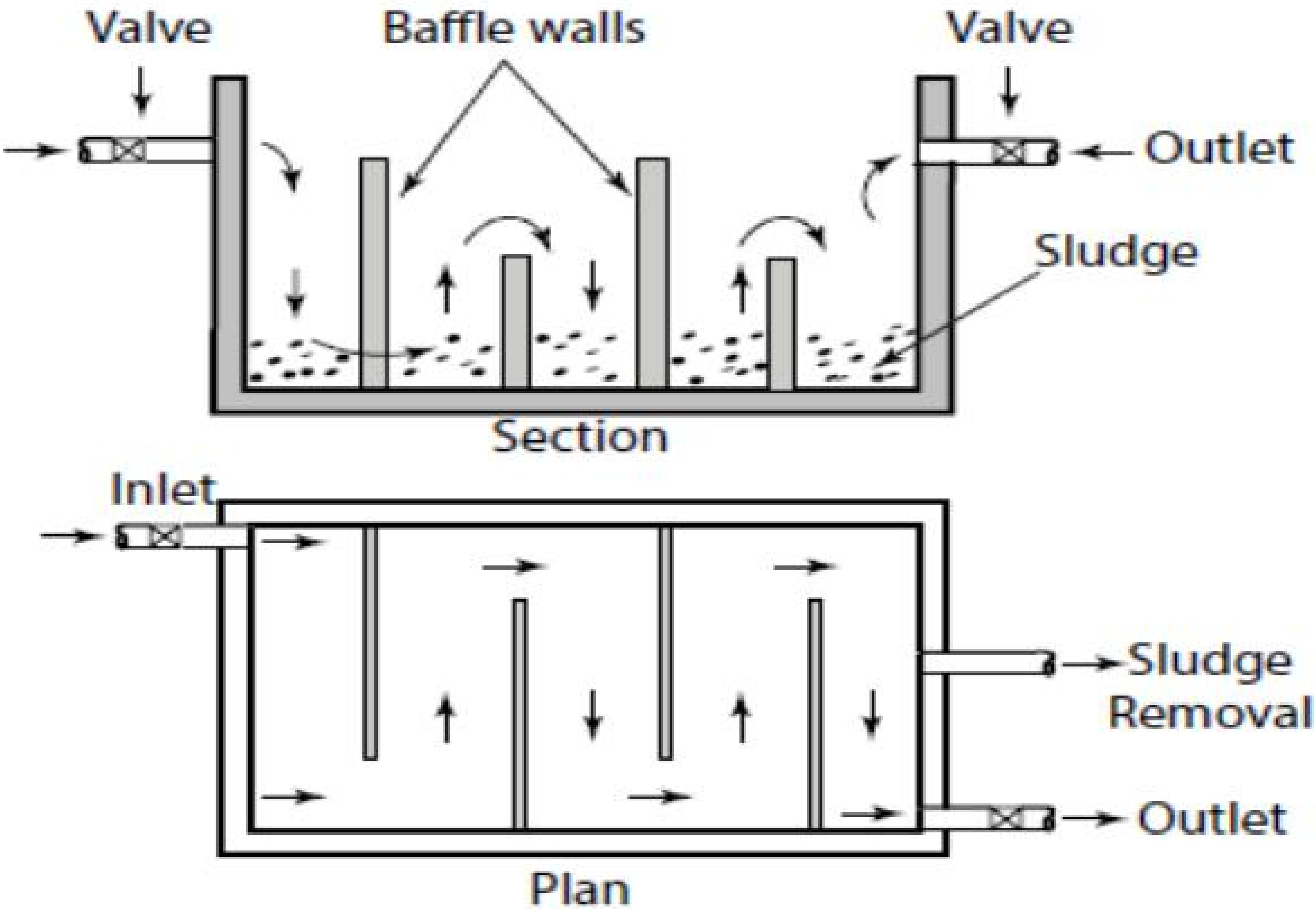


Fig 8.3 Rectangular sedimentation tank

1.

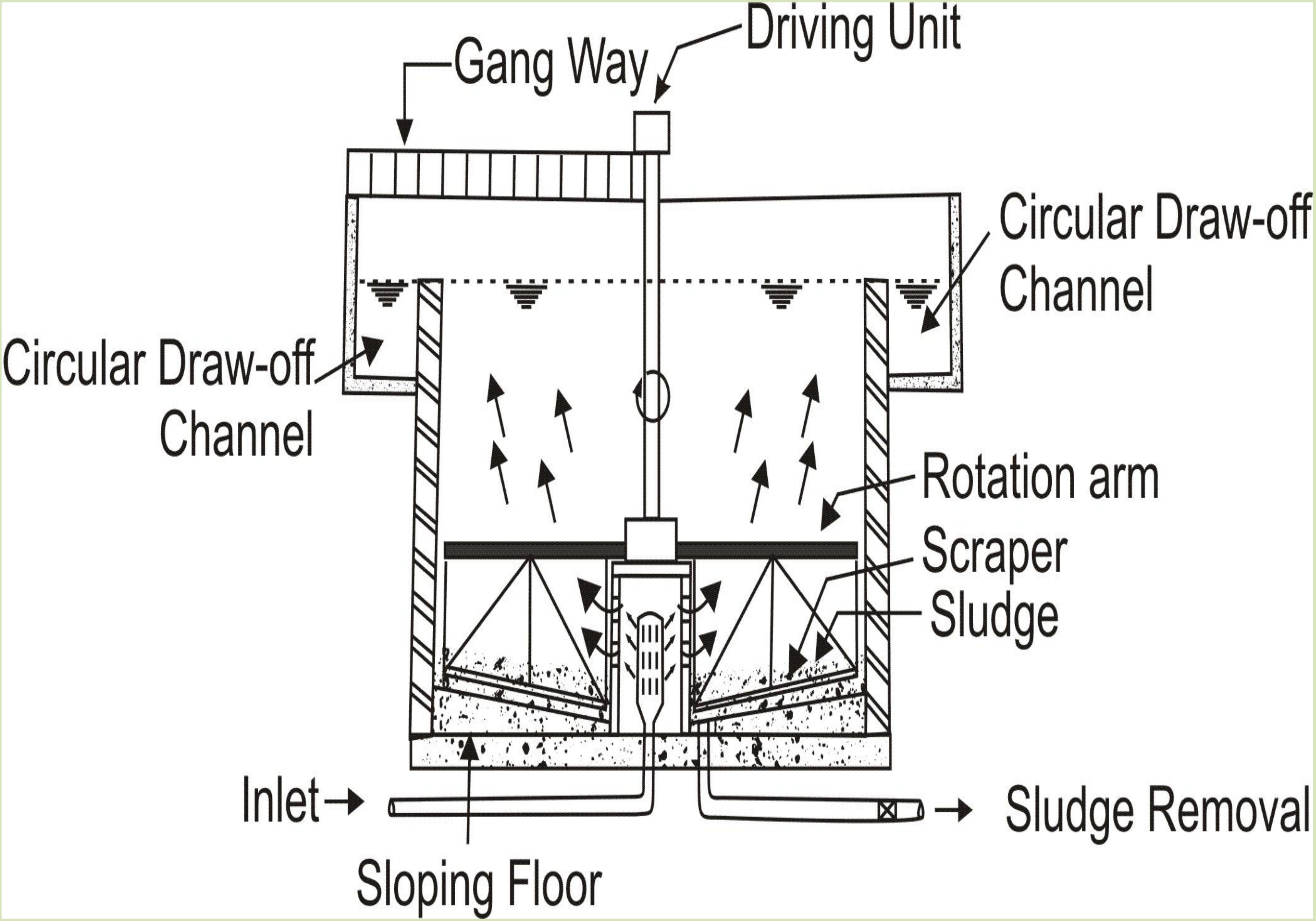
Rectangular sedimentation tank

In this type of tank, its capacity depends upon the volume of water to be treated. The length depends on the velocity of flow and detention period. The detention period may vary from 4-6 hours. The width of the tank varies from 10 m – 12 m, and the depth of the tank varies from 2 m to 4 m.

1.

Circular sedimentation tank

The circular sedimentation tank may be radial or spiral flow. But the tank with radial flow is commonly adopted. In this tank, the water is allowed to enter through the pipe which is provided at its centre. The water flows upwards gently through the openings. The water is collected at the circular draw-off channel from where it is taken to next unit through the outlet pipe as shown in Fig.6.4.


Driving Unit
Gang Way
Circular Draw-off Channel
Circular Draw-off Channel
Rotation arm
Scraper
Sludge
Inlet
Sludge Removal
Sloping Floor

1.

Circular sedimentation tank

The sediments or sludge are settled down at the bottom of the tank. A driving unit is provided for rotating an arm which consists of scrapper. The circular motion of the scrapper helps the sludge to discharge through the sludge removal pipe.

1.

Hopper Bottom sedimentation tank

In this tank, the water is allowed to enter through the centrally placed inlet pipe and is deflected downwards by the action of a deflector box. The water flows downwards inside the box and then it rises in upward direction through the opening between the box and the wall of the tank. When the water rises in upward direction, the particles having specific gravity more than 1.0 cannot follow the path and ultimately settle down at the bottom of the tank due to the property of hydraulic subsidence.

1.

Hopper Bottom sedimentation tank

The sludge settles at the bottom of the hopper, from where it is removed with the help of a sludge pipe connected to a sludge pump.

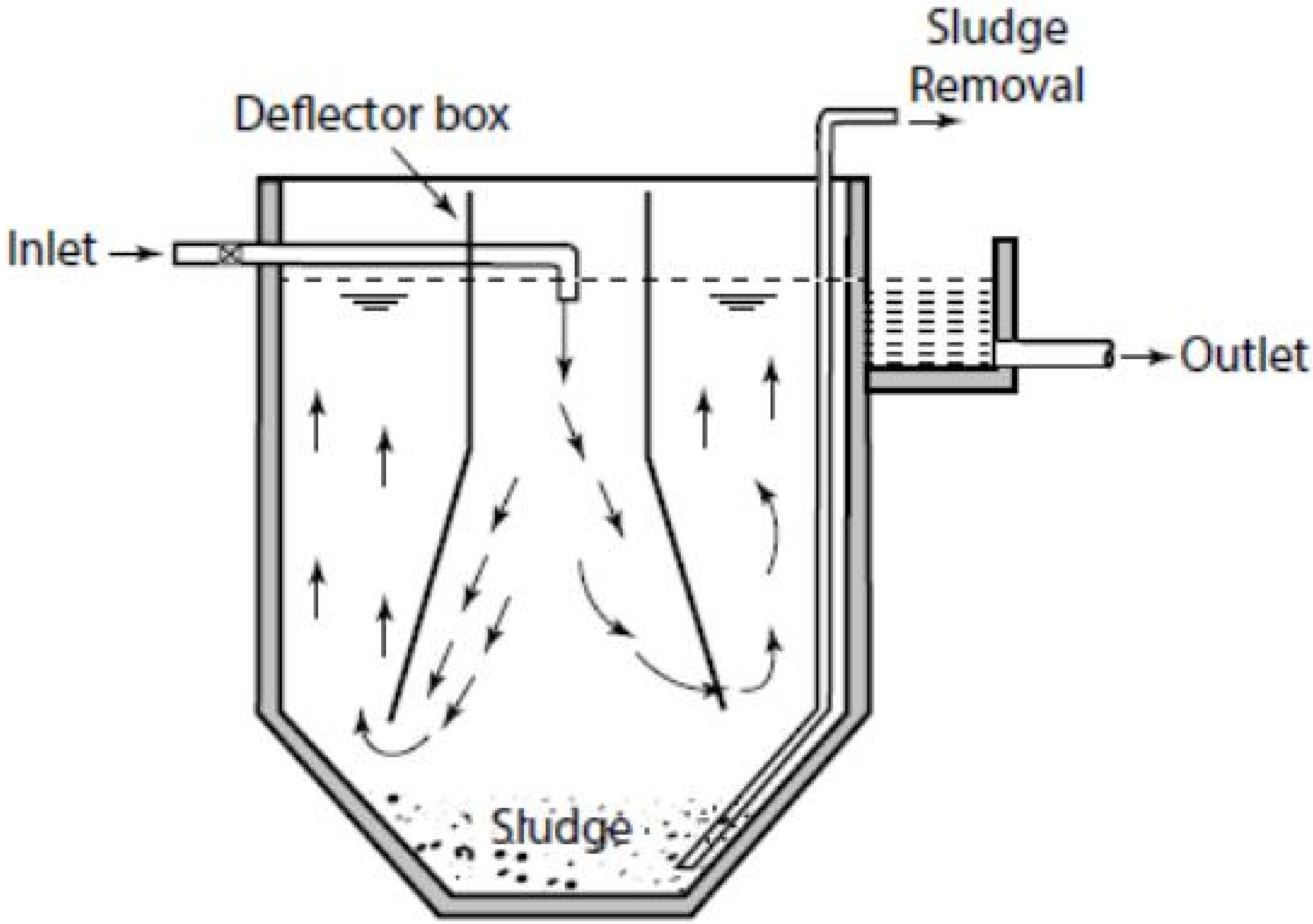


Fig. 8.5 Hopper bottom sedimentation tank

1.

Fig. 6.5 Hopper bottom sedimentation tank

Sedimentation with coagulation (clarification)

In plain sedimentation the heavier particle settle down. However, fine particles take many hours or some days to settle down. Colloidal particles which are of size finer than 0.0001mm carry electrical charges on them. These are continuous in motion and will never settle down under gravity. As it is not possible to have detention period in the sedimentation tank more than 3 to 4 hours, coagulation is adopted.

1.

Clarification

The chemically assisted sedimentation comprises several separate processes of treatment which go to make up the complete system known as clarification. It is achieved in three stages:

1. Addition of coagulants
2. Formation of floc
3. Sedimentation

1.

Clarification

In the first stage, it is usual to introduce chemical at some point of high turbulence in the water. This may be achieved by either passing the water over a weir or through an orifice plate or may be produced by mechanical stirring using a flash 'mixer'. The term coagulation is used to mean the first stage in the formation of precipitate while flocculation consists of building up the particles of floc to larger size which can be removed by sedimentation in the third stage of clarification.

PRINCIPLE OF COAGULATION

The principle of coagulation has been explained by the following phenomenon.

1. Floc formation
2. Electric charges

Floc formation

When a coagulant is added to the water and mixed thoroughly, a thick gelatinous precipitate is formed which is insoluble in water. This precipitate is called floc. As the floc settle down, it attracts and arrests the colloidal particles and brings them down.

1.

Electric charges

It is observed the ions of floc posses positive charge. Colloidal particles are negatively charged ions. So, the floc attracts the colloidal particles while it travels towards the bottom of tank.

1.

Flocculation

Flocculation is essentially an operation designed to force agitation in the fluid and induce coagulation. Basically, flocculation is a slow mixing or agitating process in which destabilized colloidal particles are brought into intimate contact in order to promote their agglomeration

Common Coagulants

The following are the usual chemicals which are commonly used for the coagulation:

1. Aluminum Sulphate or alum
2. Chlorinated copperas
3. Ferrous Sulphate and Lime
4. Magnesium Carbonate
5. Polyelectrolyte and
6. Sodium Aluminate

Aluminum Sulphate (Alum)

The chemical composition of Aluminum Sulphate is $Al_2(SO_4)_3$, 18 H_2O. It is commonly known as alum. It is available in the form of a solid lump, but applied in a powder or liquid form. I t is very effective if bicarbonate alkalinity is present in water. If the water possesses no alkalinity, some amount of lime is to be added to water.

Aluminum Sulphate (Alum) Cont….

When alum is mixed with water, a chemical reaction takes place and aluminum hydroxide ($Al(OH)_3$), calcium sulphate ($CaSO_4$) and carbon Dioxide (CO_2) are formed. The aluminum hydroxide is insoluble in water and it forms the floc. It is effective between pH values 6.5 and 8.5. The dosage of this coagulant depends on various factors such as turbidity, colour, pH-value, etc. In practice the dosage of alum varies from 10 to 30 mg per liter. Alum is preferred over other coagulants because it reduces taste and odour in addition to turbidity in water.

b) Chlorinated Copperas

When chlorine is mixed with the solution of ferrous sulphate, a chemical reaction takes place which forms ferric sulphate [$Fe_2(SO_4)_3$] and ferric chloride [$FeCl_3$]. The combination of these two compounds is known as chlorinated copperas. Both the compounds are effective for the formation of floc. Sometimes, ferric sulphate and ferric chloride may be applied independently with lime. In that case, ferric hydroxide [$Fe(OH_3)$] is formed which is also effective for the formation of floc. The ferric sulphate is effective for pH-value 4 to 9 and ferric chloride is effective for pH-value 3.5 to 6.5.

Ferrous Sulphate and lime

The ferrous sulphate and lime when mixed with water, a chemical reaction takes place and ferrous hydroxide [$Fe(OH)_2$] is formed. This compound is again oxidized by the dissolved oxygen in water and finally ferric hydroxide is formed. This ferric hydroxide forms the floc.

Magnesium carbonate and lime

When magnesium carbonate and lime are dissolved in water, magnesium hydroxide and calcium carbonate are formed. Both these are soluble in water, resulting in the formation of sludge which is in slurry form. Due to this, it is not commonly used.

Polyelectrolytes

Polyelectrolytes are high molecular weight water-soluble polymers. The amount of polyelectrolyte used is very small in reaction to the amount of primary coagulant. The usual dosage of polyelectrolyte is 1ppm..

Sodium Aluminate

Sodium Aluminate $Na_2Al_2O_4$ is alkaline in reaction and is used very much less often than alum because of its cost. This coagulant removes both temporary and permanent hardness, and is effective for a pH range of 6 to 8.5 naturally available in water.

Problem 6.1

Design a primary settling tank of rectangular shape for a town having population of 60,000, with a water supply of 150 litres per capita per day.

Solution:

Assuming that 80% of water supplied to the city is converted into sewage,

$\therefore$ Total sewage flow $= 0.8 \times 60000 \times 150$

$= 7200 \times 10^3$ ltrs/day

$= 7200\ m^3$/day

Let us assume a detention period of 2 hours.

$$\therefore \text{Capacity required} = \frac{7200}{24} \times 2 = 600 m^3$$

Again, let us assume an overflow rate of $30 m^3/d/m^2$ for average flow

$$\text{Surface area} = \frac{7200}{30} = 240\ m^2$$

$$\therefore \text{Effective depth} = \frac{600}{240} = 2.5\ m$$

Again, B x L = $240\ m^2$

Taking L = 4 B

B (4B) = 240

From which, B = 7.75 m

Let us Take B = 7.75 m and L = 31m

Provide 15% extra length for inlet and outlet arrangements.

∴ Total length = 1.15 x 31 =35.65 m

Say 36 mtr.

Also, provide 1.2 m depth for sludge accumulation and 0.3 m as free board. Hence total depth = 2.5 + 1.2+0.3=4 m.

Hence dimension of the sedimentation tank is **36 m x 7.75 m x 4 m.**

MODULE IV

CE 402 ENVIRONMENTAL ENGINEERING – II

MODULE IV

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M. A. College of Engineering
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BIOLOGICAL TREATMENT

The objectives of the biological treatment of wastewater are to coagulate and remove the nonsettleble colloidal solids and to stabilize the organic matter. Biological treatment systems are living systems which rely on mixed biological cultures to break down waste organics and remove organic matter from the solution.

BIOLOGICAL TREATMENT

The objective of the biological treatment of wastewater is to remove organic matter from the wastewater which is present in soluble and colloidal form or to remove nutrients such as nitrogen and phosphorous from the wastewater. The microorganisms (principally bacteria) are used to convert the colloidal and dissolved carbonaceous organic matter into various gases and into cell tissue. Cell tissue having high specific gravity than water can be removed in settling tank.

BIOLOGICAL TREATMENT

Biological processes are classified by oxygen dependence of primary micro-organisms responsible for waste treatment. Based on this, biological processes may

a) Aerobic

b) An aerobic and

c) Aerobic-anaerobic

a) Aerobic Processes

Aerobic processes are those which occur in the presence of dissolved oxygen. The aerobic processes include the following.

1. Trickling filters
2. Activated sludge processes
3. Aerobic stabilization ponds
4. Aerobic lagoons

b) Anaerobic Processes

Anaerobic waste treatment involves the decomposition of organic and/or inorganic matter in absence of molecular oxygen. An anaerobic process consists of the following.

1. Anaerobic sludge digestion
2. Anaerobic contact processes
3. Anaerobic filters
4. Aerobic lagoons and ponds

c) Aerobic-anaerobic Processes

Aerobic-anaerobic Processes are those in which stabilization of waste is brought about by a combination of aerobic, anaerobic and faculative bacteria. Most of the biological treatment processes are preferred to work on aerobic bacterial decomposition because such decomposition does not produce bad smells and gases as produced by anaerobic decomposition, and also because aerobic bacteria are about three times more active than anaerobic bacteria at $30^{o}C$.

BIOLOGICAL TREATMENT TECHNIQUES

The biological treatment techniques used may be classified under the following three heads:

(i) Attached growth processes (or fixed film processes),

(ii) Suspended growth processes, and

(iii) Combined processes.

(a) Attached growth processes (or fixed film processes)

These are the biological treatment processes in which the microorganisms responsible for the conversion of the organic matter or other constituents in the wastewater to gases and cell tissue are attached to some inert medium, such as rock, slag or specially designed ceramic or plastic materials. Such processes include the followings.

1. Intermittent sand filters
2. Trickling filters
3. Rotating biological contactors
4. Packed bed reactors
5. Anaerobic lagoons (ponds)

(b) Suspended growth processes.

These are the biological treatment processes in which the micro-organisms responsible for the conversion of the organic matter or other constituents in the wastewater to gases and cell tissue are maintained in suspension within the liquid in the reactor by employing either natural or mechanical mixing. In most processes, the required volume is reduced by returning bacteria from the secondary clarifier in order to maintain a high solids concentration. The suspended growth processes include the following:

1. Activated sludge processes
2. Aerated lagoons
3. Sludge digestion systems

(a) Combined processes

These consist of both attached growth processes as well as suspended growth processes. They include the following in sequence:

1. Trickling filter, activated sludge
2. Activated sludge, trickling filter
3. Faculative lagoons

It should be noted that various types of biological processes only help in changing the unstable organic matter into stable forms which are then removed in the secondary settling tanks. Thus, in the biological treatment processes, secondary settling tanks are essential. Out of various biological treatment processes enumerated above, sewage filtration and activated sludge processes are the one which are most commonly used.

TYPES OF SEWAGE FILTERS

Common filters, used for sewage filters, are of the following types:

1. Intermittent sand filters
2. Contact beds
3. Trickling filters

INTERMITTENT SAND FILTERS

Intermittent sand filter, falling under the early developments of biological sewage treatment, consists of a layer of sand with an effective size of 0.2 to 0.5 mm, uniformity coefficient of 2 to 5 and depth 75 to 100 cm. Sewage effluents from primary clarifiers is applied by means of a dosing tank and siphon. The effluent then escapes from the side openings of trough and ultimately, flows on surface of sand bed. In order to facilitate the drainage of filtered effluent, a layer of about 15 cm to 30 cm depth of gravel is provided at the bottom of sand layer.

INTERMITTENT SAND FILTERS

To carry off the effluent, open jointed drainage pipes are laid in the gravel layer. The filters are generally rectangular in plan, with length to width ratio between 3 to 4, and area of each unit varying from 0.2 to 0.4 hectares. Usually 3 to 4 beds are provided adjacent to each other, so that they can work in rotation.

Operation: On each bed, sewage effluent from primary settling tank is applied intermittently, through a dosing tank containing a siphon. The flooding is done from 5 to 10 cm in depth after an interval of 24 hours. The sewage effluent then percolates through the sand bed, and in this process, the suspended organic matter gets trapped in the voids of top portion of sand, through straining action. During the rest period, the trapped organic matter is acted upon by aerobic bacteria present in the filter layer. These aerobic bacteria flourish well in the presence of free oxygen available from atmosphere during the rest period when the sewage dose has percolated down.

The rate of filtration depends upon the size of sand. Finer sand will result in better quality effluent but the rate of filtration is less. Coarser sand will permit higher filtration rate, but in that case the penetration of solids will be too deep. Generally, top 15 cm layer is kept of finer variety, than the rest, which is kept of uniform size.

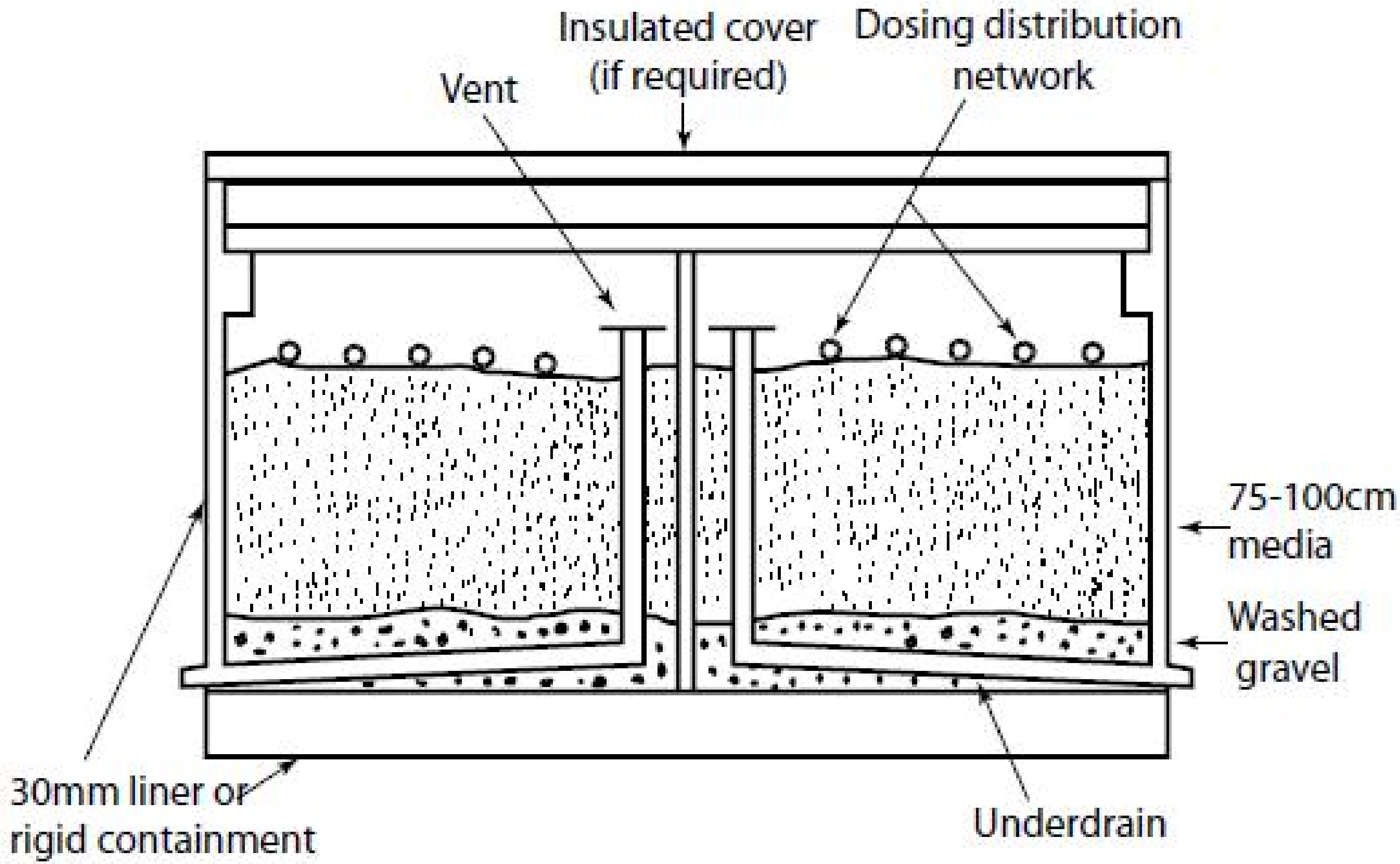


Figure 9.1 Intermittend sand filter.

Figure 7.1 Intermittent sand filter.

Advantages

1. The effluent from intermittent sand filter is of better quality. It is cleaner and more stable, and hence does not need further treatment before disposal.
2. The filter works under aerobic conditions, and hence there is no trouble of odour, flies and insects.
3. The operation is very simple, requiring no mechanical equipment except for dosing.
4. Smaller head is required for applying the sewage on the surface of the filter.
5. There is no secondary sludge which is to be disposed of, except for the occasional sand scraping.

Disadvantages

1. The rate of filtration, and hence that of loading is very small, per unit surface area of the filter. Hence they cannot be employed for medium size or bigger plants.
2. They require large area and large quantity of sand, due to which their construction is very costly.

CONTACT BEDS

Contact beds, also called contact filters, are similar to intermittent sand filters in construction, except that the filtering media is very coarse, consisting of broken stone, called ballast of 20 to 50 mm gauge. A contact bed is a water tight tank of masonry walls and of rectangular shape. The depth of filtering media is kept between 1 to 1.8 m (common depth being 1.2 m). The tank is generally dug below ground level and is lined with water-tight cement plaster, or of concrete surfaces instead of masonry. A siphonic dosing tank is provided to serve two or three contact beds. The effluent from primary settling tank is first received by the dosing tank and then distributed over one contact bed at a time

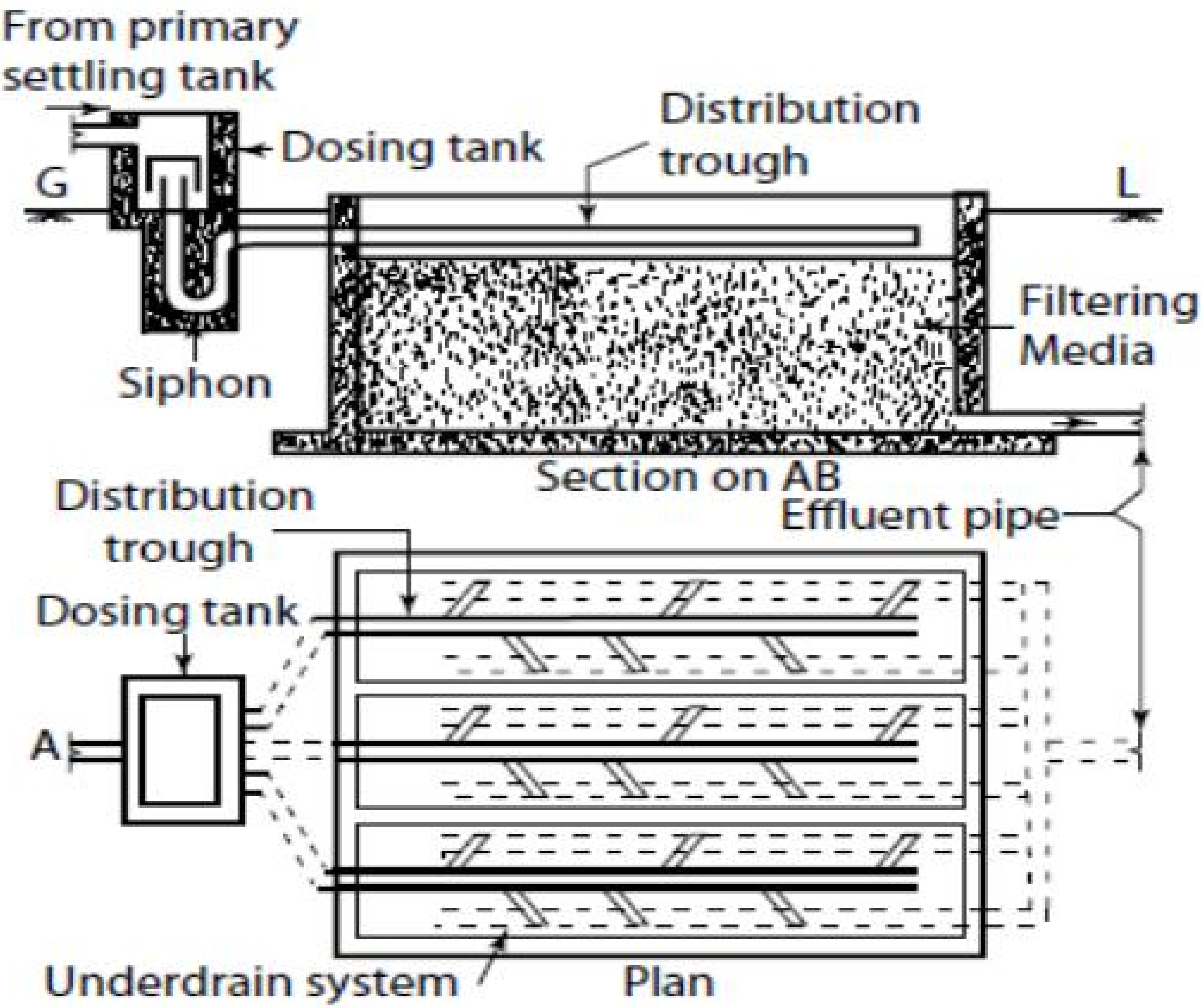


Figure 9.2 Contact beds

CONTACT BEDS

The effluent, after passing over the coarse filtering media is collected at the bottom and conveyed through the under drainage system to the effluent pipe which may be taken to secondary sedimentation tank for settling out the oxidised organic matter. The sewage is uniformly applied over the whole surface of a contact bed by means of distributing troughs having perforations/outlets at regular interval. The rate of loading is slow and may vary from 4000 to 6000 m^3/hectare/day. The area of one bed generally does not exceed 0.2 hectares. Figure 7.2 shows the details of contact beds.

Operation: The complete cycle of operation of a contact bed consists of the following four operations.

1. Filling: The outlet valve of the underdrain is closed and the tank is slowly filled with sewage effluent through the dosing tank. The depth of sewage effluent may be 5 to 10 cm over the top of the bed. This filling may take about 1 to 2 hours.

2. Contact: The dosing tank outlet is then closed, and the sewage admitted over the contact bed is allowed to stand for about 2 hours. During this period, the colloidal and dissolved matter gets transferred to the filter media, and comes in contact with the bacterial film covering the filter media.

3. Emptying: The outlet valve of the underdrain is then opened and the sewage present in the contact bed is withdrawn slowly, without disturbing the organic film. This operation may take about 1 to 2 hours.

4. Oxidation: The contact bed is then allowed to stand empty for about 4 to 6 hours. During this period of rest, atmospheric air enters into the void space of the contact media, thus supplying oxygen to the aerobic bacteria, resulting in the oxidation of the organic matter present in the film.

Advantages

Contact beds can work under small heads.

Contact beds can be operated without exposing the sewage effluent to view.

There is no nuisance of filter files.

The problem of odour is much less as compared to trickling filters.

Disadvantages

Rate of loading is much less in comparison to trickling filters.

Large area of land is required for their installation.

Intermittent operation requires continuous attendance.

The cost of contact beds much more as compared to trickling filters.

TRICKLING FILTERS

Trickling filters, also known as percolating filters or sprinkling filters are similar to contact beds in construction, but their operation is continuous and they allow constant aeration. In this system, sewage is allowed to sprinkle or trickle over a bed of course, rough, hard filter media, and it is then collected through the under drainage system. Spray nozzles or rotary distributors are used for this purpose. The biological purification is brought about mainly by aerobic bacteria which form a bacterial film, known as biofilm, around the particles of the filtering media. The colour of this film is blackish, greenish and yellowish and apart from bacteria, it may consist of fungi, algae, lichens, protozoa etc.

TRICKLING FILTERS

For the existence of this film, sufficient oxygen is supplied by providing suitable ventilation facilities in the body of the filter. The straining due to mechanical action of filter bed is much less. Organic removal occurs by biosorption from the rapidly moving part of the flow, and by progressive removal of soluble constituents from the more slowly moving portion.

The process of trickling filter.

A trickling filter holds a cylindrical tank and a high surface area material. The high specific surface offers a big region for creating a biofilm. Organisms growing in this slim biofilm over the media surface oxidize the organic content in the wastewater.

This chemical reaction emits carbon dioxide and water, and also forms new biomass at the same time. It occurs chiefly in the external region of the slime layer having a thickness of usually 0.1 to 0.2 mm.

The incoming pre-processed wastewater trickles via the bed or over the filter by using a rotating sprinkler. It is continuously distributed from above the media bed via a rotating sprinkler.

The process of trickling filter.

In the waste water, the microorganisms attach themselves to the filter media or bed, which is enclosed with bacteria. The bacteria oxidize the organic waste and emit pollutants out of it.

Proper aerobic conditions are sustained by splattering, diffusing, and either by natural air convection in case of porous filter medium or forced air via the bed. Diffusion of wastewater over the media supplies oxygen for oxidation of the organic compounds by the slime layer and emits carbon dioxide, water, and some more end products.

The process of trickling filter.

With the thickening of the slime layer, it becomes tougher for the air to enter into the layer. This results in an internal anaerobic layer. This slime layer keeps on developing until it sloughs off to form the processed effluent as a sludge requiring successive removal and disposal.

Classification of trickling filters

On the basis of hydraulic and organic loading rates, filters are usually divided into two classes:

1. Low rate or conventional Trickling filters and
2. High rate Trickling filters

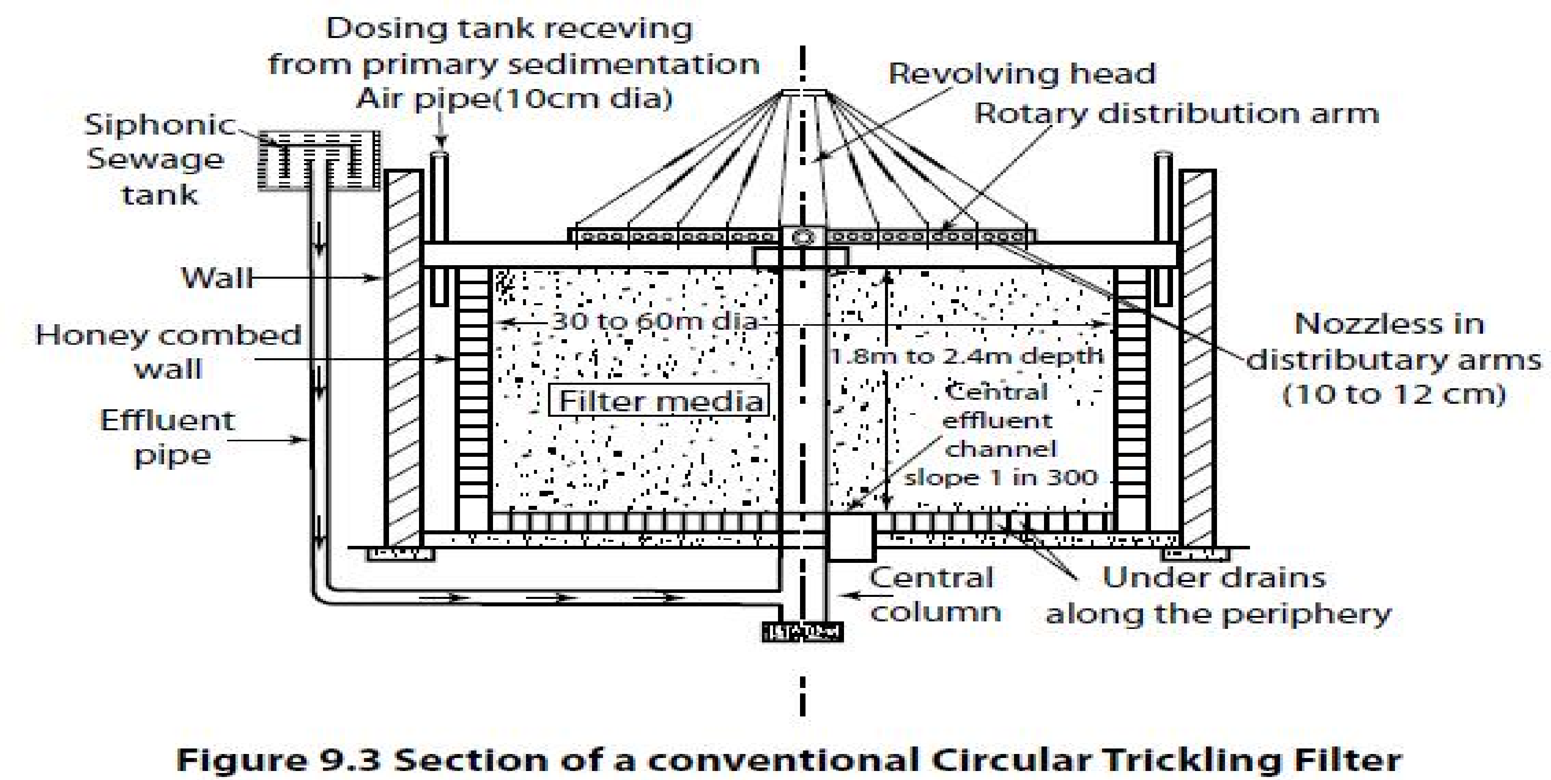


Figure 7.3 Section of a conventional Circular Trickling Filter

The Figure 7.3 shows the cross-section of a conventional circular trickling filter. It consists of following parts:

1. Water tight holding tank
2. Distribution system
3. Filter media
4. Under drainage system

1. Water tight holding tank

The trickling filters are generally constructed above the ground. They may either be rectangular or more generally circular. The tank walls of either masonry or concrete are made water tight. The walls of the tank are designed to withstand the pressure of sewage from inside. The under-drain system is supported by a floor which slopes to a collection channel.

2. Distribution system

Rectangular filters are provided with a network of pipes having fixed nozzles, which spray the incoming sewage into the air, which then falls over the bed of the filter under gravity. The circular filter tanks on the other hand, are provided with rotary distributors having a number of distributing arms. These distributors rotate around a central support either by an electric motor, or more generally by the force of reaction on the sprays. The rate of revolutions vary from 2 RPM for small distributors to less than 0.5 RPM for large distributors. The distributing arms should remain about 15 to 20 cm above the top surface of the filtering media in the tank.

3. Filter media

The filter media used for trickling filters should have high specific area; high percent void space, resistance to abrasion or disintegration during placement, insoluble in sewage or wastewater and resistance to spalling and flaking. Particles of filter media should be approximately round or cubical in shape and the filtering media should be free from flat or elongated pieces and should not contain dirt or any other undesirable materials. The most commonly used filter media is broken stone, slag or gravel of size 25 to 75 mm. For stone media, trap rock, granite or limestone may be used.

point or channel.

4. Under drainage system

The purpose of under drainage system is two fold: (i) to carry away the liquid effluent and sloughed biological solids, and (ii) to distribute air through the bed. The under-drains cover the entire floor of the filter to form a false bottom and consist of drains with semicircular or equivalent inverts. The slope of the under-drains should be the same as that of the floor, sloping towards a common collection point or channel.

Loading on trickling filters

The hydraulic loading rate is the total flow including recirculation applied on unit area of the filter in a day and it is 1 to 4 m^3/day/m^2. The organic loading rate is the 5-day 20^{o}C BOD, excluding the BOD of the recirculant, applied per unit volume in a day and it is 80 to 320 g/day/m^3.

Merits and demerits of conventional trickling filters

Advantages:

The various advantages of the trickling filters are:

1. The effluent obtained from trickling filters is highly nitrified and stabilised.
2. It has good dependability to produce good effluent under very widely varying weather and other conditions.
3. They can remove about 80% of suspended solids and about 75 to 80% of BOD.
4. The rate of filter loading is relatively higher, in comparison to contact beds or intermittent sand filters. Hence it requires lesser land space.

Merits and demerits of conventional trickling filters

Advantages:

5. The working of trickling filter is simple and cheap and does not require any skilled supervision.
6. As it contains less mechanical equipment, mechanical wear and tear is small.
7. Operation of trickling filters requires less electrical power to run the mechanical equipment.
8. The moisture content of sludge obtained from trickling filter system is as high as 90%.

Demerits:

1. The loss of head through the filter system is high, thus making the automatic dosing through siphonic dosing tanks necessary.
2. The cost of construction of the filter is high.
3. They require large area in comparison to other biological treatment processes.
4. They require preliminary treatment and therefore cannot treat raw sewage as such.
5. Final settlement in humus tank is necessary.
6. The process may develop odour and fly nuisance due to Psychoda which may be carried away into human habitation, proving serious nuisance and health problem.

Problem 7.1

Design a circular trickling filter unit for treating 4 million litres of sewage per day, having a 5 day BOD of 160 mg/L. Also design the underdrainage system as well as rotary system for the filter. Assume suitable design data where ever required.

Solution

(a) Design of Filter

Quantity sewage generated per day $= 4 \times 10^6$ MLD

$= 4 \times 10^3\ m^3/day$

Assume a hydraulic loading of $2\ m^3/d/m^2$

$$\therefore \text{Surface area required} = \frac{4 \times 10^3}{2} = 2000\ m^2$$

Assume an organic loading of $150\ g/d/m^3$

5 day BOD $= 160$ mg/L (given)

$$\text{Now total BOD present} = \frac{160}{1000} \times (4 \times 10^6) = 640000\ g/day$$

$$\therefore \text{Volume of filter media required} = \frac{640000}{150}$$

$$= 4266.7\ m^3$$

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

$$= \frac{4266.7}{2000} = 2.134 \text{ m}$$

$$\text{Also, dia. of filter} = \sqrt{\frac{2000 \ X\ 4}{\pi}} = 50.46 \text{ m}$$

Let us provide 50 m dia. filter unit.

$$\text{Actual surface area} = \frac{\pi}{4}(50)^2 = 1963.5 \text{ m}^2$$

$$\text{Required Depth} = \frac{4266.7}{1963.5} = 2.173 \text{ m}$$

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

$$= \frac{4266.7}{2000} = 2.134 \text{ m}$$

$$\text{Also, dia. of filter} \quad = \sqrt{\frac{2000\ X\ 4}{\pi}} = 50.46 \text{ m}$$

Let us provide 50 m dia. filter unit.

$$\text{Actual surface area} = \frac{\pi}{4}(50)^2 = 1963.5 \text{ m}^2$$

$$\text{Required Depth} = \frac{4266.7}{1963.5} = 2.173 \text{ m}$$

So provide filter depth of 2.2 m

$$\text{Hence, actual organic loading} = \frac{640000}{1963.5 \text{ X } 2.2}$$

$$= 148.16 \text{ g/d/m}^3 \text{ (Satisfactoty)}$$

$$\text{And actual hydraulic loading} = \frac{4 \text{ x } 10^6 \text{x} 10^{-3}}{1963.5} = 2.04 \text{ m}^3\text{/d/m}^2$$

(Satisfactoty)

(b)Design of rotary distributors

The pipe of rotary distributor is designed for peak velocity of not greater than 2.0 m/s and for average velocity of not greater than 2.0 m/s and for average velocity not less than 1 m/s. Let us take peak flow factor as 2.25.

$$\text{Peak flow} = \frac{2.25 \text{ X } 4\text{X } 10^6 \text{ X}10^{-3}}{24\text{X}60\text{X}60} = 0.1042 \text{ m}^3/\text{sec}$$

$$\therefore \text{Flow area of central column} = \frac{0.1042}{2} = 0.0521 \text{ m}^2$$

$$\therefore \text{Dia of central column} = \sqrt{\frac{0.0521\text{X } 4}{\pi}} = 0.2575 \text{ m}$$

Hence probide 25 cm dia. central column.

Design of arms

Let us provide 4 arms for the rotatary reaction spray type distributor.

$$\therefore \text{Peak discharge per arm} = \frac{0.1042}{4} = 0.02605 \text{ m}^3/\text{sec.}$$

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

Let us provide 24.875 m long arms with its size reducing from the centre to the end. For this purpose, let us provide 3 sections of arm, with first two sections of 8 m length and the third section of 8.875 m length. The flow in these sections of each arm has to be adjusted in proportion to the filter area covered by these lengths of arm. Let A_1, A_2 and A_3 be the circular filter areas covered by each length of the arm. Let us provide for 0.34 m dia. in the centre for the central column etc.

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

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

$$A_3 = \pi\,[(25)^2 - (16.17)^2] = \underline{1142.07\ m^2}$$

$$\text{Total} = 1963.41\ m^2$$

Hence proportionate areas served by each section of arm are:

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

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

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

$$\text{Total} = 100\%$$

Discharge through arm = $0.02605 m^3/sec$. The flow through velocity in the arm, at peak flow, should be less than 1.2 m/s

Design of first section of the arm

Discharge $= 0.02605\ m^3/sec.$

Design velocity $= 1.2\ m/s$

$\therefore$ Area required $= \frac{0.02605}{1.2} = 0.02171\ m^2$

$\therefore$ Dia required $= \sqrt{\frac{0.02171\ X\ 4}{\pi}} = 0.166\ m$

Hence provide 170 mm dia.

Design of second section of the arm

Discharge $= \frac{100-10.67}{100}$ x 0.02605

$= 0.02327\ m^3/sec.$

$\therefore$ Area required $= \frac{0.02327}{1.2} = 0.01939\ m^2$

$\therefore$ Dia required $= \sqrt{\frac{0.01939\ X\ 4}{\pi}} = 0.1571\ m$

Hence provide 160 mm dia.

Design of third section of the arm

$$\text{Discharge} = \frac{[100-(10.67+31.16)]}{100} \times 0.02605$$

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

$$\therefore \text{Area required} = \frac{0.01515}{1.2} = 0.012625\ m^2$$

$$\therefore \text{Dia required} = \sqrt{\frac{0.012625\ X\ 4}{\pi}} = 0.1268\ m$$

Hence provide 130 mm dia.

Design of orifices

Let us provide 12 mm dia. orifices with a coefficient of discharge C_d equal to 0.6 causing flow equal to 1.5 m.

$$\therefore \text{Discharge through each orifice} = C_d \,.\, a \,.\, \sqrt{2gh}$$

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

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

$$\therefore \text{No of orifices required in each arm} = \frac{0.02605}{3.6813 \times 10^{-4}} = 71 \text{ Nos.}$$

No of orifices (n) in each section of the arm will be as under:

$$\text{First sectioon, } n_1 = \frac{10.67}{100} \times 71 = 7.58 \quad \text{say } 8$$

$$\text{Second sectioon, } n_2 = \frac{31.16}{100} \times 71 = 22.12 \text{ say } 22$$

$$\text{Third sectioon, } n_3 = \frac{58.17}{100} \times 71 = 41.3 \quad \underline{\text{say } 41}$$

$$\text{Total} = 71$$

No of orifices (n) in each section of the arm will be as under:

$$\text{First sectioon, } n_1 = \frac{10.67}{100} \text{ x } 71 = 7.58 \quad \text{say } 8$$

$$\text{Second sectioon, } n_2 = \frac{31.16}{100} \text{ x } 71 = 22.12 \text{ say } 22$$

$$\text{Third sectioon, } n_3 = \frac{58.17}{100} \text{ x } 71 = 41.3 \quad \underline{\text{say } 41}$$

$$\text{Total} \qquad = 71$$

Spacing(s) of orifices in each section will be as under:

First sectioon, $S_1 = \frac{800}{8} = 100$ cm c/c

Second sectioon, $S_2 = \frac{800}{22} = 36.36$ cm c/c

Third sectioon, $S_3 = \frac{887.5}{41} = 21.65$ cm c/c *(c) Design of underdrainage system*

Peak flow = $0.1042\ m^3/sec$

Let us provide central channel of rectangular secttion, fed by radial laterals of semi-circular section discharging into the central channel. The radial laterals, laid at a slope of 1 in 40, will be in the form of under-drain block lengths containing semi-elliptical openings.

Design of rectangular effluent channel

The velocity of flow should not be less than 0.75 m/s at peak instantaneous hydraulic loading or not less than 0.6 m/s at average instantaneous hydraulic loading.

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

$$\text{Peak flow} = 0.1042\ m^3/sec$$

$$\therefore \text{Area of channel} = \frac{0.1042}{1} = 0.1042\ m^2$$

Let us provide a width of 0.25 m

$$\therefore \quad \text{Depth} = \frac{0.1042}{0.25} = 0.4168$$

Let us provide a width of 0.25 m and a depth of 0.40 m so that:

Area $A = 0.25 \times 0.4 = 0.10\ m^2$

$$\text{Actual velocity} = \frac{0.1042}{0.1} = 1.042$$

$$R = \frac{A}{P} = \frac{0.1042}{(0.25+2 \times 0.4)} = 0.0952$$

The bed slope of the channel is determined by Mannings formula:

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

Taking $N = 0.018$ (say), we have

$$0.1042 = \frac{1}{0.018} 0.10 (0.0952)^{2/3} S^{1/2}$$

From which $S = \frac{1}{123.6}$ Say 1 in 120

Hence, provide the central effluent channel of width 25 cm, and depth 40 cm below the level oflateral, and lay the channel at a slope of 1 in 120.

Design of radial laterals

Provide radial under-drains at a slope of 1 in 40. The laterals (semi circular) are designed to run approximately half full,

ie. $a/A = 0.25$ (semi circular)

Corresponding to this, we get, from the table of hydraulic elements of circular sewers running partially full (table 10.7) and by extrapolation

$d/D = 0.298$, $q/Q = 0.194$ and $r/R = 0.680$

Now permissible velocity at peak flow ≥ 0.75m/s

$$\therefore q/a = 0.75 \qquad \ldots.1$$

$$\text{But } Q = \frac{1}{N} A R^{2/3} S^{1/2} \text{ and } q = \frac{1}{N} a r^{2/3} S^{1/2}$$

$$\frac{q}{Q} = \frac{a}{A} \left(\frac{r}{R} \right)^{2/3}$$

$$\frac{q}{a} = \frac{Q}{A} \left(\frac{r}{R} \right)^{2/3} \qquad \ldots.2$$

Equating (1) and (2) and noting that $\frac{r}{R} = 0.68$, we get

$$0.75 = \frac{Q}{A}\left(\frac{r}{R}\right)^{2/3} = \frac{Q}{A}(0.68)^{2/3}$$

From which $\frac{Q}{A} = 0.9699$

or $\frac{Q}{A} = \frac{1}{N} A\, R^{2/3} S^{1/2} = 0.9699$

Taking N=0.015 and noting that R=D/4 and S = 1/40 we get

$$\frac{1}{0.015}\left(\frac{D}{4}\right)^{2/3}\frac{1}{\sqrt{40}} = 0.9699$$

From which D = 0.112 m

Hence provide 12 cm dia. semi-circular drain blocks.

Trickling filter troubles and remedies

Following are the common troubles occurring at the site and operation of trickling filters.

a) Fly nuisance

Slow rate trickling filters often becomes infested with small moth like, deceptively fragile flies called ‘Psychoda’. This flies donot bite, but may get into the eyes, nostrils and ears of men and animal.

Remedial measures

Flooding the filter is for about 24 hours at weekly intervals.

Jetting down the inside walls of the filter with a high pressure hose.

Chlorinating the filter influent

b) Odour nuisance

Odours from filter are due to undesirable growth, sludging and anaerobic decomposition. The presence of excessive odours is an indication that the filter is not operating efficiently.

Remedial measures

Maintaining a well ventilated filter, either by natural ventilation or by forced ventilation.

Recirculation of filter effluent or secondary clarifier effluent through the filter bed, dilute the influent sewage and add dissolved oxygen.

Aeration or chlorination of sewage before primary settling of sewage.

c) Ponding nuisance

This nuisance is caused when all the voids of the trickling filters are filled up due to chocking by heavy fungus or other suspended matters, due to which the sewage cannot pass through the filter and accumulation at the surface in the form of pond. Ponding decreases filter ventilation, reduce the effective volume of the filter and reduce the filter efficiency.

c) Ponding nuisance

Remedial measures

1. Open the clogged section by flushing with a fire hose and simultaneously loosening the aggregates by steel bar.
2. Reduce the strength of filter influent by recirculation.
3. Flooding the filter once in a day and allowing of it to stand for 24 hours.
4. Chlorinate the influent.
5. Stopping the distributor over ponded area.
6. Allowing the zoogleal mass to dry by keeping the filter out of operation for 12 to 48 hours.

HIGH RATE TRICKLING FILTERS

The basic difference between high rate trickling filter and conventional trickling filter is that the rate of filter loading of the former is several times more than that of the latter. The construction details and functioning of high rate trickling filters are same that of the conventional trickling filter, but with the difference that provision is made in them for recirculation of sewage through the filter, by pumping a part of the filter-effluent to the primary settling tank, and repassing through it and the filter. The high rate filters make it possible to pass sewage at greater loadings, thus requiring lesser space and lesser filter media. The main defect of the conventional trickling filter is that it has high initial cost, it requires large area of construction and it requires large quantity of filtering media.

Comparison between conventional and high rate trickling filters

The Comparison between conventional and high rate trickling filters is given in the table shown below.

Table 7.1 Comparison between conventional and high rate trickling filters.

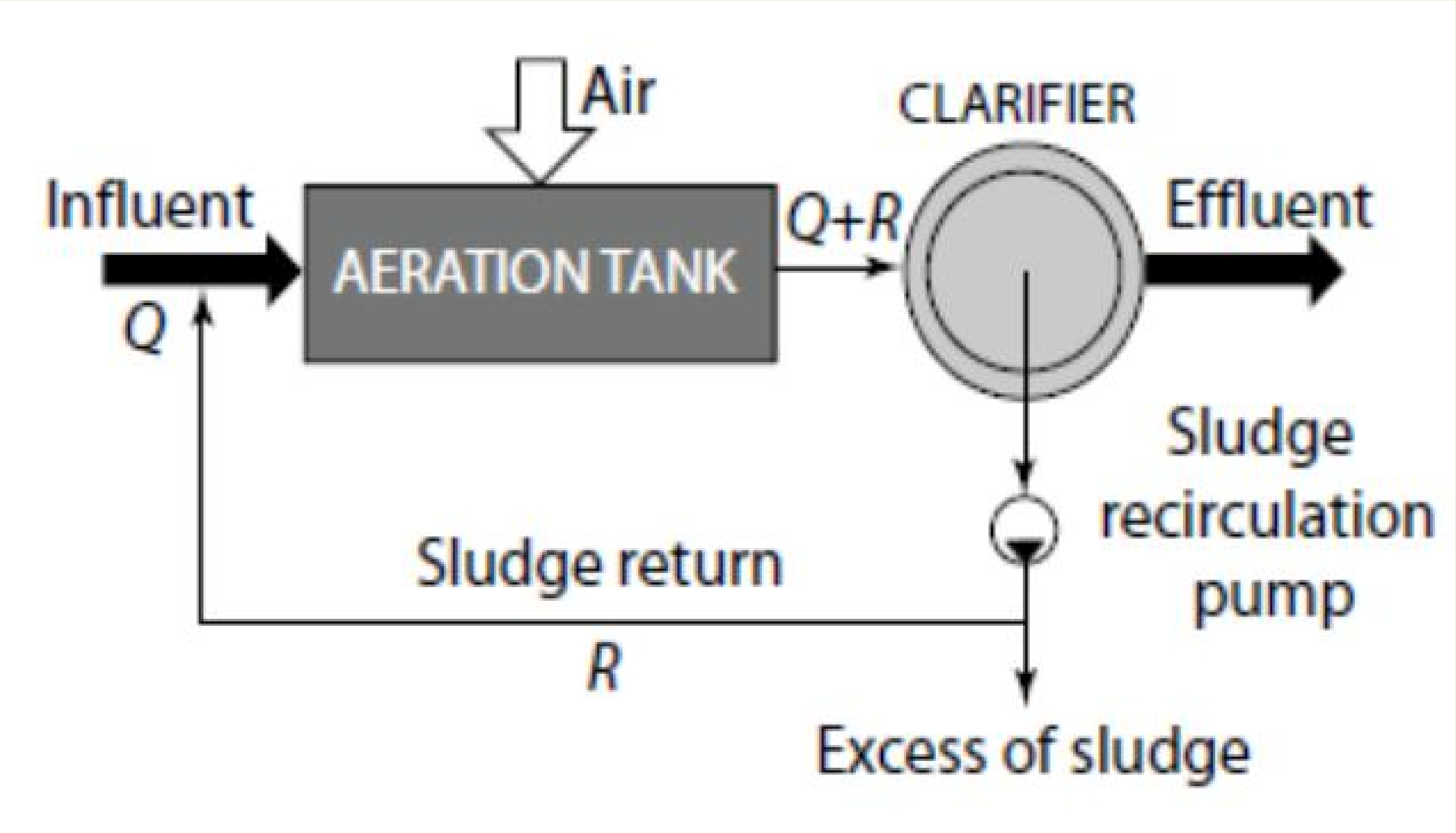
Sl No	Charecteristics	Conventional filter	High rate filter
1	Depth of filter media	1.8 to 2.4 m	1.2 to 1.8m
2	Size of filter media	25 to75 mm	25 to 60 mm
3	Land required	More	Less
4	Cost of operation	More	less
5	Hydraulic loading ($m^3/d/m^2$)	1 to 4	10 to 30 (Including recirculation)
6	Organic loading as 5-day BOD in $g/d/m^3$	80 to 320	500 to1000 (excluding recirculation)
7	Recirculation system	Usually not provided	Always provided
8	Sloughing	Intermittent	Continuous
9	Dosing interval	It generally varies between 3 to 10 minutes.	It is not more than 15 seconds
10	Characteristics of final effluent	Contains BOD $\leq$ 20 mg/L	Contains BOD $\geq$ 30 mg/L

ACTIVATED SLUDGE PROCESS

The activated sludge process provides an excellent method of treating either raw sewage or more generally the settled sewage. The sewage effluent from primary sedimentation tank, which is, thus normally utilised in this process, is mixed with 20 to 30 per cent of own volume of activated sludge, which contains a large concentration of highly active aerobic micro-organisms. The mixture enters an aeration tank, where the micro-organisms and the sewage, are intimately mixed together, with a large quantity of air for about 4 to 8 hours. Under these conditions the micro organisms will oxidise the organic matter and the suspended and colloidal matter tend to coagulate and form a precipitate, which settles down readily in the secondary settling tank.

ACTIVATED SLUDGE PROCESS

The settled sludge which contains large amount of active microorganisms is called activated sludge. This activated sludge is recycled to the head of the aeration tank, to be mixed again with the sewage being treated. New activated sludge is continuously being produced by this process, and a portion of it being utilised and sent back to the aeration tank, whereas the excess portion is disposed of properly along with the sludge collected during primary treatment after digestion.


Air
CLARIFIER
Influent
AERATION TANK
Q+R
Effluent
Q
Sludge
recirculation
pump
Sludge return
R
Excess of sludge

Flow diagram

The figure 7.4 shows the flow diagram of activated sludge process. Following are the three basic operations involved in the activated sludge process:

1. Mixing of activated sludge
2. Aeration of mixed liquor
3. Settling in secondary clarifier

a) Mixing of activated sludge

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

b) Aeration of mixed liquor

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

Methods of aeration

Following are the three methods which are employed for the purpose of aeration in activated sludge process.

1. *Diffused air aeration*
2. Mechanical aeration
3. Combination Diffused air aeration and mechanical aeration

i) Diffused air aeration

In this system compressed air is blown through sewage in aeration tank. Diffused air aeration involves the introduction of compressed air into the sewage through submerged diffusers or nozzles. The aerators may be of fine bubble or coarse bubble type.

ii) Mechanical aeration

Mechanical aerator generally consists of large diameter impeller plates revolving on vertical shaft at the surface of the liquid with or without draft tubes. A hydraulic jump is created by the impellers at the surface causing air entrainment in the sewage. The impellers also induce mixing.

iii) Combination of diffused air aeration and mechanical aeration

In this system, diffused air aeration and mechanical aeration are combined in a single unit, so as to achieve both the efficiency of diffused air system and economy of the mechanical aeration system simultaneously.

c) Settling in secondary clarifier

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

Extended aeration process

Primary sedimentation is avoided in this process, but grit chamber or communitor is often provided for screenings.

As its name suggests, the aeration period is quite large and extended to about 20 to 30 hours. The BOD removal efficiency is also quite high, to say about 90-98% as compared to 85- 95% of conventional plant.

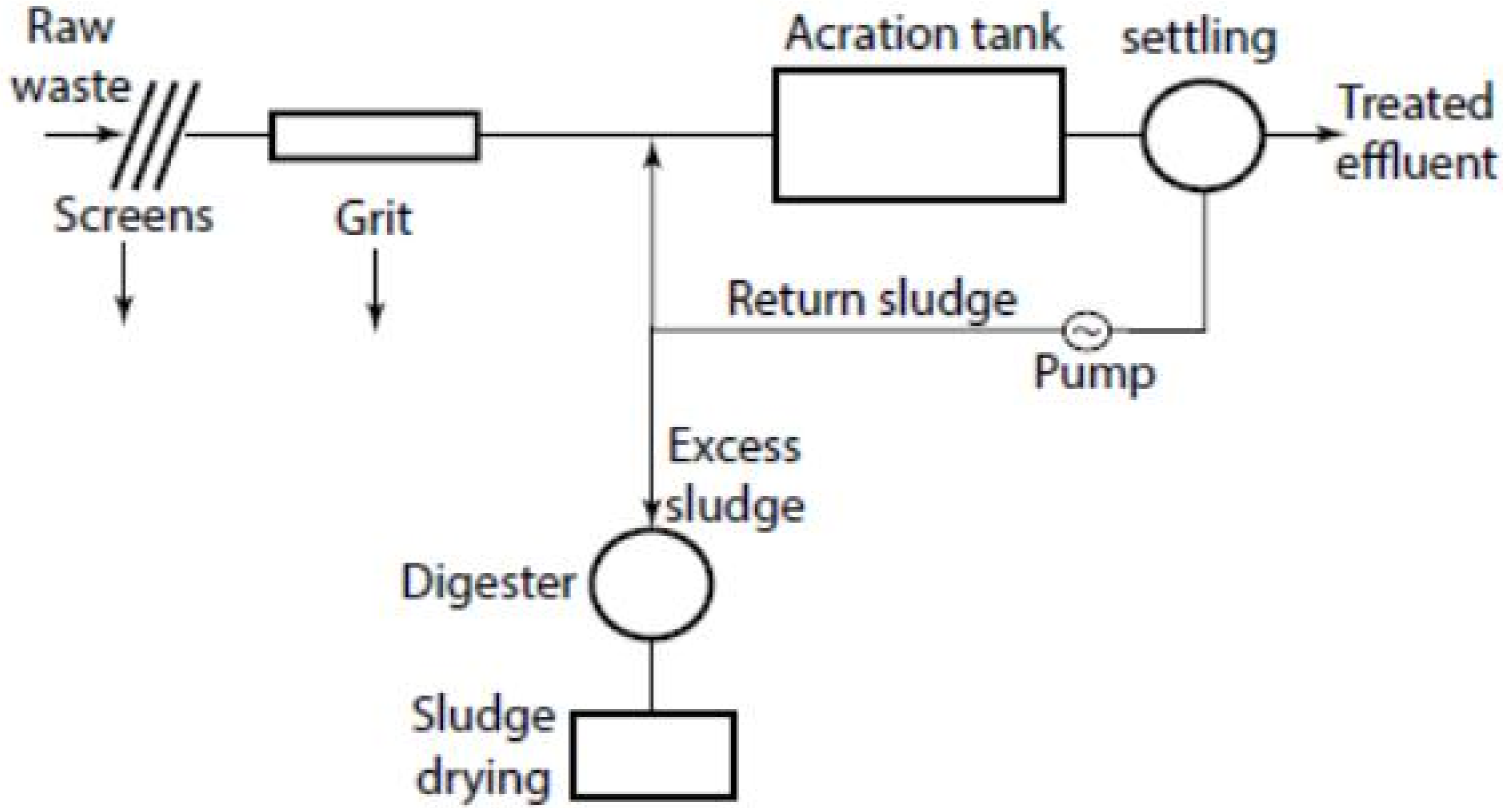


Figure 9.5 Flow diagram of extended aeration process

The air requirement is of course quite high, which increases the running cost of the plant considerably. No separate sludge digester is required here, because the solids undergo considerable decomposition and get well established over the long detention periods, adopted in the aeration tank. The sludge produced can be directly taken to sludge drying bed. Sludge production is also minimum in this method. Oxidation pitch is working on this principle.

Advantages of activated sludge plant

Following are the advantages of activated sludge process:

1. Lesser land area is required
2. No odours during the process as compared to other biological process.
3. The head loss in the plant is quite low
4. There is no fly or odour nuisance
5. Capital cost is less
6. Greater flexibility of operation
7. The excess sludge has higher fertlizing value

Disadvantages of activated sludge plant

Following are the advantages of activated sludge process:

1. High cost of operation with greater power consumption
2. A lot of machinery to be handled
3. The sudden change in the quantity and character of sewage may produce adverse effects on the working of the process
4. Bulking of sludge is a common trouble
5. The quantity of returned sludge has to be adjusted every time, as and when there is a change in the quantity of sewage flow, thus making the operation a little combersome.

Activated sludge process versus trickling filters

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

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

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

Rotating Biological Contactor (RBC)

A rotating biological contactor or RBC is a biological treatment process used in the treatment of wastewater following the primary treatment process.

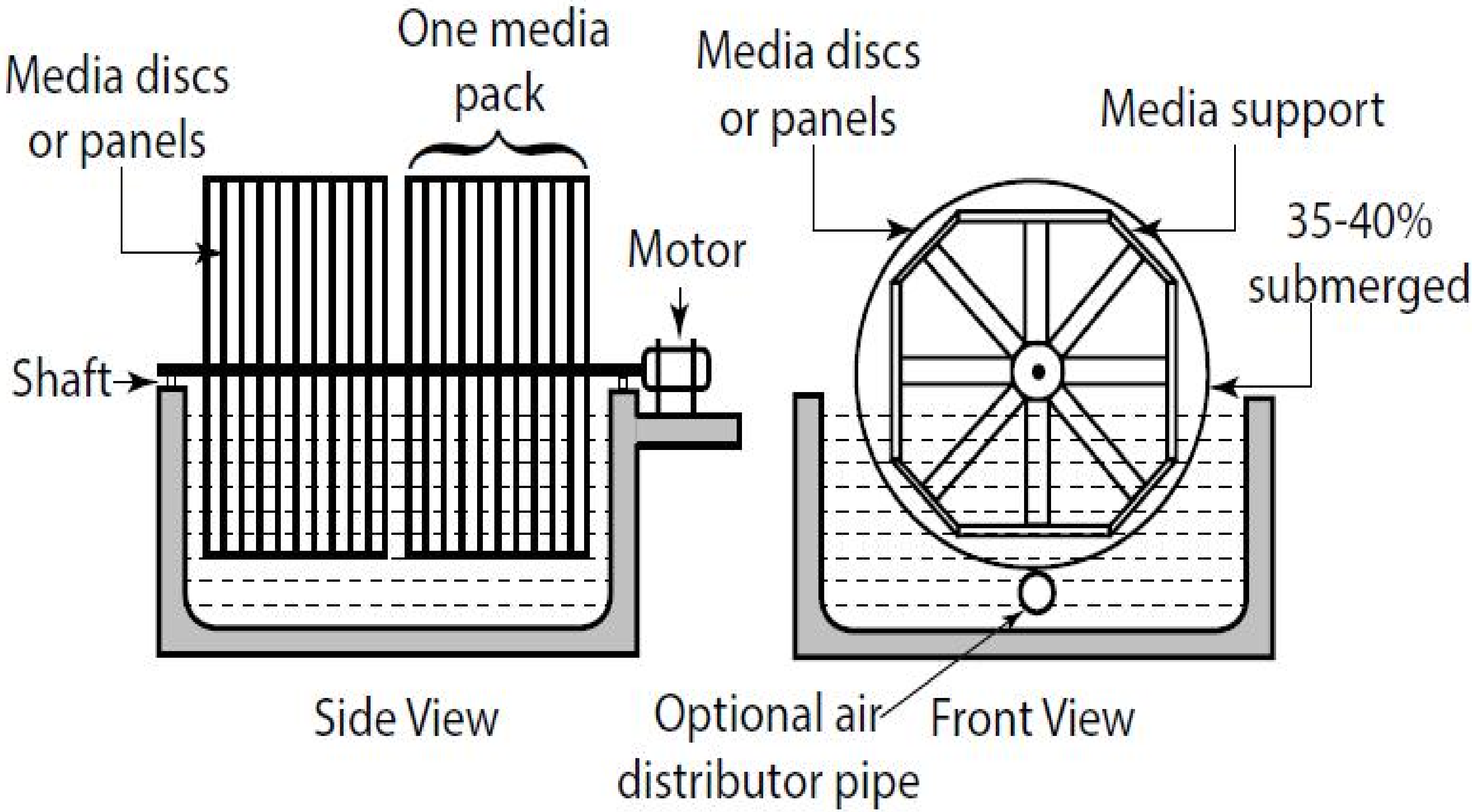


Fig 9.14 Rotating Biological Contactor

MODULE V

SEPTIC TANKS

A septic tank is a special form of primary sedimentation tank with a longer detention time, in which digestion of settled sludge also takes place. In other words, a septic tank is a combined sedimentation cum digestion tank so as to accommodate and hold the settled sludge for its subsequent digestion. The digestion of settled sludge is carried out by anaerobic decomposition process. In a septic tank raw sewage is admitted directly and sewage moves very slowly, allowing 60 to 70 percent of suspended matter to settle at the bottom of the tank.

SEPTIC TANKS

Some of the lighter solids including the grease and fat rise to the surface of the sewage as scum. The sludge and scum are retained in the tank for a period of several months during which they are decomposed by anaerobic bacteria to form gases like methane, carbon dioxide and hydrogen sulphide through a process called anaerobic digestion consequently, there is a reduction in the volume of the sludge to be disposed off.

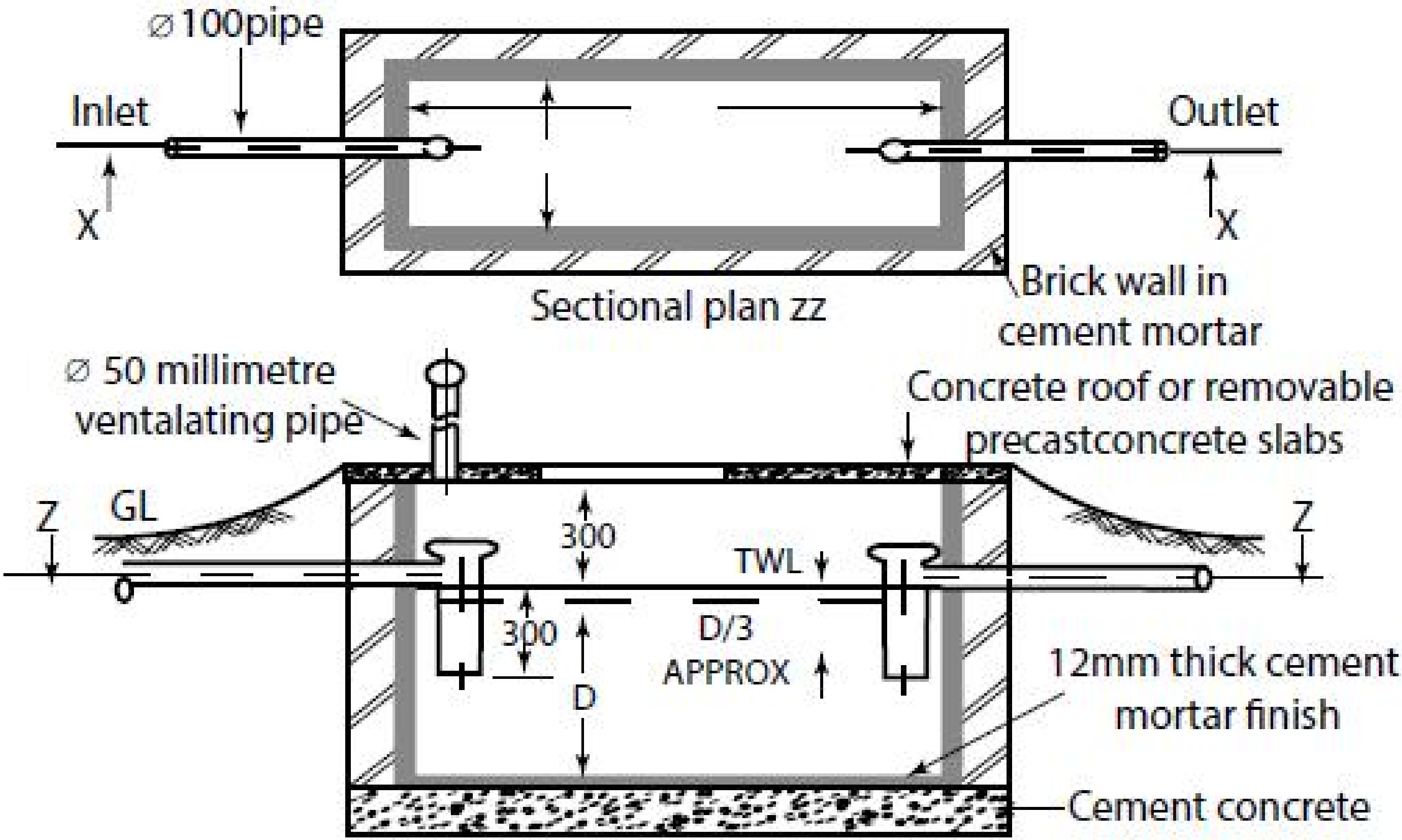


Figure 9.6 Plan and section of septic tank

Construction and operational features

Septic tanks are rectangular in plan with either flat or hopper bottom. The raw sewage enters at one end flows horizontally to the opposite end and discharges it through outlet pipe. The tank is designed to prevent direct currents between the tank inlet and outlet. This ensures effective sedimentation, and is achieved by using pipe tees with submerged ends as inlet and outlet as shown in figure. As a precaution against the disturbance of scum as well as sludge settling a vertical portion wall is built in the tank which is called a baffle wall. The baffle wall is usually constructed of RCC.

Construction and operational features

Septic tanks are usually constructed of brick wall, stone masonry, plain or RCC. The bottom of tank is made in rich cement concrete 1:2:4 with proper slope towards the sludge outlet. The tank is generally covered at top with a RCC slab. The cover prevents the escape of foul gases and odours from the tank to the ground. The gases are removed through a vent pipe projecting atleast 2 m above the top of the highest building within a radius of 20 m.

Effluent disposal from septic tank

The septic tank does not fulfill the object of complete treatment of sewage. The effluent coming out of a septic tank is highly offensive, odours and potentially dangerous. This effluent should, therefore, be disposed of carefully, so as to cause minimum nuisance or risk to the health. The effluent which is coming out from the septic tank is commonly disposed off on land by soak pit.

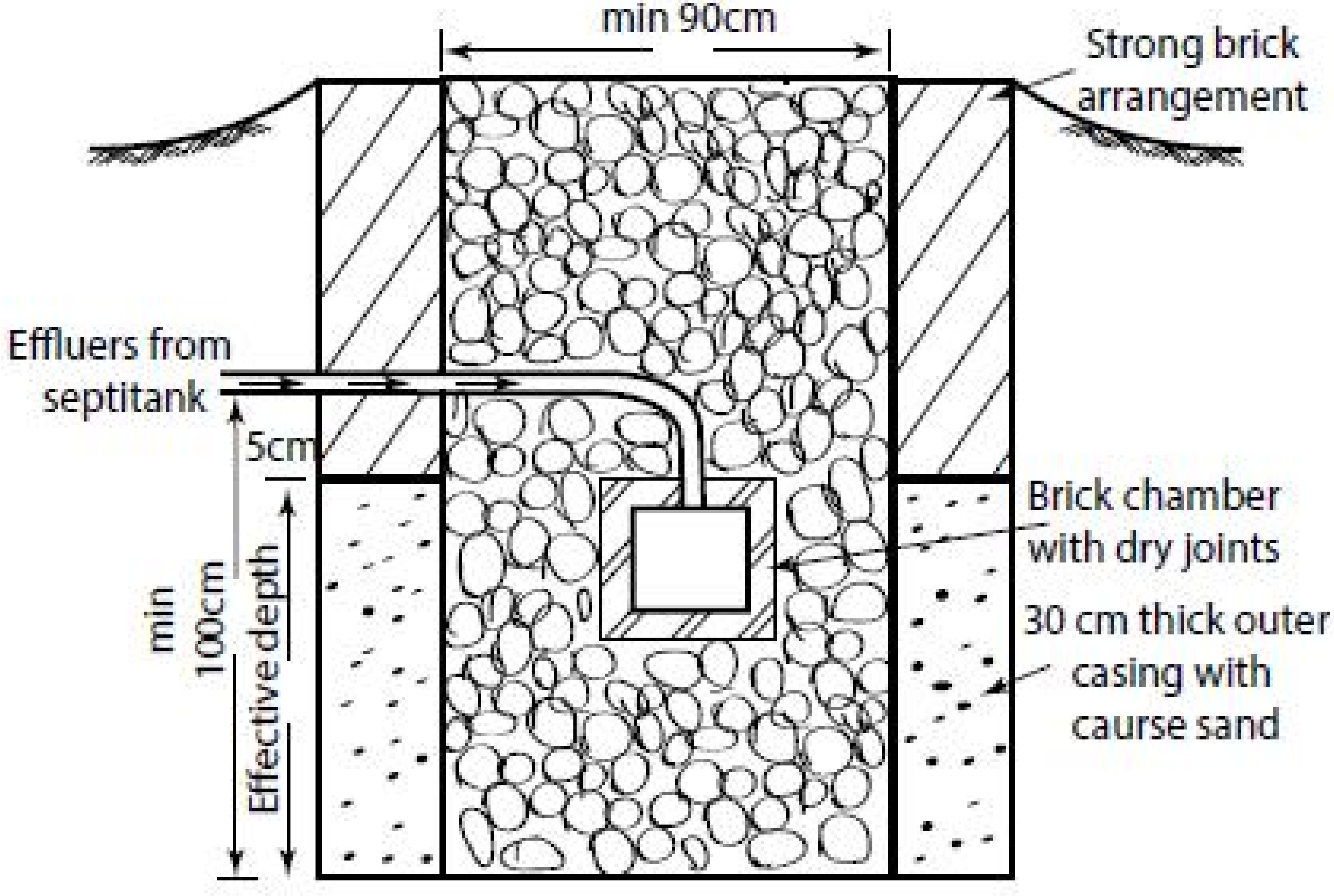


Figure 9.7 Soak Pit

A soak pit is a circular covered pit through which the effluent is allowed to be soaked or absorbed into the surrounding soil. The soak pit may either be filled with stone aggregates or may be kept empty.

When the soak pit is empty, the pit is lined with brick, stone or concrete block with dry open joints. When the soak pit is filled with stone or aggregate, no lining is required.

Problem 7.2

Design a septic tank for 200 users. Water allowance is 120 ltrs per head per day. Detension period may be 18 hours. Draw a neat dimensioned sketch of the septic tank you designed.

Solution

Flow of sewage per day = 200 x 120

= 24000 ltrs.

Detension Period = 18 hrs.

$\therefore$ Tank capacity = $\frac{24000}{24} \times 18 = 18000$ ltrs.

Assume that the tank is to be cleaned every year. Then sludge storage capacity @ 15 ltrs per person per year.

∴ Quantity of sludge generated per year = 200 x 15

= 3000 ltrs.

∴ Total tank capacity = 18000 + 3000

= 21000 ltrs.

Considering future expansion say 25000 ltrs.

= 25 m^3

Assume depth of liquid as 1.7 mtr.

$$\therefore \text{Plan areaof tank} = \frac{25}{1.7} = 14.71 \text{ say } 15 \text{ m}^2$$

Let the ratio of length to width of tank be 3

$$\therefore B \times 3B = 15 \text{ m}^2$$

$$B = \sqrt{\frac{15}{3}} = 2.23 \text{ mtr.}$$

Say 3 mtr.

$$\therefore \text{Length of tank} = 3 \times 3$$

$$= 9 \text{ mtr.}$$

Assume a free board of 30 cm. the total depth of tank would be

$$= 1.7 + 0.3 = 2 \text{ mtr.}$$

Hence provide a septic tank of size 9 x 3 x 2 mtr. size.

Design of Soak pit

Assume the percolating capacity of filter media is 1250 $ltrs/m^3/day$.

$$\therefore \text{Volume of soak pit} = \frac{25 \text{ X } 1000}{1250}$$

$$= 20 \text{ m}^3$$

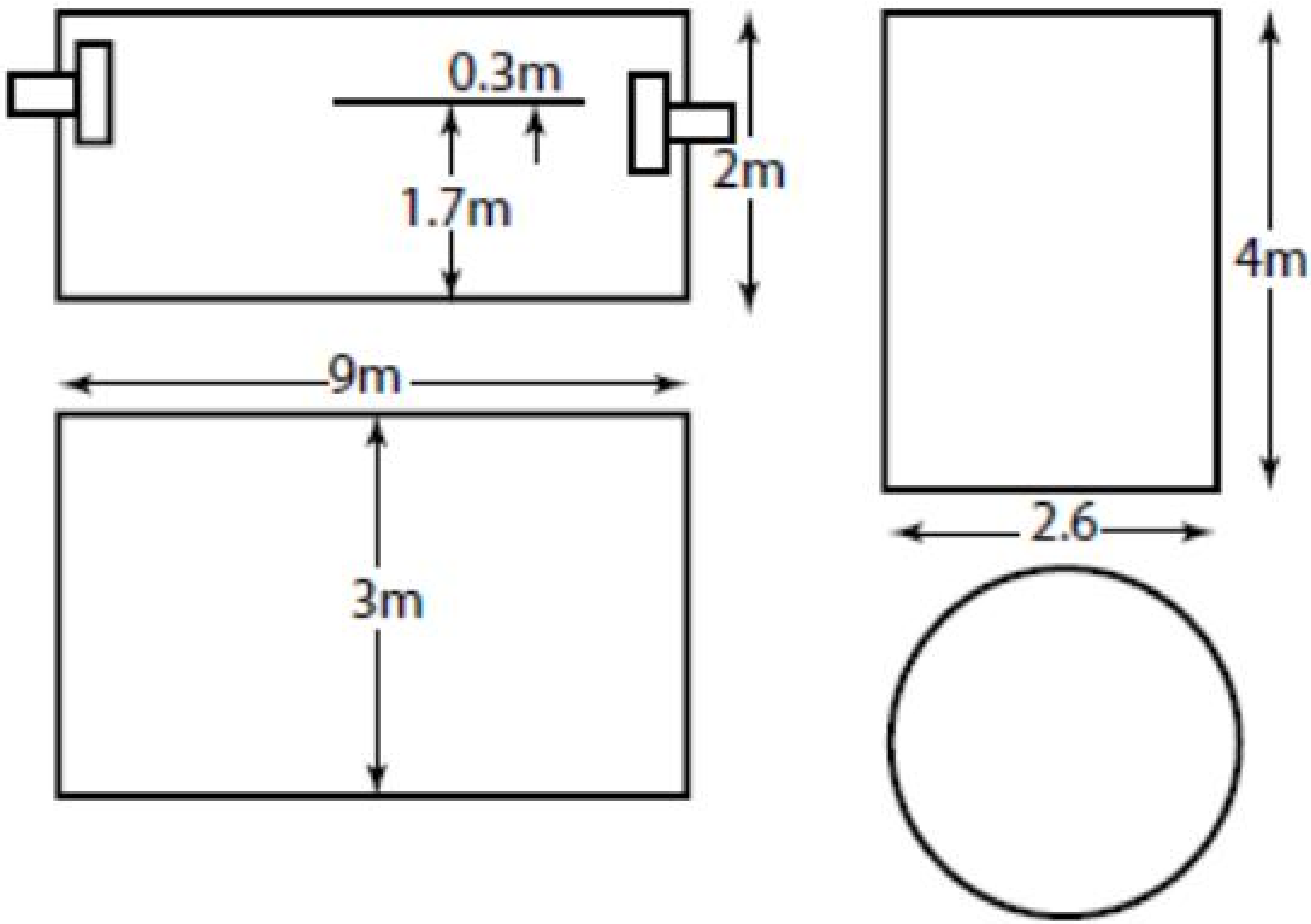
Assume depth of soak pit = 4 mtr.

$$\therefore \text{Area of soak pit} = \frac{20}{4} = 5 \text{ m}^2$$

$$\frac{\pi d^2}{4} = 5$$

$$\therefore d = \sqrt{\frac{5 \text{ X } 4}{\pi}} = 2.52 \text{ mtr.}$$

Say d = 2.6 mtr.


0.3m
1.7m
2m
9m
3m
4m
2.6

IMHOFF TANK

An imhoff tank is an improvement over septic tank, in which the incoming sewage is not allowed to mixed up with the sludge produced, and the outgoing effluent is not allowed to carry with it large amount of organic matter, as in the case of a septic tank. An Imhoff tank is a two storied tank, and was designed by a German scientist Mr. Karl Imhoff. They are sometimes also known as two-story digestion tank.

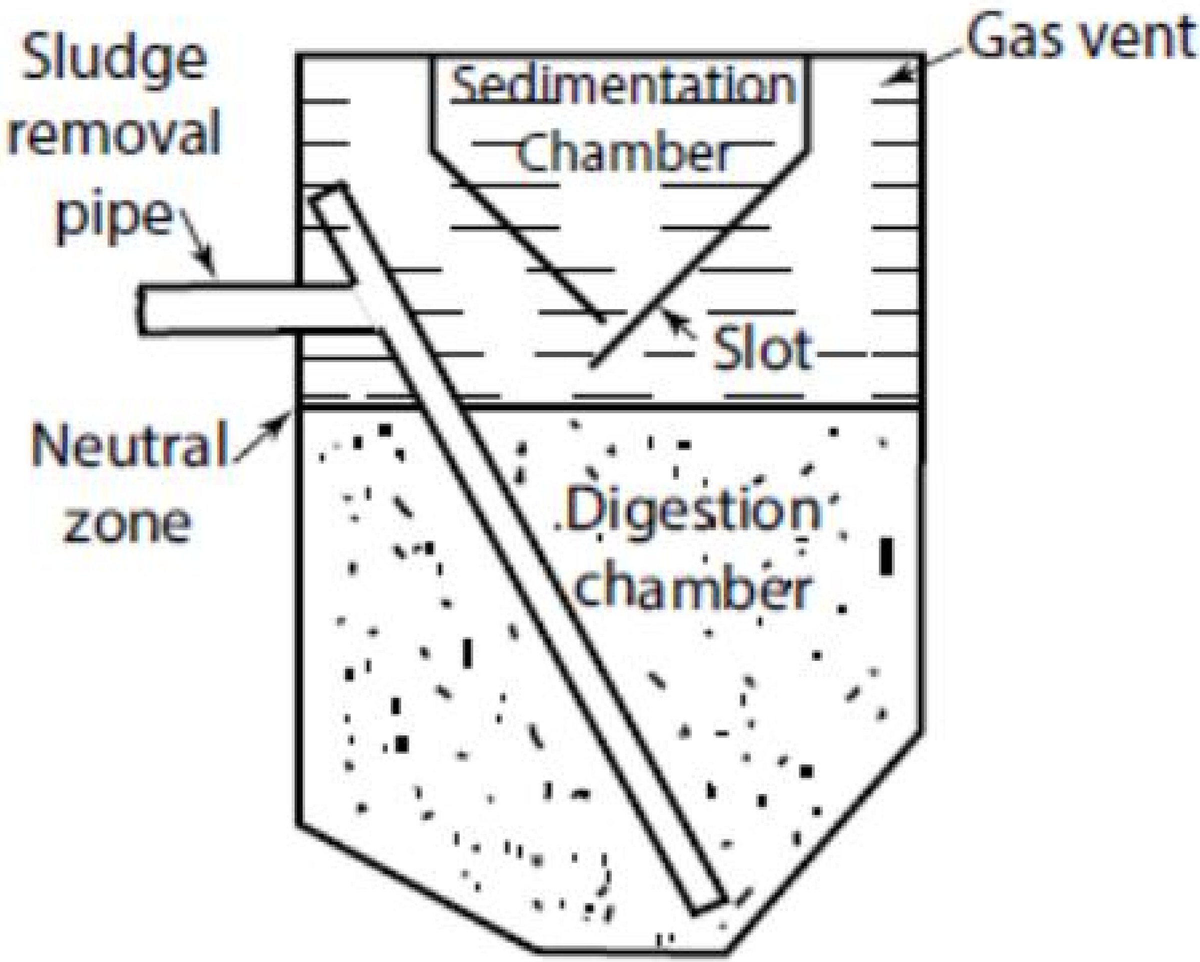


Figure 9.8 Imhoff tank

It is a double chamber rectangular tank. The upper chamber is called the sedimentation chamber or flowing through chamber through which the sewage flows at a very low velocity; and the lower chamber is the digestion chamber in which the sludge gets digested due to anaerobic decomposition.

The solids of the slowly moving sewage, settling down to the bottom of the sedimentation chamber, through the sloping bottom sides of the sedimentation chamber will slide down into the digestion chamber through an entrance slot at the lowest point of a sedimentation chamber. The slot is tapered in which a way that the gases generated in the digestion chamber cannot enter the sedimentation chamber, and thus avoiding direct contact of sewage with the foul gases, and its consequent pollution.

A gas vent is also provided above the digestion chamber to take care of the gases escaping to the surface.

7.11 UPFLOW ANAEROBIC SLUDGE BLANKET (UASB) REACTOR

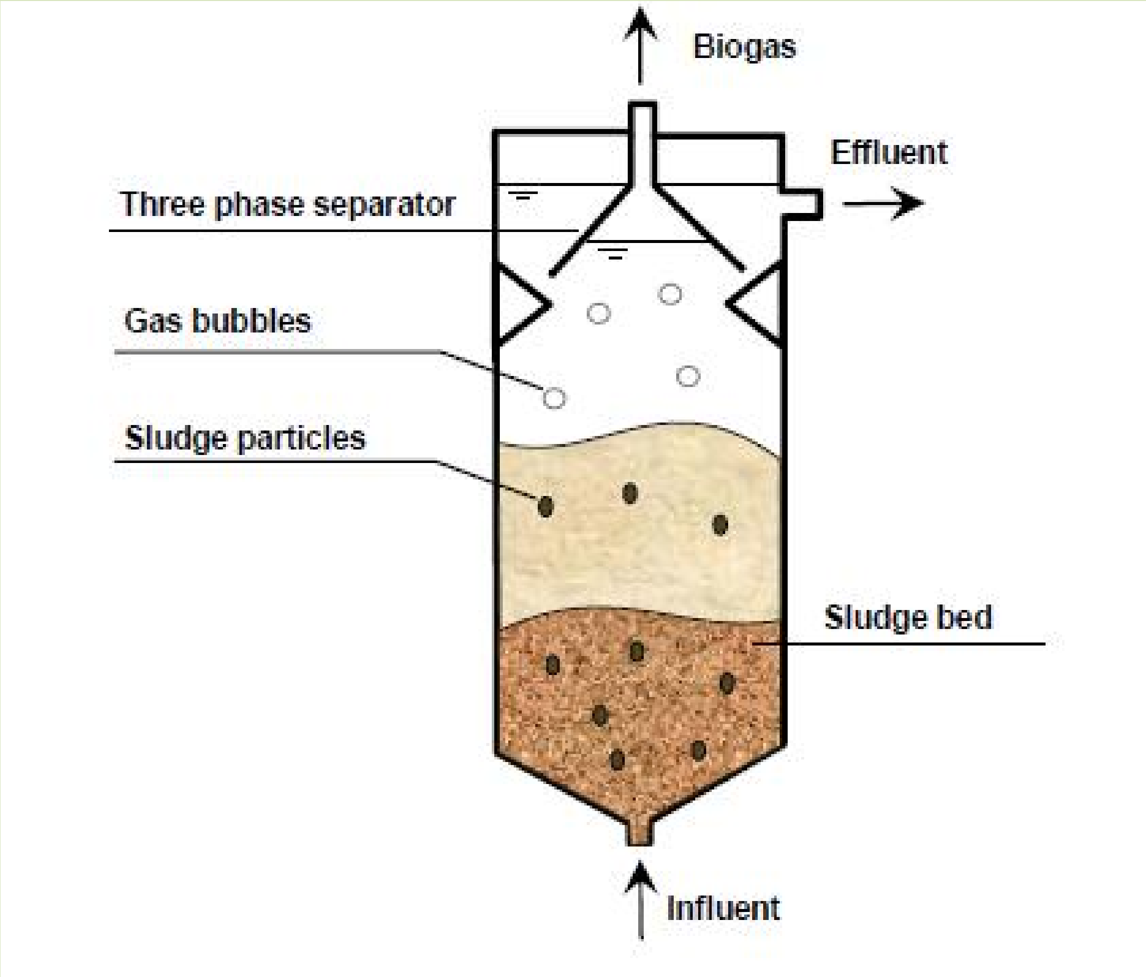


Figure 7.9 upflow anaerobic sludge blanket (UASB) reactor

It is somewhat modified version of the contact process, based on an upward movement of the liquid waste through a dense blanket of anaerobic sludge. No inert medium is provided in these systems. The biomass growth takes place on the fine sludge particles, which then develop as sludge granules of high specific gravity. The reactor can be divided in three parts (Figure 7.9), sludge bed, sludge blanket and three phase separator (gas-liquid-solid, GLS separator) provided at the top of the reactor.

Treatment occurs as the wastewater comes in contact with the granules and/or thick flocculent sludge. The gases produced causes internal mixing in the reactor. Some of the gas produced within the sludge bed gets attached to the biological granules. The free gas and the particles with the attached gas rise to the top of the reactor. On the top of sludge bed and below GLS separator, thin concentration of sludge is maintained, which is called as sludge blanket. This zone occupies 15 to 25% of reactor volume. Maintaining sludge blanket zone is important to dilute and further treat the wastewater stream that has bypassed the sludge bed portion following the rising biogas.

The GLS separator occupies about 20 to 30% of the reactor volume. The particles that raise to the liquid surface strike the bottom of the degassing baffles, which causes the attached gas bubbles to be released. The degassed granules typically drop back to the surface of the sludge bed. The free gas and gas released from the granules is captured in the gas collection domes located at the top of the reactor. Liquid containing some residual solids and biological granules passes into a settling chamber, where the residual solids are separated from the liquid. The separated solids fall back through the baffle system to the top of the sludge blanket.

OXIDATION DITCH

The oxidation ditch is essentially an extended-aeration activated sludge process. An oxidation ditch consists of an endless ditch for the aeration tank and a rotor for aeration mechanism. The ditch consists of a long continuous channel, usually oval in plan. The channel may be earthen with lined sloping sides and lined floor or it may be built in concrete or brick with vertical walls. There is normally no primary tank used in the oxidation ditch process. Raw sewage passes directly through a bar screen to the ditch.

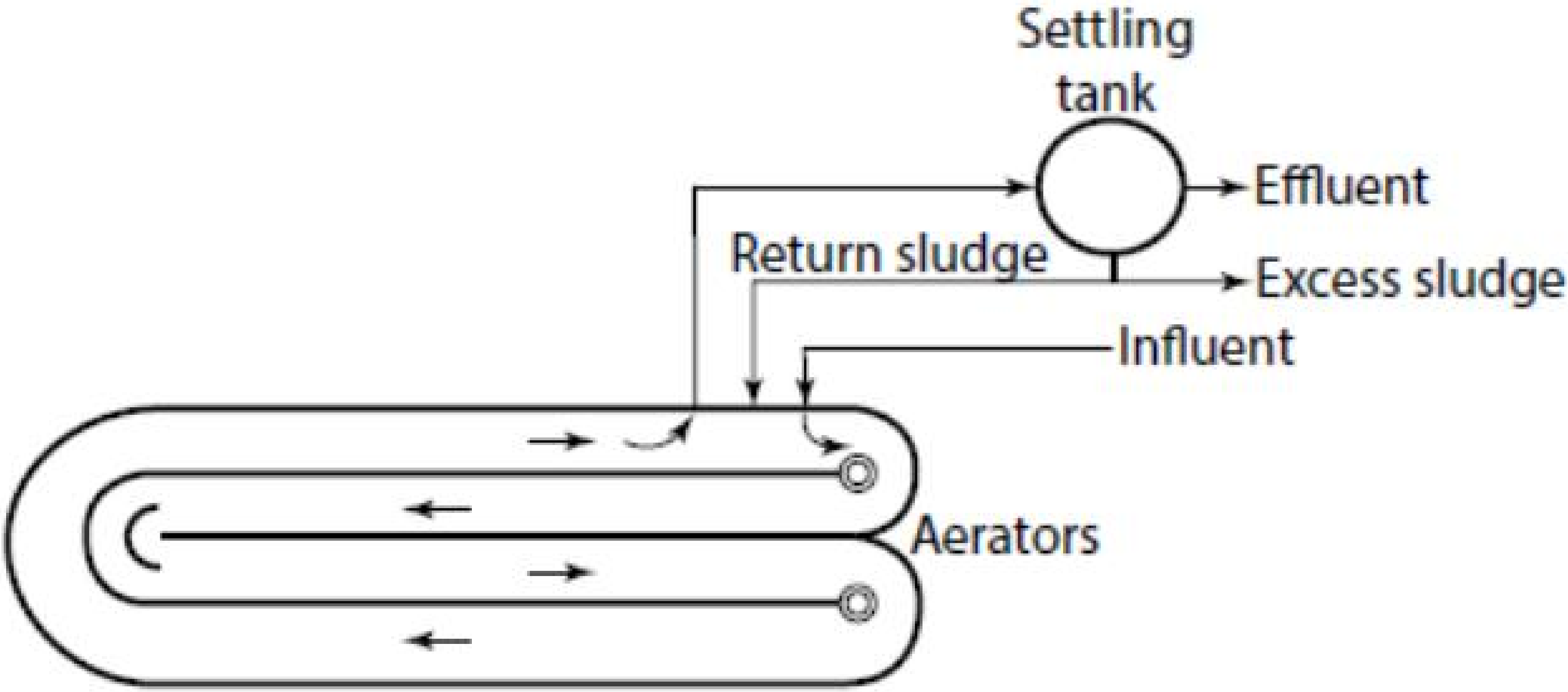


Figure 9.9 Oxidation Ditch

The sewage is aerated by surface rotar placed across the channel, the rotar entrains the necessary oxygen into the liquid and the contents of the ditch mixed and moving. They are designed to impart a velocity of 0.3 to 0.4 m/s to the mixed liquor, preventing the biological sludge from setting out. The width of the ditch divided by the rotor length should give a ratio between 1.5 and 2.8. The longer ratios are normally used for short length of 0.9 to 1.2 m. Oxidation ditches are constructed in two types.

1. Continuous flow type
2. Intermittent flow type

(i) Continuous flow type

In the continuous flow type oxidation ditch the operation is kept continuous by allowing the mixed liquor to settle in a separate settling tank. Quiescent conditions in the clarified liquid passes over the effluent weir for final disposal. The settled sludge is removed from the bottom of the clarifier by an air lift or pump and returned to the ditch. The oxidation ditch is operated as a closed system, and the net growth of the volatile suspended solids will require periodic removal of some sludge from the process

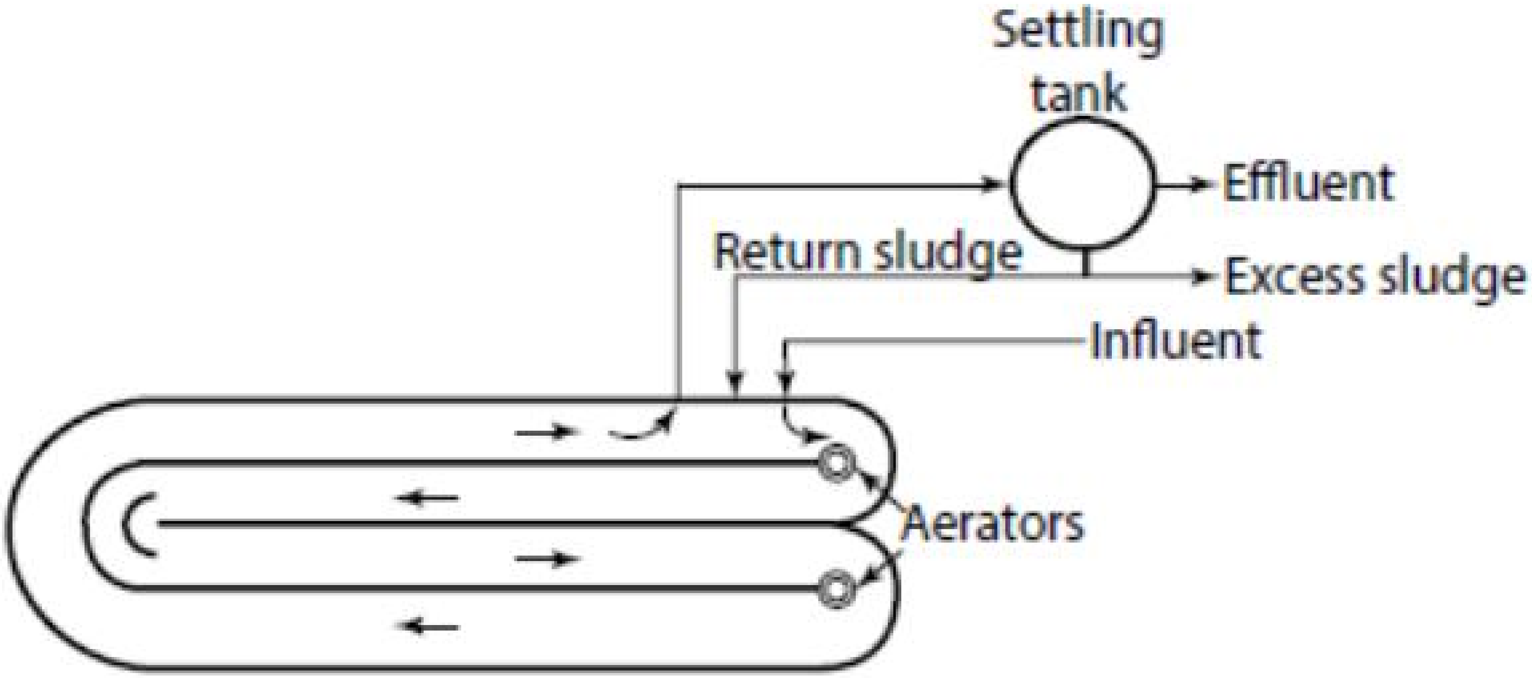


Figure 9.10 Oxidation Ditch continuous flow type

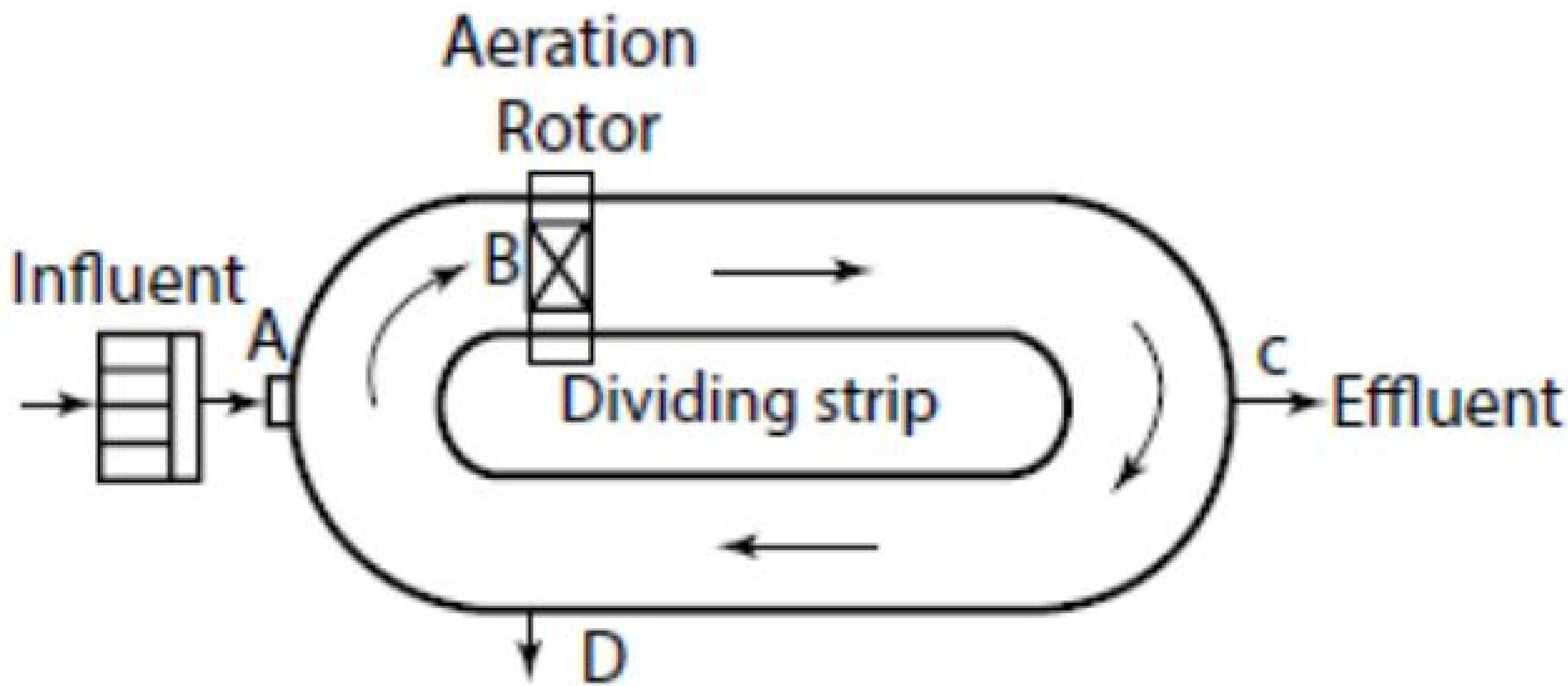


Figure 9.11 Oxidation Ditch intermittent flow type

In the intermittent type oxidation ditch, the numbers of separate settling tanks are used. The flow in the ditch remains suspended during a predetermined period, by stopping the rotar and the ditch itself is used for settling. The supernatant is withdrawn through the outlet. The surplus sludge, settled in the ditch is removed with the aid of a sludge trap. For intermittent operation, the cycle consists of:

i) Closing the inlet valve (A) and aerating the waste water.

ii) Stopping the rotor and lefting the contents settlle.

Opening both inlet, and outlet valves, thereby allowing the incoming wastewater to displace on equal volume of clarified effluent.

OXIDATION PONDS (STABILIZATION PONDS)

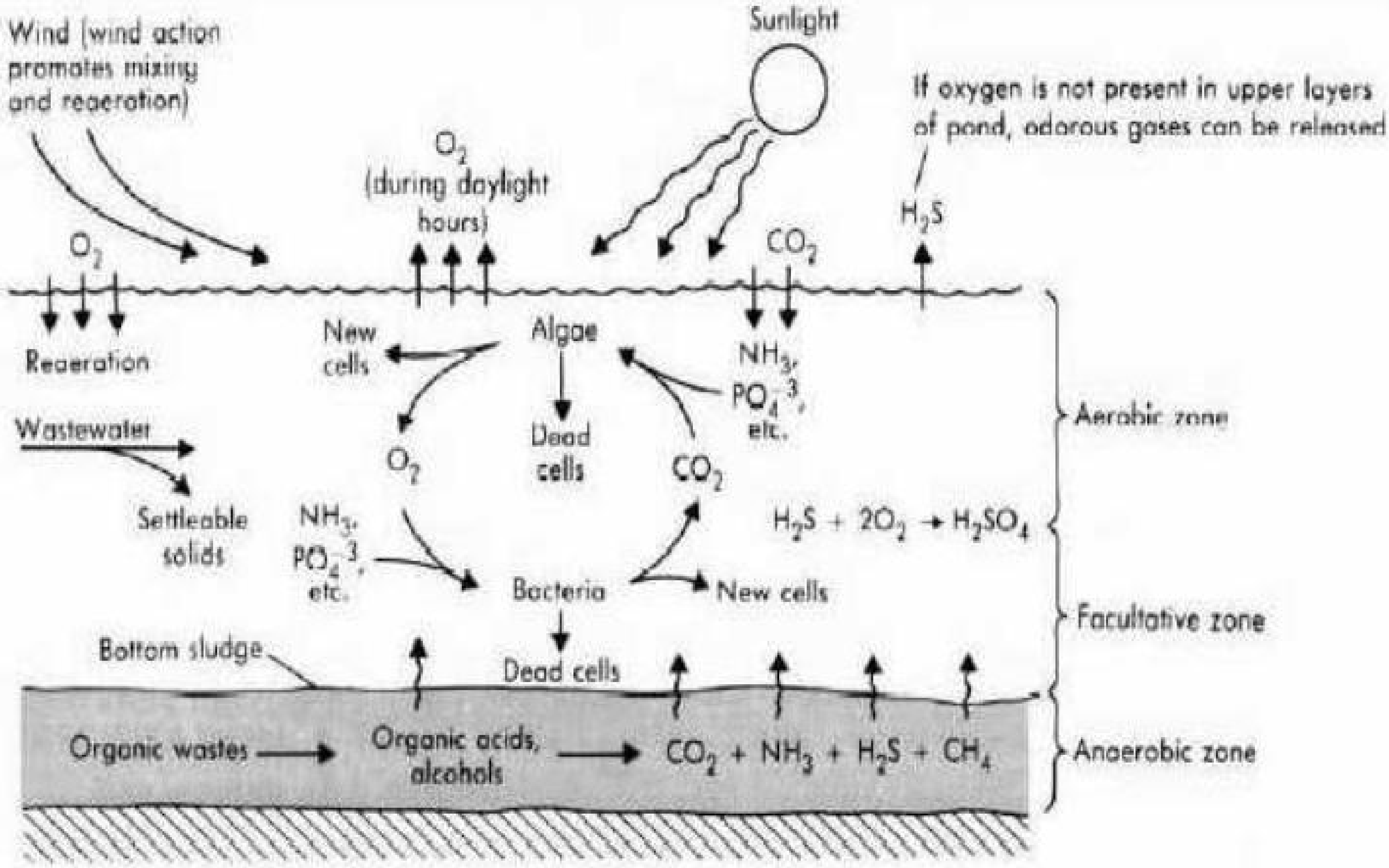
It is a shallow body of water contained in an earthen basin, open to sun and air. Longer time of retention from few days to weeks is provided in the pond. The purification of wastewater occurs due to symbiotic relationship of bacteria and algae. The ponds are classified according to the nature of the biological activity which takes place within the pond as aerobic, facultative and anaerobic. These are cheaper to construct and operate in warm climate as compared to conventional treatment system and hence they are considered as low cost wastewater treatment systems. However, they require higher land area as compared to conventional treatment system.

Aerobic pond

In aerobic pond, the stabilisation of wastes is brought about by aerobic bacteria, which flourish in the presence of oxygen. The oxygen demand of such bacteria in such a pond is met by the combined action of algae and other microorganisms, called algal photosynthesis, or algal-symbiosis. In this symbiosis, the algae while growing in the presence of sun light produce oxygen by the action of photosynthesis; and this oxygen is utilised by the bacteria for oxidising the waste organic matter. The end products of the process are carbon dioxide, ammonia and phosphates, which are required by the algae to grow and continue to produce oxygen.

Anaerobic pond

In anaerobic pond, the entire depth is under anaerobic condition except an extremely shallow top layer. Normally these ponds are used in series followed by facultative or aerobic pond for complete treatment. The depth of these ponds is in the range of 2.5 to 6 m. They are generally used for the treatment of high strength industrial wastewaters and sometimes for municipal wastewater and sludges. Depending upon the strength of the wastewater, longer retention time up to 50 days is maintained in the anaerobic ponds. Anaerobic lagoons are covered these days by polyethylene sheet for biogas recovery and eliminating smell problem and green house gas emission in atmosphere.



Wind (wind action promotes mixing and reaeration)
Sunlight
If oxygen is not present in upper layers of pond, odorous gases can be released
O_2 (during daylight hours)
H_2S
CO_2
O_2
Reaeration
New cells
Algae
NH_3, PO_4^{-3}, etc.
Aerobic zone
Wastewater
Dead cells
O_2
CO_2
Settleable solids
NH_3, PO_4^{-3}, etc.
$H_2S + 2O_2 \rightarrow H_2SO_4$
Bacteria
New cells
Facultative zone
Bottom sludge
Dead cells
Organic wastes
Organic acids, alcohols
$CO_2 + NH_3 + H_2S + CH_4$
Anaerobic zone

Most of the ponds exist in facultative nature. Three zones exist in this type of ponds (Figure 7.12). The top zone is an aerobic zone in which the algal photosynthesis and aerobic biodegradation takes place. In the bottom zone, the organic matter present in wastewater and cells generated in aerobic zone settle down and undergo anaerobic decomposition. The intermediate zone is partly aerobic and partly anaerobic. The decomposition of organic waste in this zone is carried out by facultative bacteria. The nuisance associated with the anaerobic reaction is eliminated due to the presence of top aerobic zone. Maintenance of an aerobic condition at top layer is important for proper functioning of facultative stabilization pond, and it depends on solar radiation, wastewater characteristics, BOD loading and temperature. Performance of these ponds is comparable with conventional wastewater treatment.

Constructional details

A typical plan of an oxidation pond is shown in figure 7.13. It is an earthen pond, dug into the ground, with shallow depth. Oxidation ponds are rectangular in shape (L/B = 2-3/1) having side slopes (1:1.5) and are constructed by building embankments of earth. They are of shallow depth usually 0.9-1.5m and as such effective in permitting penetration of sunlight to all parts of the waste water encouraging algal growth. Influents is applied in the middle of pond and allowed to be spread by the action of wind currents which prevents any odour nuisance due to concentration.

Constructional details

pond depth should not exceed 1.8 m or so, as otherwise the pond may turn into a deeper anaerobic pond rather than remaining faculative in character without giving foul odours. The detention time in the pond is usually 2 to 6 weeks, depending upon sun light and temperature. In cold countries, higher figure is to be adopted.

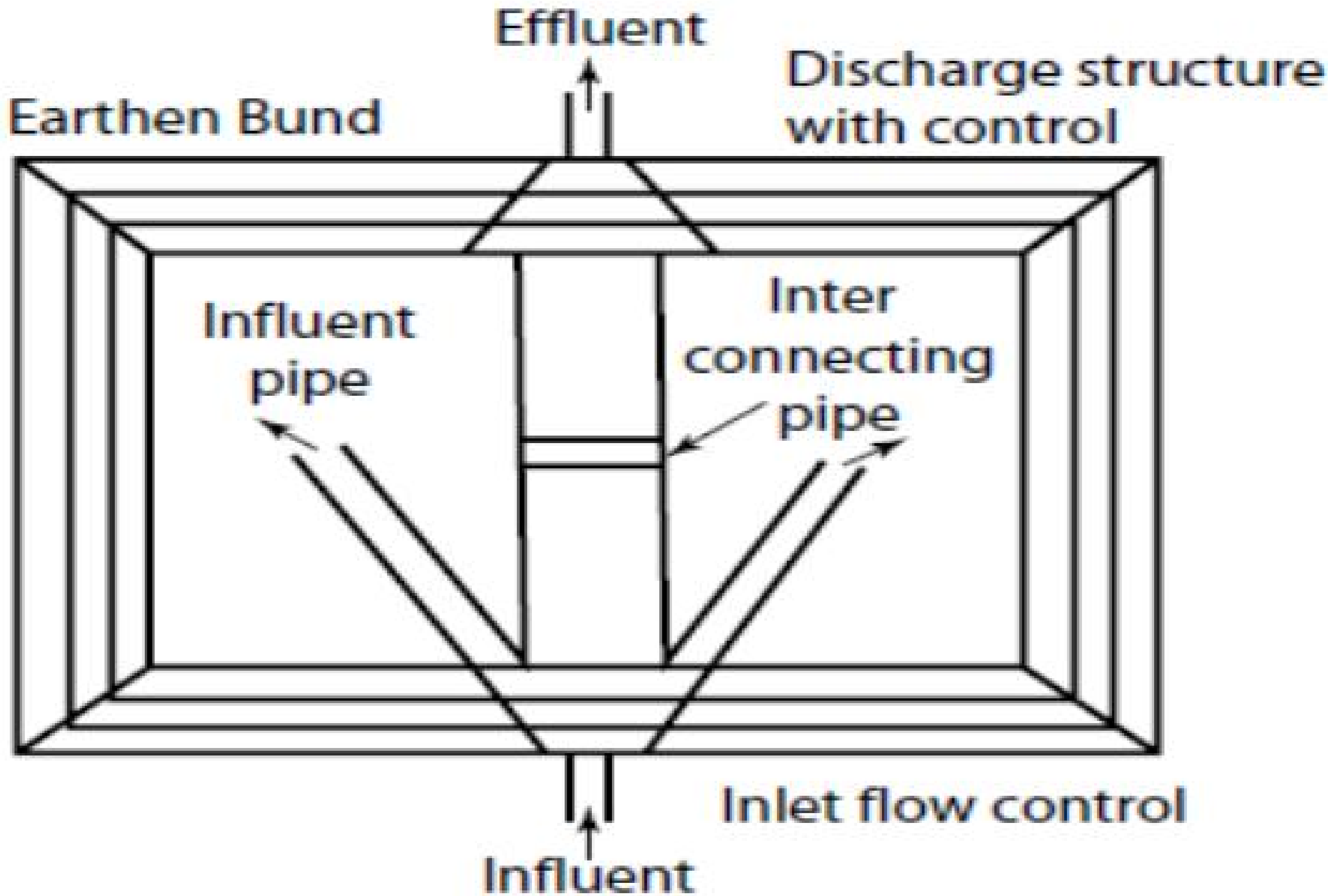


Figure 9.13 Plan of oxidation pond

Advantages of oxidation ponds

1. Low cost
2. Quickness of construction
3. Easy maintenance
4. High efficiency of BOD removal

Disadvantages of oxidation ponds

1. Nuisance due to mosquito breeding and odours.

Problem 7.3

Design an oxidation pond for treating domestic sewage of 15000 persons supplied with 200 liters per capita water per day. The BOD and suspended solids are each of 400 mg/L. Permissible organic loading for the pond is not less than 600 kg/ha/day. The detention period is not exceed 6 days. Assume any other suitable data.

Solution:

The quantity of sewage to be treated per day

$$= 15000 \text{ X } 200$$

$$= 3000000 = 3 \text{ X } 10^6 \text{ ltrs.}$$

$$= 3 \text{ MLD}$$

$$= 3000 \text{ m}^3$$

The BOD content per day $= \dfrac{3 \text{X} 10^6 \text{X } 400}{1000 \text{ X } 1000}$

$$= 1200 \text{ kg/day}$$

Organic loading $= 600 \text{ kg/ha/day}$

∴ Surface area required $= \dfrac{1200 \text{ kg/day}}{600 \text{ kg/ha/day}}$

$$= 2 \text{ Ha}$$

$$= 2 \text{ x } 10^4 \text{ m}^2$$

Assuming the length of the tank (L), as twice of its width (B), we have

$$B \times 2B = 2 \times 10^4 M^2$$

$$\therefore B = \sqrt{\frac{2 X 10^4}{2}}$$

$$= 100 \text{ mtr.}$$

$$\therefore \text{Length} = 2 \times 100 = 200 \text{ mtr.}$$

Using a tank with effective depth as 1.8 m; we have,

$$\text{The provided capacity} = 200 \times 100 \times 1.8$$

$$= 36000 \text{ m}^3$$

Now, capacity = sewage flow per day X Detention time in days

$$\therefore \text{Detention time days} = \frac{\text{Capacity in } m^3}{\text{Sewage flow per day in } m^3/\text{day}}$$

$$= \frac{36000}{3000}$$

$$= 12 \text{ days} > 6 \text{ days}$$

Hence use two oxidation ponds with length

= 200 m x 100 m

and over all depth = (1.8 +1) = 2.2 mtr. and a detention period of 6 days.

Design of inlet pipe

Assuming an average velocity of sewage as 0.9 m/sec. and daily flow for 8 hours only,

$$\text{Discharge} = \frac{3000/2}{8 \text{ X } 60 \text{ X } 60} \text{ m}^3/\text{Sec.}$$

$$\therefore \text{Area of inlet pipe required} = \frac{\text{Discharge}}{\text{velocity}}$$

$$= \left(\frac{3000}{2 \text{ X } 8 \text{ X } 60 \text{ X } 60}\right) \frac{1}{0.9} \text{ m}^2$$

$$= 578.7 \text{ cm}^2$$

$$\therefore \text{Dia of inlet pipe} = \sqrt{\frac{4 \, X \, 578.7}{\pi}} = 27.14 \text{ cm.}$$

say = 28 cm

Diameter of outlet pipe may be taken as 1.5 times that of the inlet pipe.

ie = 1.5 x 28

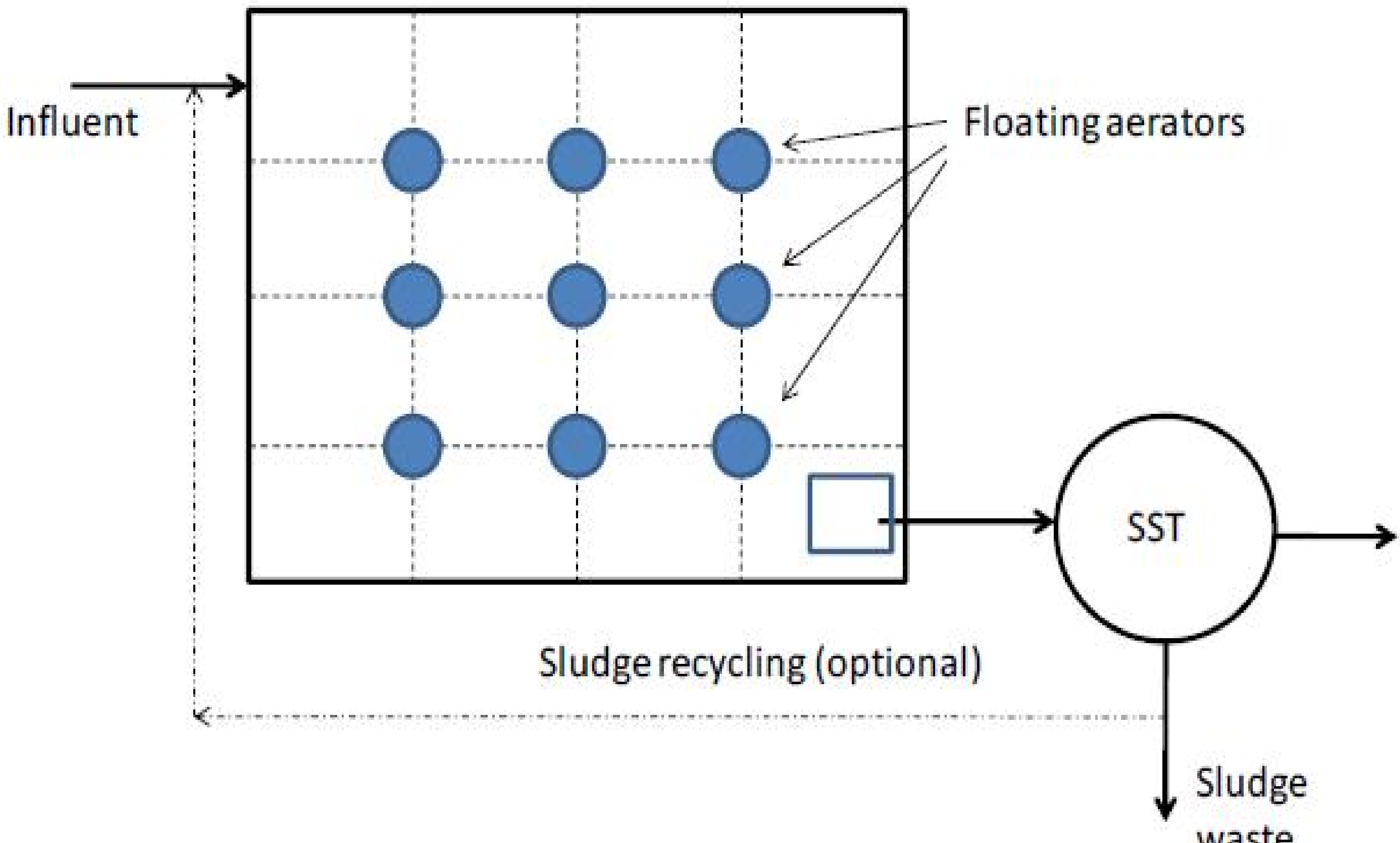
Say = **42** cm

AERATED LAGOONS

Aerated Lagoon Aerated lagoons are one of the aerobic suspended growth processes. An aerated lagoon is a basin in which wastewater is treated either on a flow through basis or with solids recycle. Oxygen is usually supplied by means of surface aerators on floats or on fixed platforms or diffused air aeration units instead of photosynthetic oxygen yield as in case of oxidation pond.

AERATED LAGOONS

The action of the aerators and that of the rising air bubbles from the diffuser are used to keep the contents of the basin in suspension. They are constructed with depth varying from 2 to 5 m. The contents of an aerobic lagoon are mixed completely. Depending on detention time, the effluent contents of about 1/3 to ½ the value of the incoming BOD in the form of cell tissue. Before the effluent can be discharge, the solids must be removed by settling. If the solids are returned to the lagoon, there is no difference between this and modified ASP.


Influent
Floating aerators
SST
Sludge recycling (optional)
Sludge
waste

AERATED LAGOONS

The action of the aerators and that of the rising air bubbles from the diffuser are used to keep the contents of the basin in suspension. They are constructed with depth varying from 2 to 5 m. The contents of an aerobic lagoon are mixed completely. Depending on detention time, the effluent contents of about 1/3 to ½ the value of the incoming BOD in the form of cell tissue. Before the effluent can be discharge, the solids must be removed by settling. If the solids are returned to the lagoon, there is no difference between this and modified ASP.

MODULE VI

TREATMENT AND DISPOSAL OF SLUDGE

There are two end products obtained from various wastewater treatment plants: (i) effluent (ii) sludge. The treated effluent is directly discharged either in the receiving water, or on land. However, the sludges are to be first processed before their final disposal. Objectives of processing sludge are extract water from the solids and dispose the dewatered residue through a combination of physical, chemical and biological operations. It is necessary to treat properly or dispose the sludge generated during the various stages of wastewater treatment like primary sedimentation, secondary sedimentation and sludge generated from advanced (tertiary) treatment, if any. The quantity of sludge generated depends upon the degree of treatment or quality of treated effluent required

Primary Sludge: Sludge settled in primary settling tanks comes under this category which contains 3% to 7% solids out of which approximately 60% to 80% are organic. Primary sludge solids are usually gray in color, slimy, fairly coarse, and with highly obnoxious odors. This sludge is difficult to dewater without treatment, hence digestion is necessary. This type of sludge can be digested readily by aerobic or anaerobic bacteria under favorable operating conditions.

Secondary Sludge: This type of sludge from secondary settling tanks has commonly a brownish, flocculent appearance and an earthy odor. It consists mainly of microorganism containing 75% to 90% organic fraction and remaining inert materials. The organic matter may be assumed to have a specific gravity of 1.01 to 1.05, depending on its source, whereas the inorganic particles have high a specific gravity of about 2.5.

Tertiary Sludge: The nature of sludge from the tertiary (advanced) treatment process depends on the unit process followed like membrane processes or chemical methods, etc. Chemical sludge from phosphorus removal is difficult to handle and treat. Tertiary sludge from biological nitrification and denitrification is similar to waste activated sludge.

SLUDGE TREATMENT PROCESSES

Sludge treatment may include all or a combination of the following unit operations and processes.

1. Thickening or concentration
2. Digestion
3. Conditioning
4. Dewatering
5. Drying
6. Incineration

1. Thickening or concentration

The purpose of thickening is to reduce moisture content of the sludge, and consequently to increase the solids concentration. This practice is adopted for the separation of greater amount of water from the sludge solids than can be attained in settling tanks. This will help in following:

1. Increasing the loading on the digester, requiring lesser digester volume,
2. Increase feed solids concentration to vacuum filters,
3. Economize transport and handling cost of sludge within the plant and final disposal,
4. Minimize land required and handling cost for final disposal of the digested sludge on land, and
5. Save fuel if incineration is practiced.

The thickening of the sludge can be achieved either by:

1. Gravity thickening,
2. Air floatation or
3. Centrifugation.

i. Gravity thickening

Gravity thickening is accomplished in a tank similar in design to a sedimentation tank. This is most commonly used for concentrating the sludge for achieving saving in the digester volume and sludge handing cost. This is used for primary sludge and for combine primary and secondary sludge, and it is not suitable for ASP sludge alone. When the ASP sludge is more than 40% (weight ratio) of the total combined sludge, gravity thickening is not effective and other methods of thickening have to be considered. Gravity thickeners can be operated either as continuous flow or fill and draw type, with or without chemical addition. The thickened sludge is withdrawn from the bottom of the tank and pumped to the digester. The supernatant is returned to the primary sedimentation tank.

ii. Air floatation

By applying air under pressure or vacuum the thickening of the sludge can be achieved. This is normally preferred for ASP sludge. This requires additional equipment, power for operation, skilled supervision for operation and maintenance, hence it is costly. However, better removal of oil and grease, solids, and odour control are the advantages offered by this method. Addition of alum and polyelectrolytes can increase the efficiency of the flotation unit. Alum will increase the sludge but polyelectrolyte will not increase the solids concentration but improves solids capture from 90 to 98%. The floatation units can be of two types (i) pressure type and (ii) vacuum type.

Pressure type floatation unit: In pressure type floatation unit, a portion of the subnatant is pressurized from 3 to 5 kg/cm^2 and then saturated with air in the pressure tank. The effluent from the pressure tank is mixed with influent sludge immediately before it is released into flotation tank. Once the pressure is released, excess dissolve air rises up in the form of extremely small air bubbles, attaching themselves to the sludge particles. This imparts buoyancy to the sludge particles and forms sludge blanket at the top, which is skimmed off, while the unrecycled subnatant is returned to the wastewater treatment plant. Dissolved air floatation (DAF) offers significant advantages in thickening light sludge such as activated sludge.

Vacuum type floatation: The vacuum type floatation unit employs the addition of air to saturation and applying vacuum to the unit to release the air bubbles which float the solids to the surface. The solids concentrated at the surface are skimmed off.

iii. Centrifugation

Thickening by centrifugation is used only when the land available is limited and sludge characteristics will not permit adoption of other methods. This will require high maintenance and operational cost. A centrifuge acts both ways to thicken and to dewater sludge. The centrifuge process separates liquid and solid by the influence of centrifugal force which is typically 50 to 300 times that of gravity.

2. Digestion

The principal objective of sludge digestion is to subject the organic matter present in the settled sludge to anaerobic or aerobic decomposition so as to make it innocuous and amenable to dewatering on sand beds or mechanical filters before final disposal.

In anaerobic digestion process the organic material, in mixture of primary settled sludge and biological sludge from secondary clarifier, is converted to CH_4 and CO_2 under anaerobic conditions. This is carried out in an air tight reactor in absence of oxygen. Sludge is introduced continuously or intermittently and retained in the reactor for varying periods of time. Two basic processes involved in anaerobic digestion are liquifaction and gasification.

2. Digestion

The stabilized sludge which is withdrawn continuously or intermittently from the process, is non putrescible, and its pathogen content is also greatly reduced. Anaerobic digestion is defined as being biological oxidation of degradable organic sludge by microbes under anaerobic condition. It occurs in absence of oxygen and organic matter acts as food source for microorganisms. Most microbes used in this digestion are obligate anaerobes or facultative type. This process is employed for treatment of the organic sludge.

3. Conditioning

Conditioning improves the drainability of digested sludge. Prior conditioning of sludge before application of dewatering methods renders it more ameable to dewatering. Conditioning can be achieved by various methods such as elutriation, chemical conditioning, heat treatment, freezing etc. Chemical conditioning of sludge with or without elutriation is necessary when dewatering of sludge is accomplished by vacuum filtration.

4. Dewatering

The purpose of dewatering is to further reduce the volume of sludge and there by increase the solids concentration. Most of the digested primary or mixed sludges can be compacted to a water content of about 90% in the digester itself by gravity. How ever, further dewatering is acomplished either by air drying on sand drying beds, or by mechanical means such as vaccum filtration, centrifugation, pressure filtration etc.

5. Drying

The purpose of heat drying is to reduce further the moisture content and volume of dewatering sludge. So that it can be used after drying without causing offensive odours or risk to public health. Methods are controlled heat, flash drying, rotary kiln, multiple-hearth furnace etc. Drying is brought about by directing a stream of heated air or other gases about 350^{o} C. The hot gases, dust and ash released during combustion are to be removed by suitable control mechanisms to minimise air pollution.

6. Incineration

Incineration involves the combustion of the sludge in a reactor under high temperature, along with auxiliary fuels, if needed. The purpose of incinertion is to destroy the organic material, the residual ash being generally useful as land fill. During the process, all the gases released from the sludge are burnt off and all the organisms are destroyed. Dewatered or digested sludge is subjected to temperature between 650^{o} C to 750^{o} C. Cyclone or multiple hearth and flash type furnaces are used with proper heating arrangements with temperature control and drying mechanisms.

Advantages of incineration

1) It ensures complete destruction of pathogenic bacteria.
2) There is no odour trouble.
3) Some cost can be recovered by selling the steam power and clinkers.
4) Disposal site can be conveniently located within the city.
5) It requires less space.

Disadvantages of incineration

1) It is a costly method
2) Smoke, odour and ash nuisance may result due to the improper and incompetent operation of the plant, particularly if substances giving high calorific values.

8.3 QUANTITY OF SLUDGE

The quantity of sludge, either in bulk or weight at a particular sewage treatment unit can be found out if the following three factors are known.

1. Amount of suspended solids in the incoming sewage.
2. Likely settlement to occur in the treatment unit.
3. Moisture content of the sludge.

$$V_{sl} = \frac{W_S}{\rho_w S_{sl} P_s}$$

Where,

V_{sl} = volume of sludge

W_S = weight of dry solids (kg)

ρ_w = density ofwater (10^3 kg/m^3 at 5°C)

S_{sl} = specific gravity of sludge

P_s = percent solids expressed as decimal

Problem 8.1

A trickling filter plant treats 1220 m^3/day of sewage with a BOD_5 of 230 mg/L and suspenedsolids of 240 mg/L. Estimate the total solids production, assuming that primary clarifier removes 30% of BOD and 60% of influent solids.

Solution:

Removal of solids in primary clarifier

$$= 0.60 \times 240 = 144 \text{ mg/L}$$

BOD of influent entering trickling filters

$$= (1 - 0.3) \times 230 = 161 \text{mg/L}$$

Assuming that solids production in the trickling filter is @ 0.5 kg/kg of applied BOD production of solids in secondary clarifier

$$= 0.5 \times 161 = 80.5 \text{ mg/L}$$

$\therefore$ total solid production $= 144 + 80.5 = 224.5$ mg/L/day

$$= \frac{224.5\,(1220 \times 10^3)}{10^3 \times 10^3} \text{ kg/day}$$

$$= 273.89 \text{ kg/day}$$

Problem 8.2

A sedimentation tank treats 6 MLD containing 250 mg/L of suspended solids. The tank removes 70% of the suspended solids. Compute the weight and volume of the sludge produced daily if the moisture content of sludge is 97%.

Solution:

Weight of solids in sewage $= \dfrac{(6X10^6)X250}{10^3X10^3}$

$= 1500$ kg/day

Since only 70% of the influent solids are removed, weight of solids removed in sedimentation tank

$= 0.7 \times 1500$

$= 1050$ kg/day

Now, volume of wet sludge $V_{sl} = \dfrac{W_S}{\rho_w S_{sl} P_s}$

It is given that moisture content is 97%

$\therefore P_s = 1 - 0.97 = 0.03$

Assume specific gravity = 1.02

$$\therefore \quad V_{sl} = \frac{1050}{1000 \times 1.02 \times 0.03} = 34.3\ m^3$$

Hence weight of sludge = Volume x unit weight of sludge

= 34.3 x (1.02 x 1000)

= 35000 kg.

SLUDGE DIGESTION TANK OR DIGESTERS

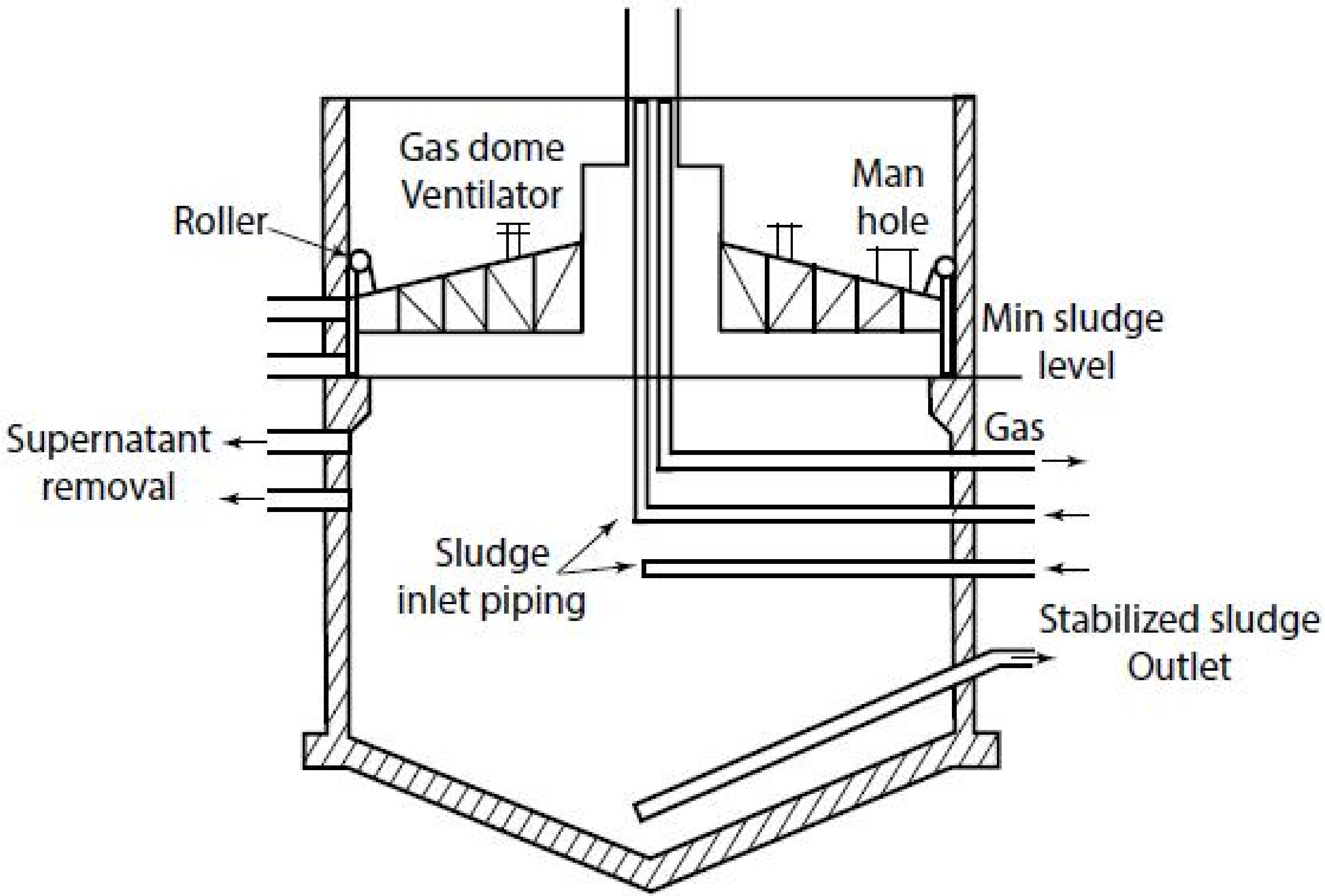


Figure 10.1 Sludge digester

A typical sludge digestion tank is as shown in figure 8.1. It consists of a circular RCC tank with hoppered bottom, and having a fixed or a floating type of roof over its top. The raw sludge is pumped into the tank, and when the tank is first put into operation, it is seeded with the digested sludge from another tank. A screw pump with an arrangement for circulating the sludge from bottom to top of the tank or vice versa is commonly used, for stirring the sludge.

In cold contries, the tank may have to be provided with heating coils through which hot water is circulated in order that the temperature inside the tank is maintained at optimum digestion temperature level. The gases of decomposition (Methane and CO_2) are collected in gas dome. The digested sludge which settles down to the hoppered bottom of the tank is removed under hydrostatic pressure, periodically, once a week. The supernatant liquor lying between the sludge and the scum is removed at suitable elevations, through a number of withdrawal pipes as shown. The supernatant liquor, being higher in BOD and suspended solids contents, is sent back for treatment along with the raw sewage in the treatment plant. The scum formed at the top surface of the supernatant liquor is broken by the recirculating flow or through the mechanical rakers called scum breakers.

The digestion tanks are cylindrical in shape with

The digestion tanks are cylindrical in shape with diameter ranging between 3 to 12 m. The bottom hoppered floor tank of slope about 1:1 to 1:3 and the depth is kept about 6 m or so.

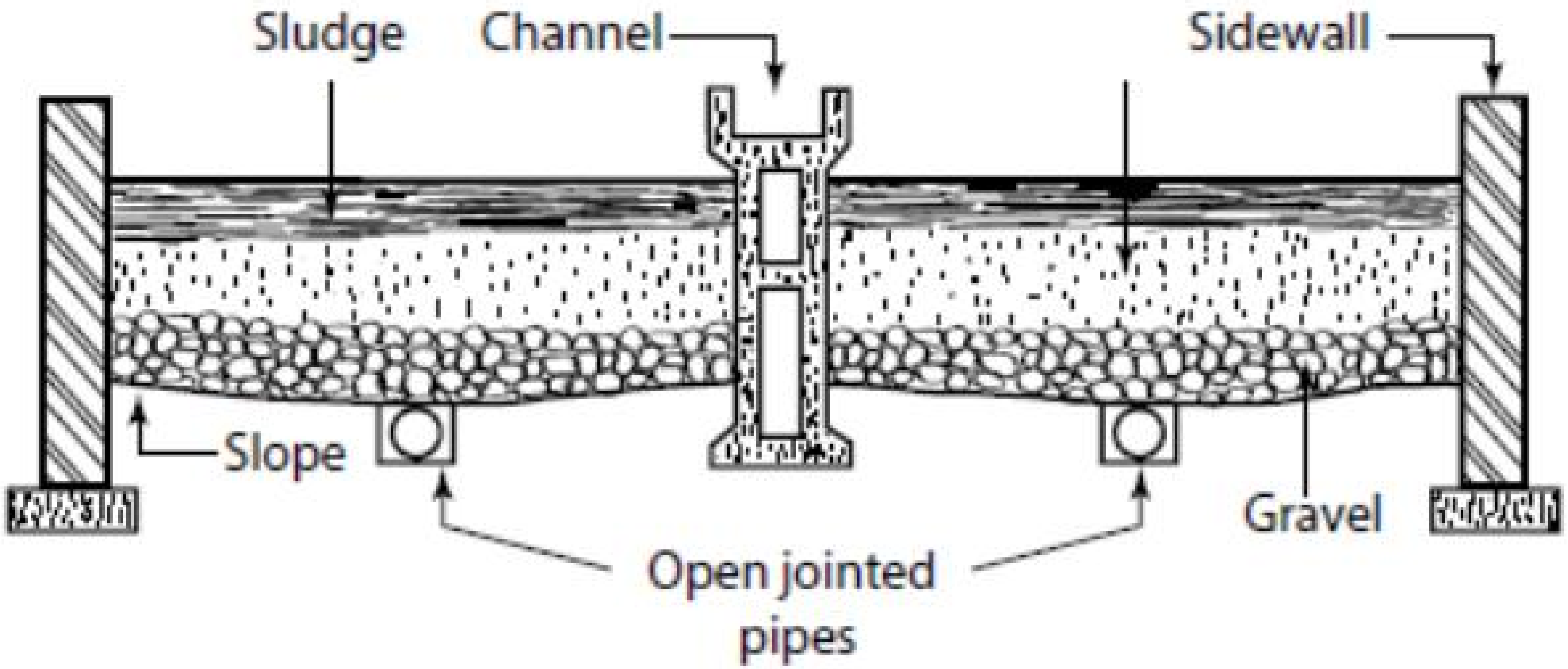


Figure 10.2 Sludge drying bed

Sludge drying beds are open beds of land, 45 to 60 cm deep, and consisting of about 30 to 45 cm thick graded layers of gravel or crushed stone varying in size from 15 cm at bottom to 1.25 cm at top, and overlaying by 10 to 15 cm thick coarse sand layer. Open jointed under-drain pipes @ 5 to 7 m c/c spacing are laid below the gravel layer in valleys, as shown in figure 8.2, at a longitudinal slope of about 1 in 100. The beds are about 15 x 30 m in plan, and are surrounded by walls rising about 1 meter above the sand surface, as shown.

The sewage sludge from the digestion tank is brought and spread over the top of the drying beds to a depth of about 20 to 30 cm, through distribution troughs having openings about 15 cm x 20 cm at a distances of about 2 m or so.

A portion of the moisture drains through the bed, while most of it is evaporated to the atmosphere. It usually takes about two weeks to two months, for drying the sludge, depending on the weather and condition of the bed. The dried sludge is generally used as manure in our country, as it contains nitrogen, phosphorus and potash. It may also be used for filling up low lying areas. It may sometimes be disposed of by burning.

METHODS OF SLUDGE DISPOSAL

Following are various methods which are adopted for disposal of sludge.

1. Spreading on farm land
2. Dumping
3. Land filling
4. Sludge lagooning
5. Disposing in water or sea

1. Spreading on farm land

Dewatered sludge may be disposed of by spreading over farm land and ploughing under after it has dried. Wet dewatered sludge can be incorporated into soil directly by injection. Usually a number of shallow trenches, 50 to 90 cm wide and 0.3 to 0.4 m deep are provided about 1 to 1.5 m apart, and wet sludge is discharged into it. After a sludge cake is formed due to evaporation of water, it is covered with dry earth. After a month, the whole land is ploughed and used for cultivation.

2. Dumping

Dumping in an abandoned mine quarry can be resorted to only for sludges and solids that have been stabilized so that no decomposition or nuisance conditions will result. This method can be safely adopted for digested sludge, clean grit and incinerator residue.

3. Land filling

If a suitable site is convenient, a sanitary land fill can be used for disposal of sludge, grease, grit and other solids, whether stabilised or not. However, dewatering is recommended before such disposal, so that the cost of hauling the sludge is reduced. In a true sanitary landfill, the waste are deposited in a designated area, compacted in place with a tractor or roller and covered with 30 cm layer of clean soil. The sanitary land fill method is most suitable if it is also used for disposal of the other solids wastes of the community. However, drainage from the site that would cause pollution of ground water supplies or surface streams must be guarded against.

4. Sludge lagooning

A lagoon is a shallow earth basin into which untreated or digested sludge is deposited. Untreated-sludge lagoons stabilize the organic solids by anaerobic and aerobic decomposition, which may give rise to objectionable odours. Hence the lagoons should be located away from the town. The depth of the lagoon may vary from 0.5 to 1.5 m. The depth of the lagoon and its area should be about twice that is required for sand drying under comparable conditions. Agricultural tile drains of about 10 cm dia. are placed at 3 m centres at the bottom of the lagoon, and a 15 cm thick layer of ashes or clinker is placed over it to facilitate drain of water from wet sludge.

The detention time may vary from 1 to 2 months. After the sludge has been stabilized and the moisture is drained/evaporated, the contents of the lagoon are dug out to about half of its volume and used as manure. Lagoons are less expensive to build and operate; they have been resorted to, particularly for digested sludge in areas where large open land suitably located is available. Use of lagoons is not generally desirable, as they present an ugly sight and cause odour and mosquito breeding.

5. Disposing in water or sea

This is not common method of disposal because it is contingent on the availability of a large body of water adequate to permit dilutions. At some sea coast sites, the sludge either raw or digested may be barged to sea far enough to make available the required dilution and dispersion. The method requires careful consideration of all factors for proper design and siting of outfall to prevent any coastal pollution or interference with navigation.

Thanks