

WATER SUPPLY AND WASTE WATER **ENGINEERING** Simple designs



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WATER SUPPLY AND WASTE WATER
ENGINEERING

Simple designs

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PREFACE

I feel very happy in presenting the book titled on “**WATER SUPPLY AND WASTE WATER ENGINEERING (SIMPLE DESIGN)**”.

This book slightly covered the designs in waste water and water supply engineering elements used in treatment of water and sewage.

I hope that this book will be very much useful for the civil engineering students in diploma, degree courses and also working engineers.

I dedicated this book to my parents, my professors in KGISL institute of technology, Coimbatore and my lectures in Nanjiah Lingammal polytechnic college, Mettupalyam.

MAY
2020

By
AUTHOR

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DESIGN OF SEDIMENTATION TANK

Filtration of water

- The detention time usually ranges between 4 to 8 hours for plain sedimentation and 2 to 4 hours when coagulants are used.
- The width of the tank is normally kept equal to 10m and not allowed to exceed 12m or so. $B < 12\text{m}$ $B > 10\text{m}$ ($B = 10\text{m to } 12\text{m}$)
- The length of the tank is not generally allowed to exceed four times the width. $L < 4B$
- Depth ranges between 3.0m to 4.5m
- The horizontal flow velocity ranging between 0.15m/min to 0.9m/min, normally kept at about 0.3m/min
- The total amount of flow from the tank within 24 hours generally equals the maximum daily demand of water.
- The maximum diameter of a circular tank may be kept 60m or so.

FILTRATION OF WATER

1. Design a continuous type sedimentation tank for a town with a population of 60,000 persons and daily consumption of 200 liters per head. Assume the detention period as 4 hours.

Given data:

Population: 60000

Daily consumption: 200 lpd

Detention period: 4 hours

Rd: design of continuous type sedimentation tank.

Quantity of water to be treated in 24 hours = $60,000 \times 200$

= 12×10^6 liters

Quantity of water to be treated during the detention period of 4 hours

$$=(\text{quantity of water}/24 \text{ hrs}) \times \text{detention period}$$

$$=(12 \times 10^6/24) \times 4$$

$$=2 \times 10^6 \text{ liters}$$

$$=(2 \times 10^6)/1000 \quad (1\text{m}^3=1000\text{l})$$

$$\text{Capacity of tank} = 2000 \text{ m}^3$$

Assume the velocity of flow to be maintained through the tank = 0.3m / minute.

The length of the tank = velocity of flow x detention period

$$=0.3 \times (4 \times 60) \quad (1 \text{ hr} = 60\text{min})$$

$$=72 \text{ m}$$

Cross sectional area of tank required = capacity of the tank / length of the tank

$$= 2000/72 = 27.78\text{m}^2$$

Assume the water depth = 3.0m

Width of the tank:

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

$$\text{Cross sectional area} = B \times D$$

$$\text{From this, } B = \text{Cross sectional area} / D$$

$$= 27.78/3$$

$$= 9.26\text{m approximately } 10\text{m}$$

(The width of the tank is normally kept equal to 10m and not allowed to exceed 12m or so. $B < 12\text{m}$ $B > 10\text{m}$ ($B = 10\text{m}$ to 12m))

Using free board of 0.5m, the overall depth = $3 + 0.5 = 3.5\text{m}$

A. Horizontal Flow Type

(a) Rectangular tanks with longitudinal flow

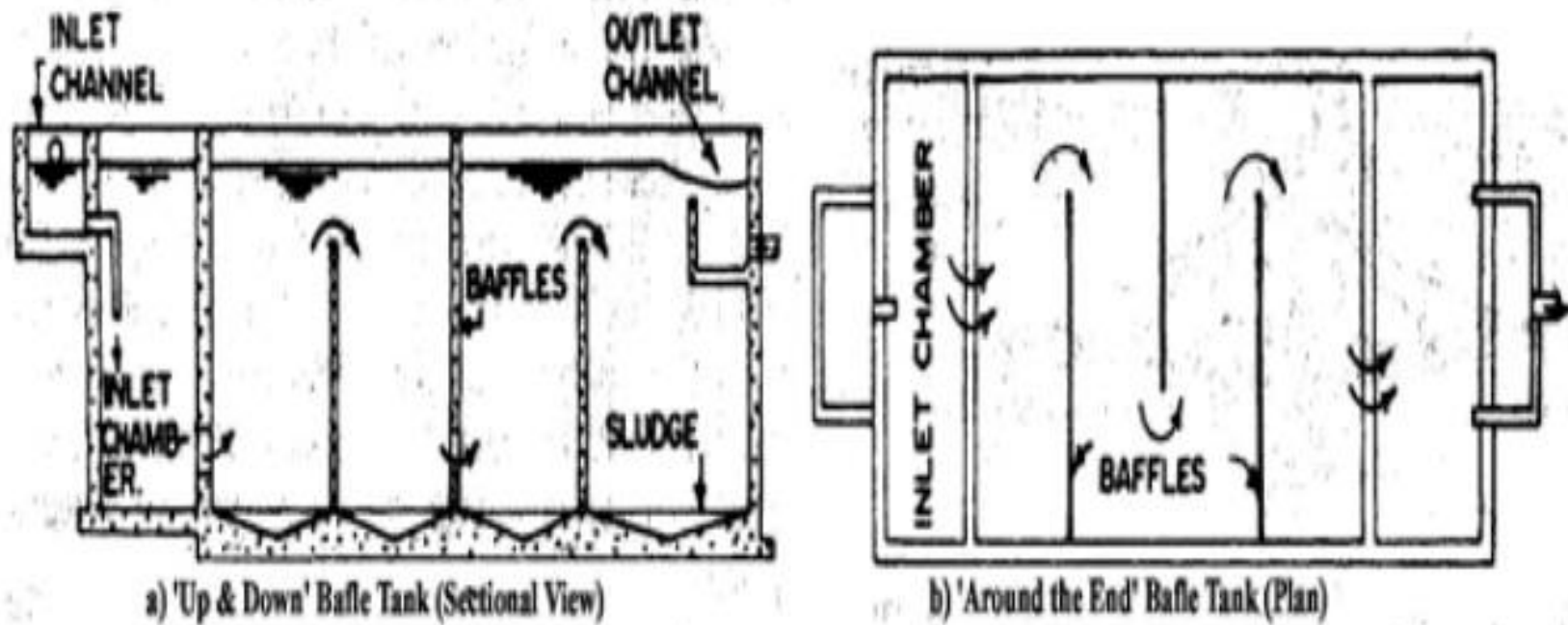


Figure 1

DESIGN OF SEDIMENTATION TANK

Sewage treatment

- Assume 80% of water supplied to the city becomes sewage, we have the quantity of sewage required to be treated per day(i.e.max daily)
- The detention time usually ranges between 1to 2 hours.
- The width of the tank is normally kept equal to 6m and not allowed to exceed 7.5m or so. (B=6m to 7.5m)
- The length of the tank is not generally allowed to exceed four to five times the width. $L=4B$ to $5B$
- Depth ranges between 2.4m to 3.6m(generally not exceeding 3m)
- The horizontal flow velocity of about 0.3m/min.
- The total amount of flow from the tank within 24 hours generally equals the maximum daily flow of sewage.

2.Design a suitable rectangular sedimentation tank (provided with mechanical cleaning equipment) for treating the sewage from a city, provided with an assured public water supply system, with a maximum daily demand of 100×10^5 liters per day.(assume suitable data wherever necessary)

GD: Sewage = 100×10^5 lpd

RD: Design a suitable sedimentation tank.

Assume 80% of water supplied to the city becomes sewage, we have the quantity of sewage required to be treated per day (i.e. max.daily)

$$\begin{aligned} &= (80/100) \times 100 \times 10^5 \\ &= 80 \times 10^5 \text{ liters per day} \end{aligned}$$

Assuming detention period = 2hrs

Quantity of sewage to be treated in 2 hrs = (quantity of sewage/24 hrs) x detention period

$$= (80 \times 10^5 / 24) \times 2$$

$$= 666.67 \times 10^3 \text{ liters}$$

$$\text{Capacity of tank} = 666.67 \text{ m}^3 \text{ approximately } 670 \text{ m}^3 \quad (1 \text{ m}^3 = 1000 \text{ l})$$

Assume the velocity of flow to be maintained through the tank = 0.3m / minute.

The length of the tank = velocity of flow x detention period

$$= 0.3 \times (2 \times 60) \quad (1 \text{ hr} = 60 \text{ min})$$

$$= 36 \text{ m}$$

Cross sectional area of tank required = capacity of the tank / length of the tank

$$= 670 / 36 = 18.61 \text{ m}^2$$

Assume the water depth = 3m

Width of the tank:

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

$$\text{Cross sectional area} = B \times D$$

From this

$$B = \text{Cross sectional area} / D$$

$$= 18.61 / 3$$

$$= 6.203 \text{ m approximately } 6.20 \text{ m}$$

The width of the tank is normally kept equal to 6m and not allowed to exceed 7.5m or so. (B=6m to 7.5m)

Using free board of 0.5m, the overall depth

$$= 3 + 0.5$$

$$= 3.5 \text{ m.}$$

(or)

Instead of assuming the depth,

we may assume an over flow rate = $40000 \text{ l/m}^2/\text{day}$

$$Q / BL = 40000 \text{ l/m}^2/\text{day}$$

Assume 80% of water supplied to the city becomes sewage, we have the quantity of sewage required to be treated per day (i.e. max.daily)

$$= (80/100) \times 100 \times 10^5 = 80 \times 10^5 \text{ liters per day}$$

$$BL = Q/40000 \quad (\text{AREA} = B \times L)$$

$$= 80 \times 10^5 / 40000$$

$$\text{Area} = 200 \text{ m}^2$$

The length of the tank = velocity of flow x detention period

$$= 0.3 \times (2 \times 60) \quad (1 \text{ hr} = 60 \text{ min})$$

$$= 36 \text{ m}$$

Width of the tank = $200 / \text{length of the tank}$ (B = AREA/L)

$$= 200 / 36$$

$$= 5.56 \text{ Approximately } 6 \text{ m}$$

Assume the Depth of the tank (d) = 3m

Volume of the tank = L X B X d

$$= 36 \times 6 \times 3$$

$$= 648 \text{ m}^3$$

Quantity of sewage to be treated in 2 hrs = (quantity of sewage/24 hrs) x detention period

$$= (80 \times 10^5 / 24) \times 2$$

$$= 666.67 \times 10^3 \text{ liters}$$

$$\text{Capacity of tank} = 666.67 \text{ m}^3 \text{ approximately } 670 \text{ m}^3 \quad (1 \text{ m}^3 = 1000 \text{ l})$$

Quantity of sewage to be treated in 2hrs > quantity of tank.

$$666.67 > 648$$

Its fails

So slightly increase the depth of the tank (d) = 3.3m

$$\text{Quantity of the tank} = L \times B \times d = 36 \times 6 \times 3.3 = 712.8 \text{ m}^3.$$

Quantity of sewage to be treated in 2hrs > Quantity of tank

$$666.67 > 712.8$$

HENCE SAFE

ASSUME FREE BOARD = 0.5m

$$\text{Overall depth of tank (D)} = 3.3 + 0.5 = 3.8 \text{ m}$$

Weight

1 tonne	=	1000 kg
1 tonne	=	1.1023 US ton
1 US ton	=	0.9072 tonnes
1 kg	=	2.2046 pounds (lb)
1 kg	=	35.274 ounce (oz)
1 lb	=	0.4536 kg
1 oz	=	28.3495 gram

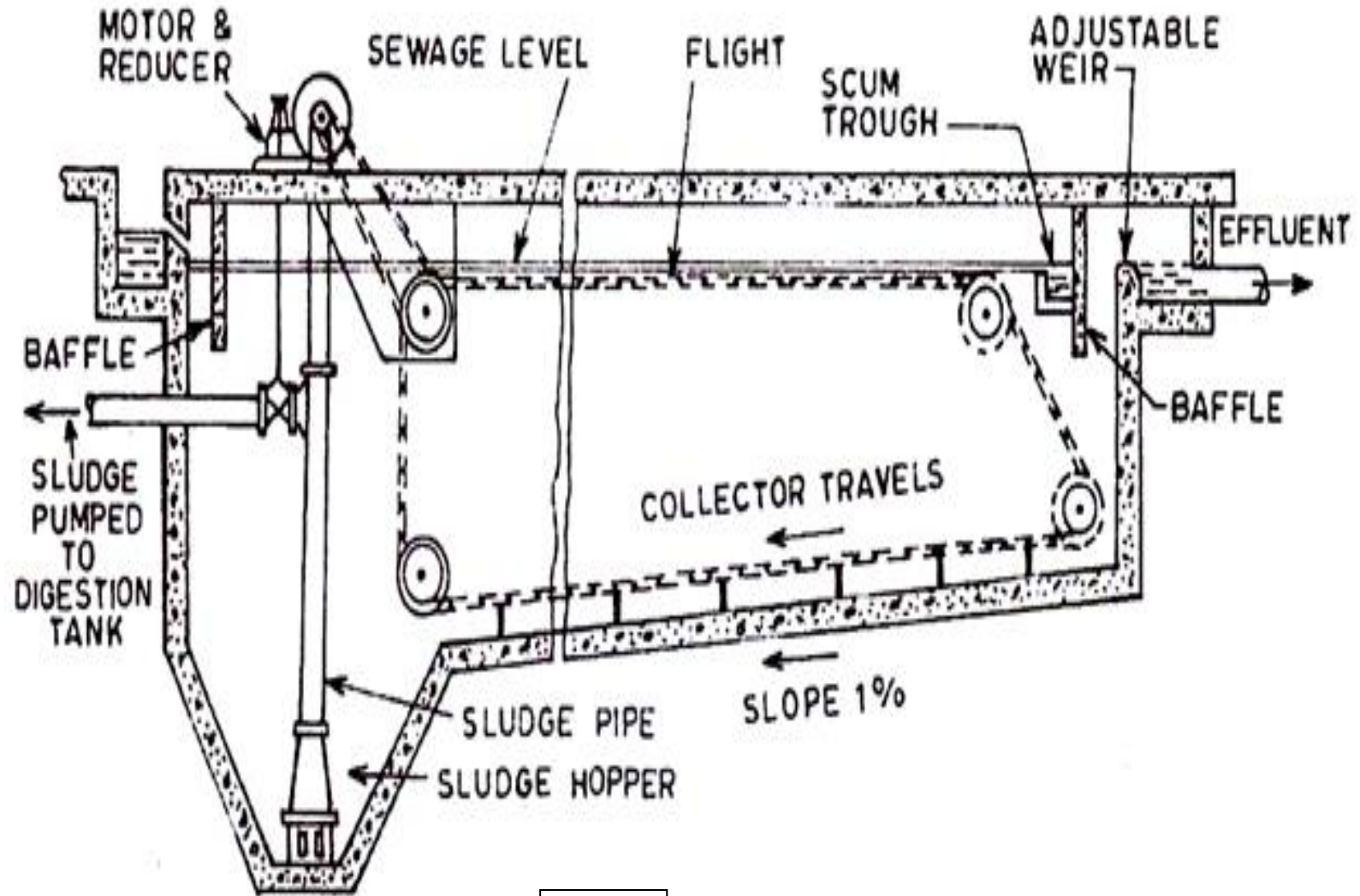


Figure 2

3. Design a circular settling tank unit for a primary treatment of sewage at 10million liter per day. (Assume suitable data necessary)

GD:

Sewage = 100×10^5 lpd

RD: Design a suitable sedimentation tank.

Assume 80% of water supplied to the city becomes sewage, we have the quantity of sewage required to be treated per day (i.e. max.daily)

$$\begin{aligned} &= (80/100) \times 100 \times 10^5 \\ &= 80 \times 10^5 \text{ liters per day} \end{aligned}$$

Assuming detention period = 2hrs

Quantity of sewage to be treated in 2 hrs = (quantity of sewage/24 hrs) x detention period

$$\begin{aligned} &= (80 \times 10^5 / 24) \times 2 \\ &= 666.67 \times 10^3 \text{ liters} \end{aligned}$$

Capacity of tank required = 666.67 m^3 approximately 670 m^3 ($1 \text{ m}^3 = 1000 \text{ l}$)

Assume the velocity of flow to be maintained through the tank = 0.3m / minute.

Assume the Depth of the tank (h) = 3m

Diameter of the tank:

$$\begin{aligned} V &= A \times h \\ 666.67 &= 0.78539 D^2 \times 3 \\ D &= (666.67 / (0.78539 \times 3))^{1/2} \\ &= 16.82 \text{m approximately } 17 \text{m} \end{aligned}$$

$$\begin{aligned}\text{Capacity of tank provided} &= 0.78539 \times 17 \times 17 \times 3 \\ &= 680.933 \text{ m}^3\end{aligned}$$

Capacity of tank required < Capacity of tank provided

$$666.67 < 680.93$$

Hence safe

ASSUME FREE BOARD = 0.5m

$$\text{Overall depth of tank (H)} = 3 + 0.5 = 3.5\text{m}$$

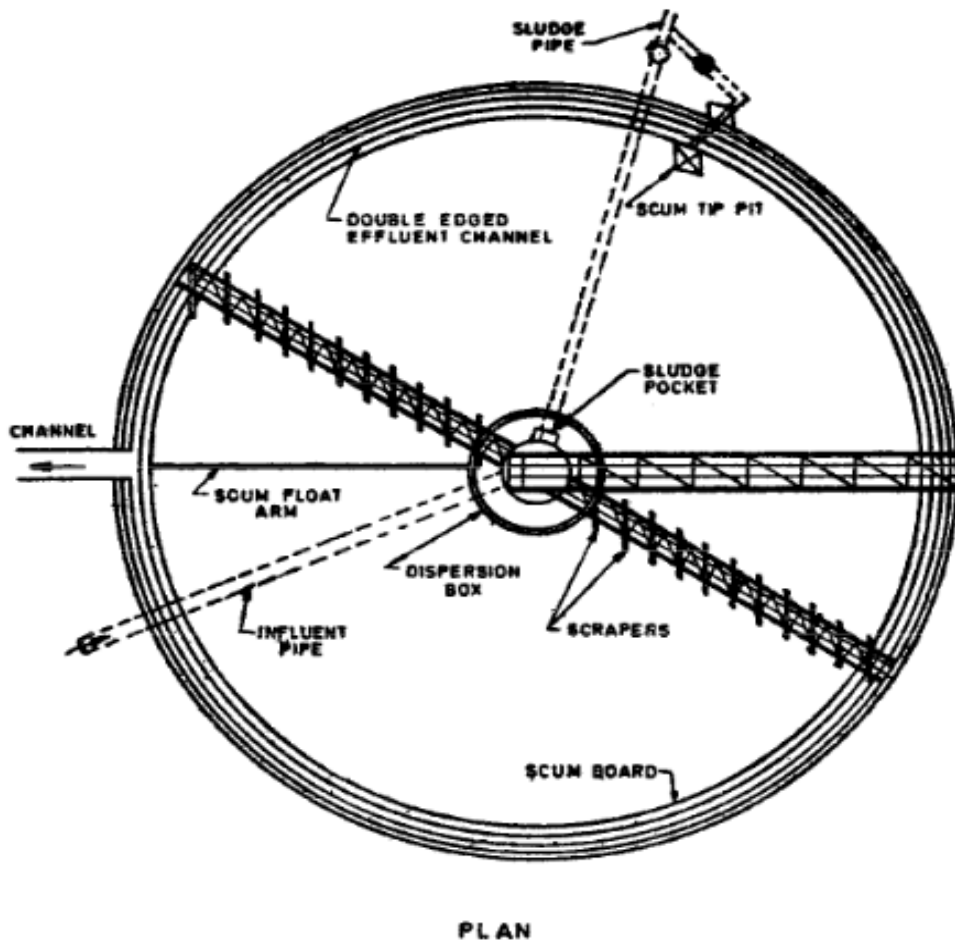


Figure 3

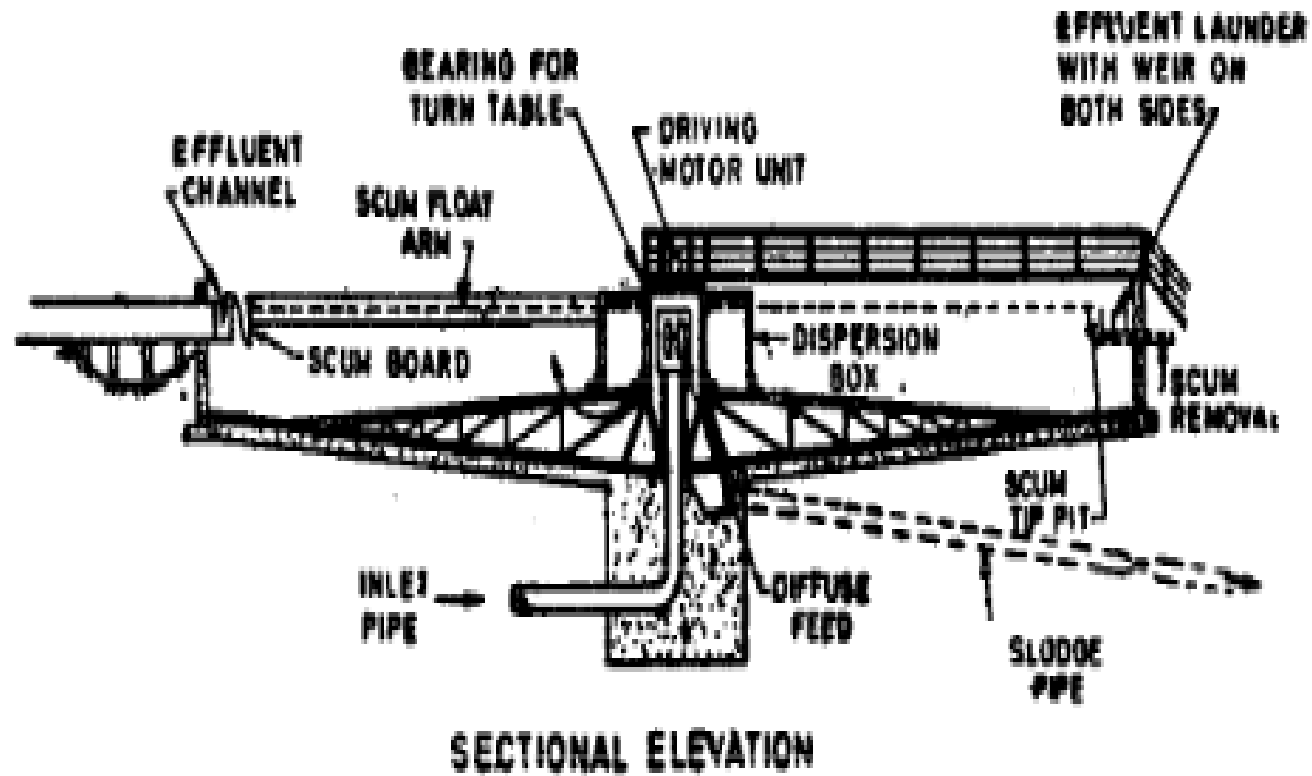


Figure 4

DESIGN OF SLOW SAND FILTER

- **Average daily demand = population x per capita demand**
- **Maximum demand = 1.8 x average daily demand**
- **Total surface area = maximum daily demand/rate of filtration per day**
- **The area of each filter unit = (1/No.of.units) x total surface area**
- **L=1.5B to 2B**
- **Rate of filtration = 100litre/hour/m² to 200litre/hour/m²**
- **Size of each unit = 30m x 60m (100m² to 2000m²)**
- **Depth of tank=2.5m to 3.5m**

4. Design a slow sand filter to treat water for to treat water for a population of 1,00,000 with a daily demand of 200 l/h/d. The rate of filtration 200 l/m²/hour and the maximum demand is 1.8 times the average demand. The length may be taken as twice the width. One of the 10 filters may be kept as a standby.

GD:

Population = 1,00,000

Per capita demand = 200 l/h/d

Rate of filtration = 200 l/m²/hour

Maximum demand = 1.8 times the average demand

RD: Design a slow sand filter

Average daily demand = population x per capita demand

$$= 100000 \times 200$$

$$= 20 \times 10^6 \text{ liters}$$

Maximum demand = 1.8 x average daily demand

$$= 1.8 \times 20 \times 10^6$$

$$= 36 \times 10^6 \text{ liters}$$

Total surface area = maximum daily demand/rate of filtration per day

$$= (36 \times 10^6) / 200 \times 24$$

$$= 7500 \text{m}^2$$

The area of each filter unit = (1/No.of.units) x total surface area

$$= (1/9) \times 7500$$

$$= 833.33 \text{m}^2$$

Length and width:

AREA = LENGTH X BREADTH

$$833.33 = 2B \times B \quad (L=2B)$$

$$833.33 = 2B^2$$

$$B = (833.33/2)^{1/2}$$

$$= 20.41 \text{m approximately } 21 \text{m}$$

$$L = 2B$$

$$= 2 \times 21$$

$$= 42 \text{m.}$$

Units

1 crore = 10 million

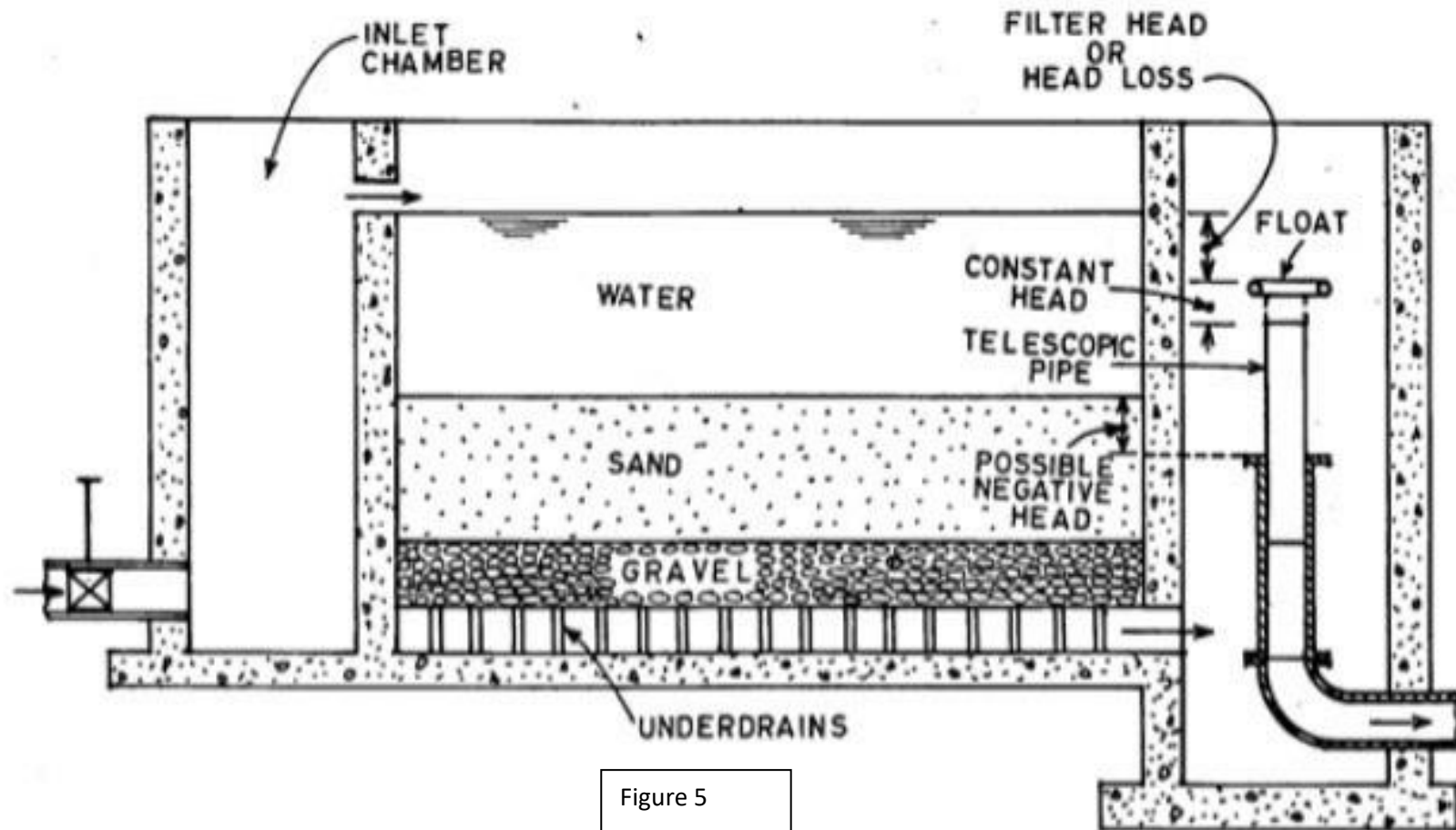
1 million = 10 lakh

1 lakh = 100,000

1 billion = 1000 million

FILTER AND THEIR CLASSIFICATION

1. Slow sand filter (SSF)



DESIGN OF RAPID SAND FILTER

- Average daily demand = population x per capita demand
- Maximum demand = 1.8 x average daily demand
- Total surface area = maximum daily demand/rate of filtration per day
- The number of units at a filter plant may be roughly estimated by the equation developed by morell and Wallace. It's states that

$$N = 1.22\sqrt{Q}$$

N= Number of filter units

Q=Plant capacity in million liters per day

- $L=1.5B$
- Rate of filtration = 3000litre/hour/m² to 6000litre/hour/m²
- Size of each unit = 5m x 8m (10m² to 80m²)
- Depth of tank=2.5m to 3.5m.

5. Design a rapid sand filter to treat water for to treat water for a population of 75,000 with a daily demand of 200 l/h/d.The rate of filtration 5000 l/m²/hour.

GD: Population = 1, 00,000

Per capita demand = 200 l/h/d

Rate of filtration = 200 l/m²/hour

RD: Design a slow sand filter

Average daily demand = population x per capita demand

$$=75000 \times 200$$

$$=15 \times 10^6 \text{ liters}$$

Maximum demand (Q) = 1.8 x average daily demand

$$=1.8 \times 15 \times 10^6$$

$$=27 \times 10^6 \text{ liters (Q=27 MILLION LITERS)}$$

Water demand per hour = maximum demand/hour

$$=27 \times 10^6/24$$

$$=1125000\text{liter}$$

Area of filter beds required = water demand per hour/rate of filtration per day

$$= 1125000/5000$$

$$=225\text{m}^2$$

The number of units at a filter plant (N) = $1.22\sqrt{Q}$

$$=1.22\sqrt{27}$$

$$=6.339 \text{ APROXIMATELY } 7$$

Hence, use 8 filter units with one unit as stand by.

The area of each filter unit = $(1/\text{No.of.units}) \times \text{total surface area}$

$$= (1/7) \times 225$$

$$=32.142\text{m}^2$$

Length and width:

$$\text{AREA} = \text{LENGTH} \times \text{BREADTH}$$

$$32.142=1.5B \times B \quad (\text{L}=1.5B)$$

$$32.142=1.5B^2$$

$$B = (32.142/1.5)^{1/2}$$

$$= 4.62\text{m approximately } 5\text{m}$$

$$L = 1.5B$$

$$=1.5 \times 5 =7.5\text{m approximately } 8$$

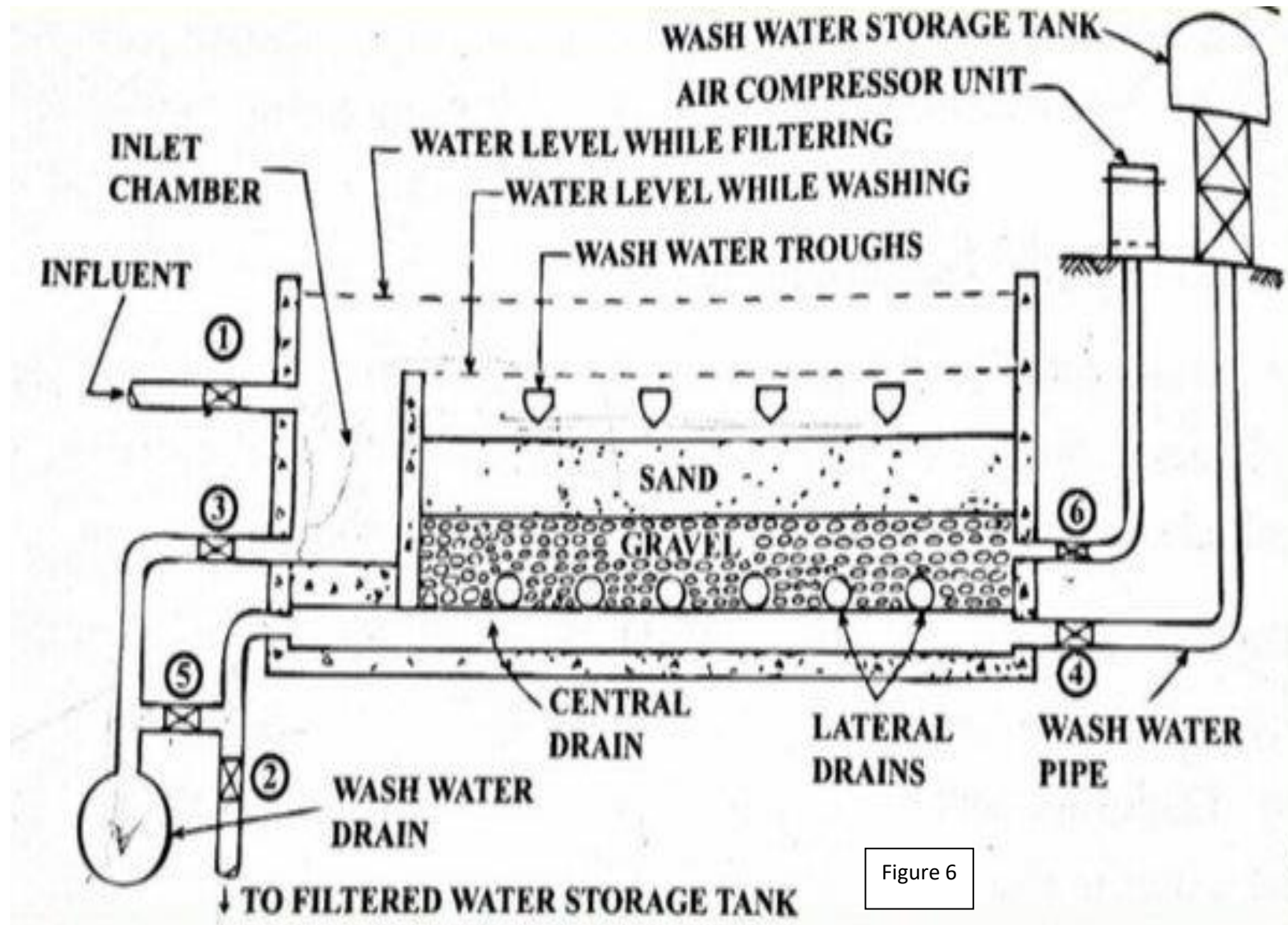


Figure 6

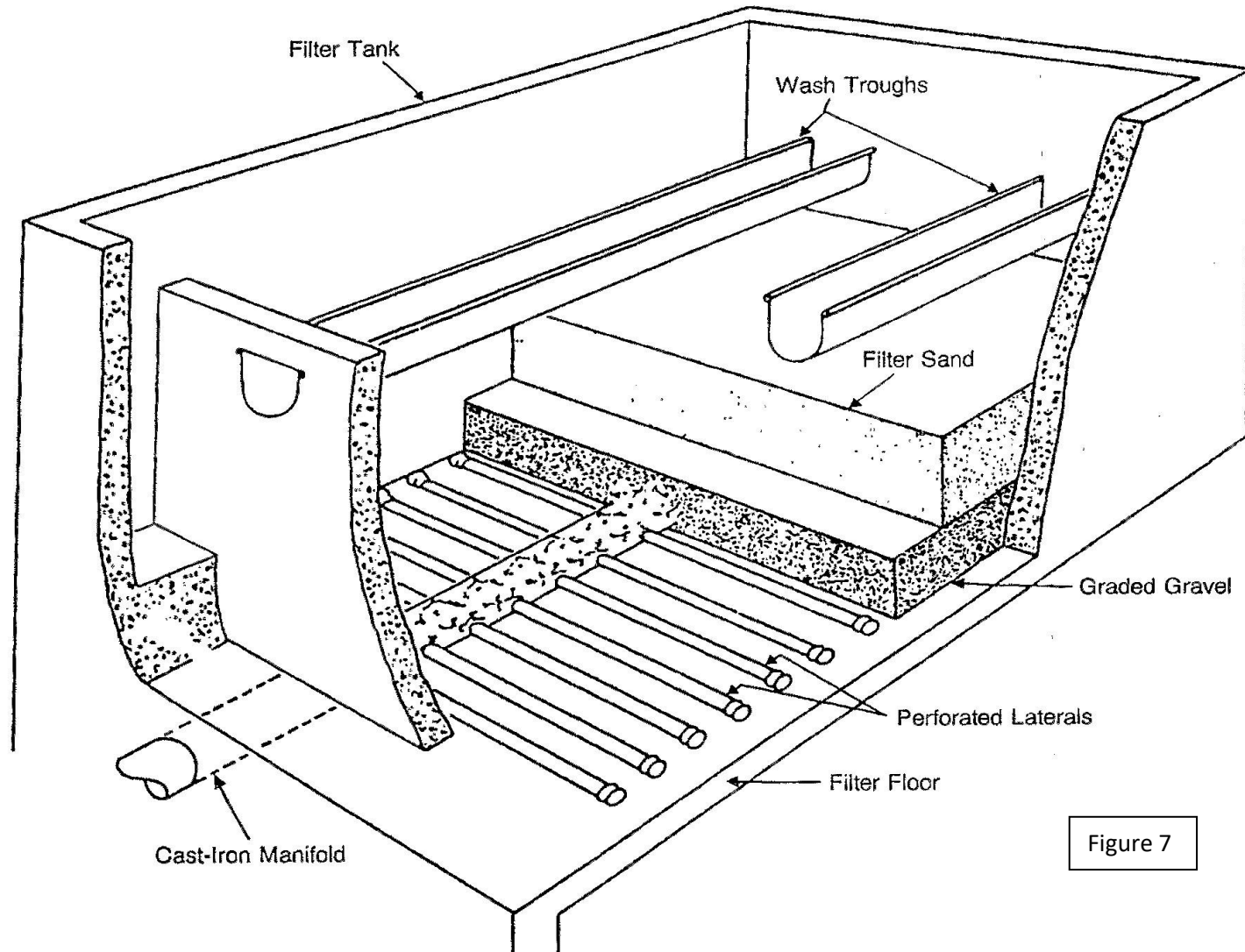


Figure 7

DESIGN OF SEPTIC TANK

Design of Septic tank:

- The digested sludge from the tank is periodically at intervals of 6 months to 12 months and in no case exceeding 3 years.
- Water closet are connected to the septic tank, the sewage flow will be about 40 to 70 liter/capita/day.
- When sullage is also discharged in to the septic tank
- The rate of accumulation of sludge has been recommended as 30liter/person/year.
- The minimum capacity of tank for about 8 to 10 persons may be kept 2250liters (when all liquid wastes are discharged into tank).
- The minimum capacity of tank for about 8 to 10 persons may be kept 1400liters (when only water closet wastes are discharged).
- Free board=0.3m to 0.5m
- Detention period for a septic tank generally varies between 12 to 36 hours, but is commonly adopted as 24 hours.
- Length (L) of the septic tank
= 2 x Width of the septic tank to 4 x Width of the septic tank.
- The depth of the tank = 1.2m to 1.8m
- Scum storage = 0.01m³/capita
- Sludge digestion = 0.028 to 0.056 m³/ capita
- Add 25% extra of storage capacity of sludge
- Sludge Storage capacity/capita:

Period of cleaning	Storage capacity/person
6 months	0.0283 m ³
1 year	0.049 m ³
2 years	0.0708 m ³
3 years	0.085 m ³

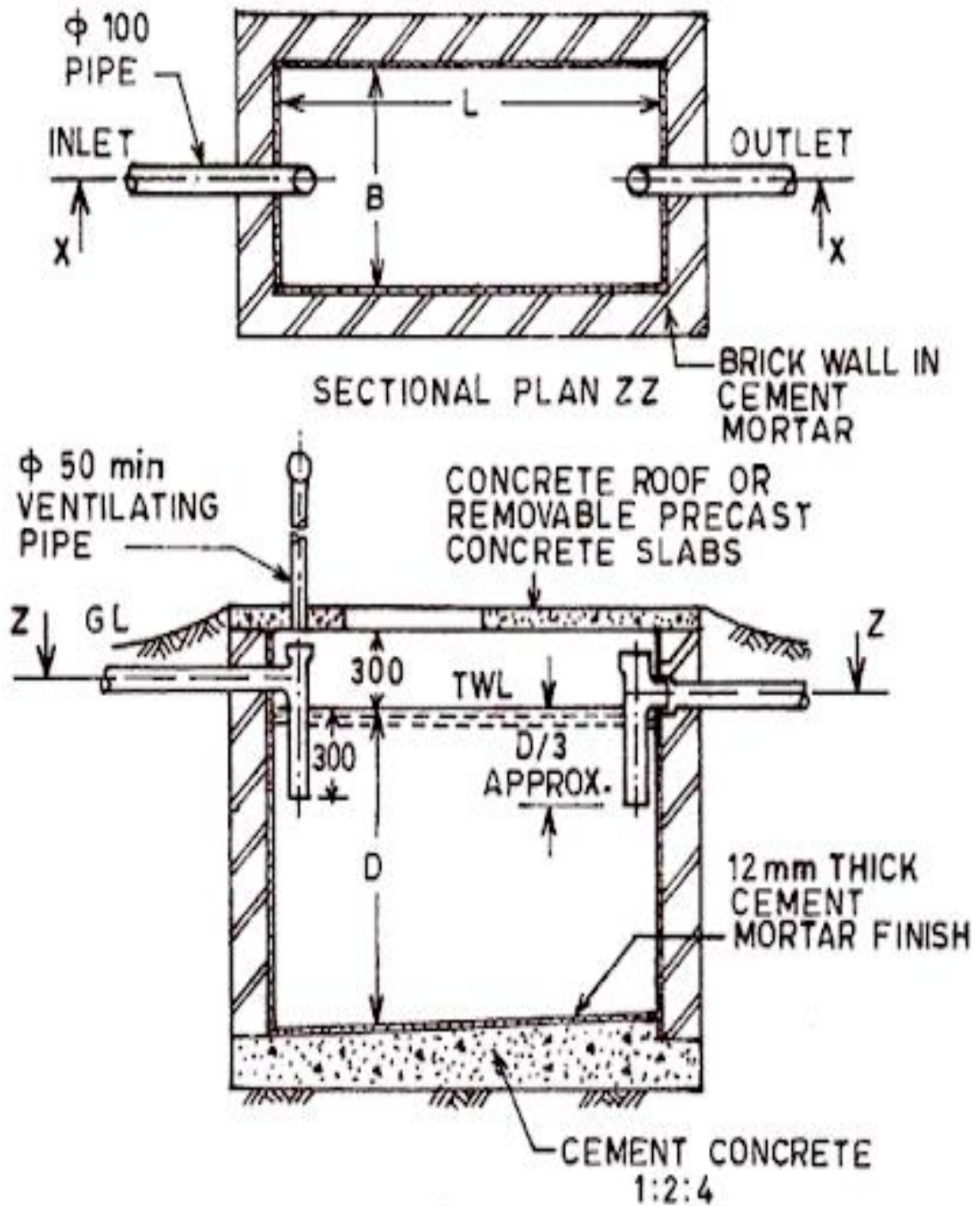


Figure 8

SECTION XX
ALL DIMENSIONS IN MILLIMETRES.

Note (1): The capacities are recommended on the assumption that discharge from only WC will be treated in the septic tank.

Note (2): A provision of 300 mm should be made for free board.

Table 1

Recommended Sizes of Septic Tanks for Residential Colonies

Number or users	Length (m)	Width (m)	Liquid depth (m) (Clearing interval of)	
			1 year	2 years
50	5.0	2.00	1.0	1.24
100	7.5	2.65	1.0	1.24
150	10.0	3.00	1.0	1.24
200	12.0	3.30	1.0	1.24
300	15.0	4.00	1.0	1.24

Note (1): A provision of 300 mm should be made for free board.

Note (2): For population over 100, the tank may be divided into independent parallel chamber for ease of maintenance and cleaning.

Table 2

Recommended Sizes of Septic Tanks for Hostels and Boarding Schools

Number or users	Length (m)	Width (m)	Liquid depth (m) (Clearing interval of)	
			1 year	2 years
50	5.0	1.6	1.3	1.4
100	5.7	2.1	1.4	1.7
150	7.7	2.4	1.4	1.7
200	8.9	2.7	1.4	1.7
300	10.7	3.3	1.4	1.7

Note (1): A provision of 300 mm should be made for free board.

Note (2): For population over 100, the tank may be divided into independent parallel chamber for ease of maintenance and cleaning.

Soil absorption system:

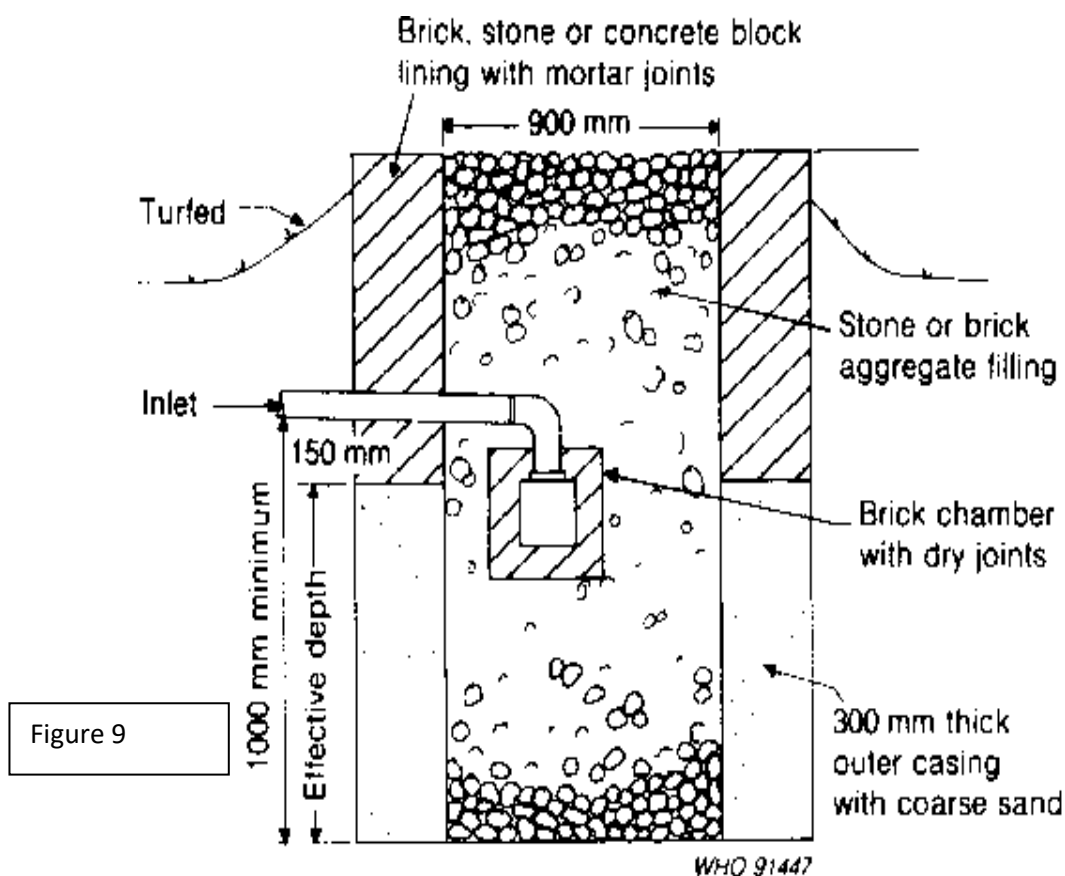
(a) Soak pit or seepage pit:

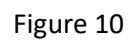
- Diameter of the soak pit is not less than 0.9m.
- Depth of the soak is not less than 1.0m.
- Percolation rate not exceeding 30 minutes.

- **Percolating capacity of filter media of the pit** = $1.25 \text{ m}^3/\text{m}^3/\text{day}$ or $=1250 \text{ l/m}^3/\text{day}$
- **Volume of filtering media required for the soak well**
= sewage outflow/percolation rate.
- **The minimum absorption area of the pits require, shall be worked out on the basis of the maximum allowable rate of effluent application (q):**

$$q = 130 \times vt$$

$$t = \text{standard percolation rate in minute.}$$
- **Area of soak pit** = Quantity of sewage / q.
- **Diameter of the soak pit** = $(\text{area of soak pit}/0.78539)^{1/2}$.





TYPICAL SOIL ABSORPTION SYSTEM WITH DISPERSION TRENCHES

(b)Dispersion trench:

- Width of the trench shall be 0.3m to 1.0m.
- Depth of the trench shall be 0.5m to 1.0m.
- Percolation rate not exceeding 60 minutes.
- Volume of filtering media required for the soak well
= sewage outflow/percolation rate.
- The minimum absorption area of the trenches require, shall be worked out on the basis of the maximum allowable rate of effluent application (q):
$$q = 204 / \sqrt{t}$$
$$t = \text{standard percolation rate in minute.}$$
- Area of Dispersion trench = Quantity of sewage / q.
- Width of trench = Area of Dispersion trench/L.

6. Design a septic tank for 200 users if the rate of water supply as 135 liters per head per day and assume suitable data wherever necessary.

Given data:

200 users

135 l/h/d

Rd: design a septic tank

Quantity of sewage produced per day

= No. of users x rate of water supplying l/h/d

= 200 x 135

= 27000 liters / day = 27 m³/ day

(1m³=1000L)

The quantity of sewage produced during the detention period:

Detention period is commonly adopted as 24 hrs

$$= 27 \times 24 / 24$$

$$= 27\text{m}^3$$

Sludge storage is 0.085m^3 / capita for a cleaning period of 3 years.

$$= 200 \times 0.085$$

$$= 17\text{m}^3$$

Total capacity = quantity of sewage produced during the detention period

+

Sludge storage capacity

$$= 27 + 17$$

$$= 44\text{m}^3$$

Add 25% extra for future expansion:

$$= (25/100) \times 44$$

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

Total design capacity = Total capacity

+

Adding 25% extra for future expansion

$$= 44 + 11$$

$$= 66\text{m}^3$$

Area of septic tank:

Let the depth of liquid be 1.8m(d)

(VOLUME =AREA X DEPTH)

Area = volume/depth

Plan area of the tank = total design capacity / depth of liquid
= 66/1.8
= 36.67m²

Width of the septic tank (B):

Length (L) of the septic tank = 2 x Width of the septic tank to
4 x Width of the septic tank.

(AREA =LENGTH X BREADTH)

Width of the septic tank (B) = Plan area of the tank / Length of the tank

$$B = 36.67 / 4B \quad (L=4B)$$

$$4B^2 = 36.67$$

$$B = (36.67 / 4)^{1/2}$$

$$B = 3.027 \text{ m approximately } 3.2\text{m}$$

Length (L) of the septic tank = 4 x Width of the septic tank
= 4 X 3.2
=12.8m

Overall depth of septic tank (D) = depth of liquid + free board

(Assume free board = 0.5m)

$$= 1.8 + 0.5 = 2.3\text{m.}$$

CHECK:

Sedimentation volume for free space of 0.5m deep

$$\begin{aligned} &= 0.5 \times L \times B \\ &= 0.5 \times 12.8 \times 3.2 \\ &= 20.48\text{m}^3 \end{aligned}$$

Scum storage is 0.01m^3 / person

$$\begin{aligned} &= 0.01 \times 200 \\ &= 2\text{m}^3 \end{aligned}$$

Sludge storage is 0.085m^3 / capita for a cleaning period of 3 years.

$$\begin{aligned} &= 200 \times 0.085 \\ &= 17\text{m}^3 \end{aligned}$$

Sludge digestion is 0.056 m^3 /person (0.028 to 0.056m^3 /person) = 0.056×200

$$= 11.2\text{ m}^3$$

Total volume of septic tank required = $20.48 + 2 + 17 + 11.2$

$$= 50.68\text{ m}^3$$

Total volume provided

$$\begin{aligned} &= L \times B \times d \\ &= 12.8 \times 3.2 \times 1.8 \\ &= 73.728\text{ m}^3. \end{aligned}$$

Total volume of septic tank required < Total volume provided

Hence the design is safe.

a) Design of soak pit:

We know that,

Total design capacity = Total capacity

+

Adding 25% extra for future expansion

= 44 + 11

= 66m³

Assume percolating capacity of filter media of the pit is 1.25m³/m³/day

Volume of pit required = quantity of sewage / percolating rate

= 27/1.25

= 21.6m³

Assume the depth of the pit as 3m below the invert level of the inlet pipe.

Area of soak pit = volume of pit required / depth

= 21.6/3

= 7.2m²

Diameter of the soak pit = (area of soak pit/0.78539)^{1/2}

= (7.2/0.78539)^{1/2}

= 3.02m approximately 3.1m

b) Design of dispersion trench:

Volume of filtering media required for the soak well

= sewage outflow/percolation rate.

(The minimum absorption area of the trenches require, shall be worked out on the basis of the maximum allowable rate of effluent application (q): $q = 204 / \sqrt{t}$

t = assume standard percolation rate is 3minute.)

$$q = 204 / \sqrt{3}$$

$$= 117.77 \text{ l / d / m}^2$$

t = assume standard percolation rate is 3minute.

Area of Dispersion trench = Quantity of sewage / q.

$$= 27000 \text{ l / } 117.77 \text{ l/d/m}^2$$

$$= 229.26 \text{ m}^2.$$

Assume width of the dispersion trench = 1m

Length of the trench = Area of Dispersion trench/ Width of trench L.

$$= 229.26 / 1\text{m} = 229.26\text{m approximately } 230\text{m}$$

Depending upon the availability of space, we may use 4 numbers of trenches

Each length of trenches = $230/4 = 57.5\text{m}$ approximately 60m

Depth of trench = 1.00m.

DESIGN OF MIXING BASIN WITH BAFFLE WALL

WATER TREATMENT

- The velocity of flow in the channels between baffles is 0.15 m/s to 0.45m/s
- The detention period is 20 minutes to 50 minutes.
- Length of flow = velocity x detention period.
- Required cross sectional area of each channel between the baffle:

$$A_{\text{req}} = \text{capacity of tank} / \text{total length of flow}$$

- The distance between baffle should not less than 450mm (0.45m).
- The depth of water in the basin=

$$\text{C/s area of each channel} / \text{distance between the baffle}$$

- The clear opening between the end of each baffle and the wall =
1.5 x distances between the baffle walls

Minimum value of clear opening between the end of each baffle and the wall = 0.6m

- The head loss across a baffled channel = 0.5m to 1.0m.
- The effective flow length of the channel =
Clear width of either half of the tank - 2.5 x clear opening
- The effective depth = 2 to 3 x distance between baffle wall
- Number of channels required =
Total length of the flow/effective flow length of each channel
- The clear length of the tank excluding the baffle walls and side wall =
Number of channels x 0.45
- Assuming the thickness of each baffle = 0.075m
- The overall inside length of the tank =
Clear length of the tank excluding the baffle walls and side wall +
(thickness of baffle x number of baffles)

7. Design the water depth for a mixing basin having around the end baffles, in order to treat 60, 00, 000 lpd of water. The tank may be divided into two similar compartments by providing a longitudinal partition wall, and each half may have a clear width of 10m. Assume suitable values of detention period and the flow velocity through the basin. The clear distance between the baffles may be kept as equal to minimum permissible. Mention the number of channels in the tank and also overall inside length of the tank.

Design:

Total volume of water to treated = 60×10^6 l/day

Assume detention period = 35 minutes

The volume of required to be treated in 35 minutes or 0.583 hours = capacity of the tank

- **$35/60 = 0.583$ hours**

Volume of water = $(60 \times 10^6 / 24) \times 0.583$
= 1457500 liters
= 1457.5 m^3 ($1\text{m}^3 = 1000\text{LITERS}$)

Approximately 1460 m^3

Assume the velocity of flow in the channels between baffles is 0.2 m/s

Length of flow = velocity x detention period
= $0.2 \times 35 \times 60$ (1minute = 60sec)
= 420m.

Required cross sectional area of each channel between the baffle:

$A_{\text{req}} = \text{capacity of tank} / \text{total length of flow}$
= $1460 / 420$
= 3.47m^2 approximately 3.5m^2

The depth of water in the basin=

C/s area of each channel/distance between the baffle

- **Distance between baffle should not less than 450mm (0.45m).**

$$= 3.5 / 0.45$$

$$= 7.78\text{m approximately } 7.8\text{m}$$

The clear opening between the end of each baffle and the wall

$$= 1.5 \times \text{distances between the baffle walls}$$

$$= 1.5 \times 0.45$$

$$= 0.67\text{m approximately } 0.7\text{m}$$

The effective flow length of the channel

$$= \text{Clear width of either half of the tank} - 2.5 \times \text{clear opening}$$

$$= 10 - (2.5 \times 0.7)$$

$$= 8.25\text{m}$$

Number of channels required

$$= \text{Total length of the flow/effective flow length of each channel}$$

$$= 420 / 8.25$$

$$= 50.90$$

Hence there will be $50.90/2 = 25.45$ approximately 26

Approximately 26 channels in each half of the tank

Clear length of the tank excluding the baffle walls and side wall

$$= \text{Number of channels} \times 0.45$$

$$= 26 \times 0.45$$

$$= 11.7 \text{ m.}$$

Overall inside length of the tank

= Clear length of the tank excluding the baffle walls and side wall + (thickness of baffle $\times$ number of baffles)

Assuming the thickness of baffle wall 0.075m

$$= 11.7 + (0.075(26-1))$$

$$= 13.575 \text{ m approximately } 13.58 \text{ m.}$$

Length

1 kilometre (km)	=	1000 metres (m)
1 km	=	0.6214 miles
1 m	=	1.0936 yards
1 m	=	3.2808 feet
1 mile	=	1760 yards
1 mile	=	1.609 km
1 yard	=	0.9144 m
1 foot	=	0.3048 m

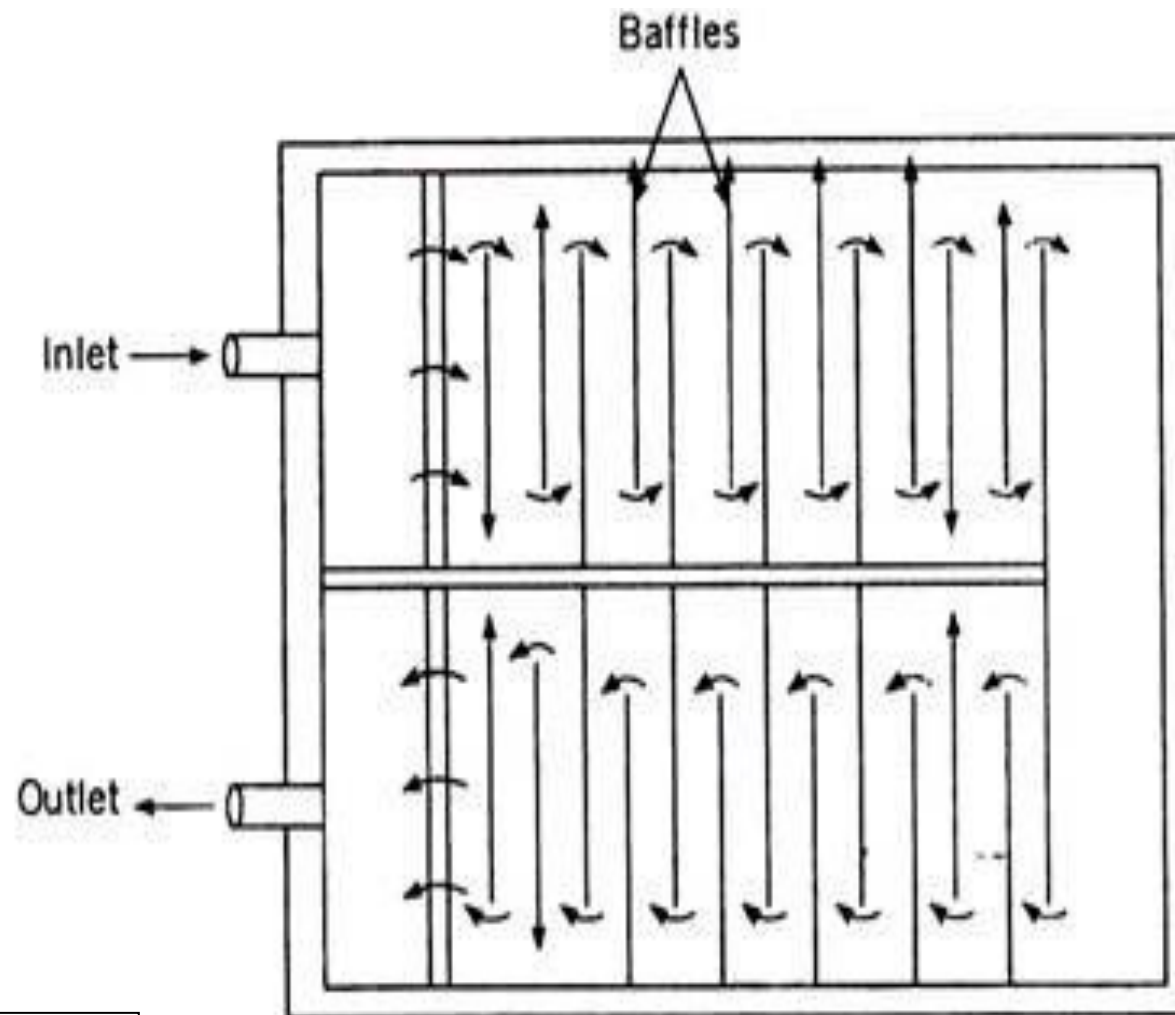


Figure 11

Round about the end baffles type mixing basin.

DESIGN OF COAGULATION-CUM-SEDIMENTATION TANK

(a)DESIGN OF SETTLING TANK:

- Average daily consumption = population x per capita demand
- Maximum daily demand = 1.8 x Average daily consumption
- detention period = 2 to 4 hours
- Quantity of water treated (capacity of tank)
= ((Maximum daily demand/24) x detention period)
- Overflow rate = 1000 to 1250 l/hr/m²

$$\text{Over flow rate} = Q/BL$$

$$Q = \text{Capacity of the tank/Detention period}$$

$$BL = \text{Plan area}$$

- Depth of tank = 3m to 6m
- Slope = 1 in 50.
- Free board = 0.5m.

(b) DESIGN OF THE FLOC CHAMBER:

- The depth of floc chamber may be kept about half that of in the settling chamber
- Detention period = 15 to 40 minutes
- Capacity of chamber
= (maximum daily consumption /24) x detention period)
- Plan area = capacity of chamber/depth of the chamber
- Use the same width of the settling tank to find out the length of flocculation chamber.

8. Design a coagulation-cum-sedimentation tank with continuous flow for a population of 1, 00,000 persons with daily per capita water allowances of 200 liters. Make suitable assumptions where needed.

Given data:

Population = 1, 00,000

Per capita demand = 200 l/day

Required data;

Design coagulation cum sedimentation tank.

Design:

(a) DESIGN OF SETTLING TANK:

Average daily consumption = population x per capita demand
= 1, 00,000 x 200
= 20,000,000 liters
= 20×10^6 Liters.

Maximum daily demand = 1.8 x Average daily consumption
= $1.8 \times 20 \times 10^6$
= 36×10^6 liters

Quantity of water treated (capacity of tank)
= ((Maximum daily demand/24) x detention period)

(Detention period = 2 to 4 hours)

= $(36 \times 10^6 / 24) \times 4$
= 6×10^6 liters

Capacity of the tank required = $(6 \times 10^6 / 1000)$

= 6000 m³ (1 m³ = 1000 L)

Plan area = capacity of the tank / depth of tank

(Depth of tank= 3m to 6m)

$$= 6000/4$$

$$= 1500\text{m}^2$$

We need a big length for construct, so design as two units

Plan area of one unit = $1500/2 = 750\text{m}^2$

Plan area of one unit

$$= \text{length of single unit (L) x breadth of single unit(B)}$$

Length of the single unit = plan area of one unit/breadth of one unit

(Assume the breadth= 15m)

$$= 750/15$$

$$= 50 \text{ m}$$

Hence, use a tank of 50m x 15m x 4.0m

Provide extra depth for sludge storage adding 0.5m = 4+0.5 =4.5m at starting end

Other end is sloping end so the depth is increasing due to slope.

Assume (Slope = 1 in 50)

Free board = 0.5m

$$= 4.5 + ((1/50) \times 50) + 0.5$$

$$= 4.5 + 1 + 0.5$$

$$= 6\text{m}$$

(b)DESIGN OF THE FLOC CHAMBER:

$$\begin{aligned}\text{Depth of floc chamber} &= \text{half of the depth of settling chamber} \\ &= 4.5/2 = 2.25\text{m}\end{aligned}$$

Capacity of chamber

$$= (\text{maximum daily consumption} / 24) \times \text{detention period}$$

- Detention period = 15 to 40 minutes

$$\begin{aligned}&= (36 \times 10^6 / 24) \times (20/60) && (1 \text{ hr} = 60 \text{ minutes}) \\ &= 500000 \text{ liter} \\ &= 500 \text{ m}^3 && (1\text{m}^3 = 1000\text{L})\end{aligned}$$

Since We need a big length for construct, so design as two units g sedimentation tank

As well as floc chamber also designed as two chamber

$$\text{Plan area of one chamber} = 500/2 = 250\text{m}^3$$

$$\begin{aligned}\text{Plan area} &= \text{capacity of the chamber} / \text{depth of chamber} \\ &= 250/2.5 && (\text{Depth of chamber}= 2.5\text{m}) \\ &= 100\text{m}^2\end{aligned}$$

Using the same width of settling tank (15m) to find out the length of the floc chamber

$$\begin{aligned}&= 100/15 \\ &= 6.67\text{m approximately } 6.7\text{m}\end{aligned}$$

TREATMENT OF WATER - CONTD

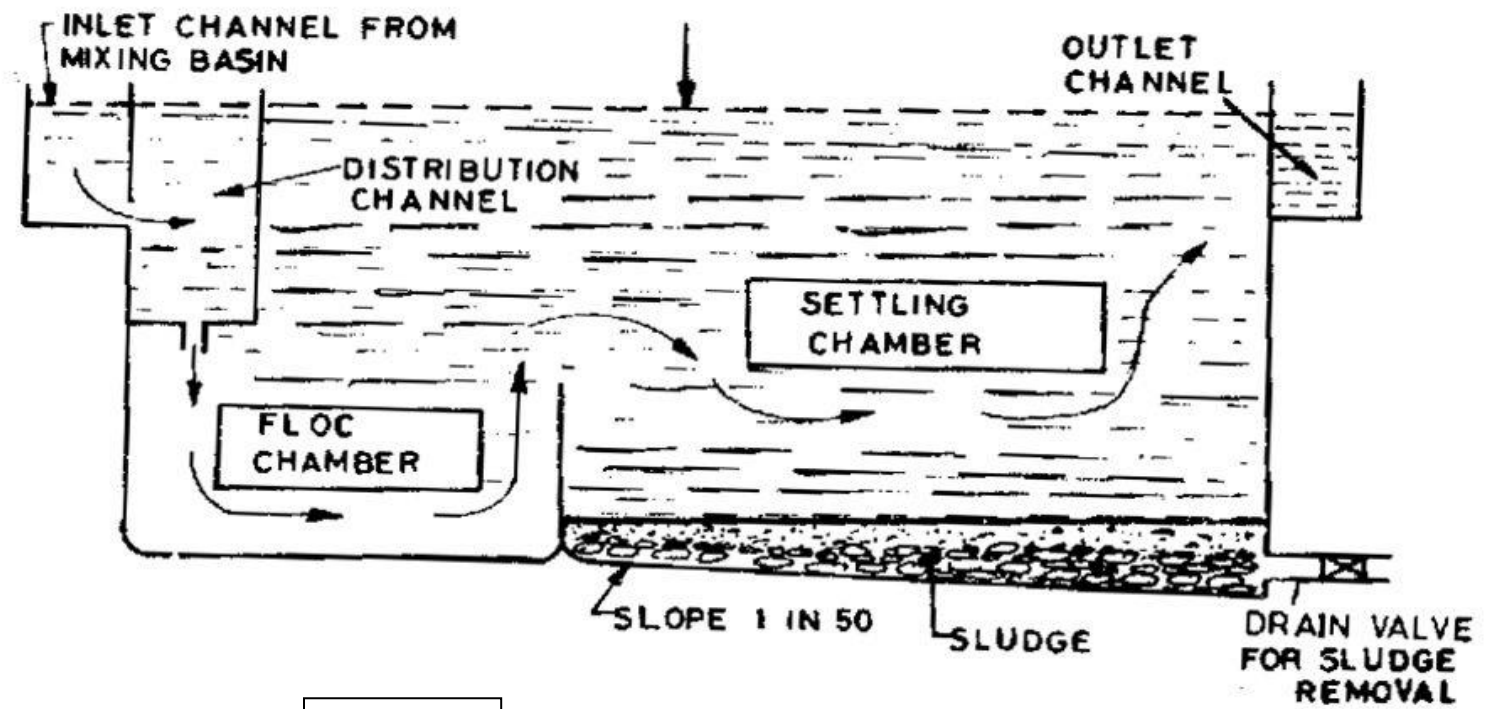


Figure 12

Coagulation-Sedimentation tank.

DESIGN OF CIRCULAR AND EQUIVALENT EGG SHAPED SEWER

- Average daily consumption = population x per capita demand
- Average daily consumption in m³/s
= (Average daily consumption/ (1000 x 24 x 60 x 60))
- Assume 80 percentage of water consumed appears as sewage
(Average quantity of discharge)
= (80/100) x Average daily consumption in m³/s
- Assume the peak sewage discharge (maximum rate of sewage produced):
q = 3 x average quantity of discharge in m³/s
- Storm water run-off:

$$Q_p = \frac{1}{36} \times K \times P_c \times A$$

Q_p = Peak steam run off

K = coefficient of runoff

P_c = critical design runoff

A = area of sewer

VALUES OF RUN-OFF COEFFICIENT (K)

SL. NO	TYPES OF LOCALITY	AVG APPROXIMATE POPULATION DENSITY IN PERSONS / HECTARE	K
1	Extreme suburban areas with 20 to 40 % parking and widely detached houses	75 - 125	0.35
2	Suburban areas with widely detached house	125 -150	0.45-0.55
3	Areas with 50% attached house	375-500	0.65
4	Areas closed by built up	500-625	0.75
5	Business areas	More than 625	0.85

- Combined maximum discharge = Q_p + maximum rate of sewage produced
- Assume that the sewer while carrying this combined peak discharge possesses 10 percentages extra capacity.

- The design discharge which the sewer running full
= combined discharge + ((10/100) x combined discharge)

- Chezy's formulae used to find out the velocity of sewage:

$$V = C \times (m \times i)^{1/2}$$

C = chezys constant

m =hydraulic radius or hydraulic mean depth

i = slope

- Chezys constant is find out by manning's formula:

$$C = (1/N) \times m^{1/6}$$

N = Manning's constant

m = wetted area / wetted perimeter = A/P

MANNINGS CO-EFFICIENT

S.No	Pipe material	Value of N at full depth	
		Good interior Surface condition	Fair interior surface condition
1	Plastic pipes	0.011	0.011
2	Cement concrete pipes	0.013	0.015
3	Cast iron pipes	0.012	0.013
4	Brick, unglazed sewers/drains	0.013	0.015
5	Salt glazed stone ware pipes	0.012	0.014
6	Asbestos cement	0.011	0.012

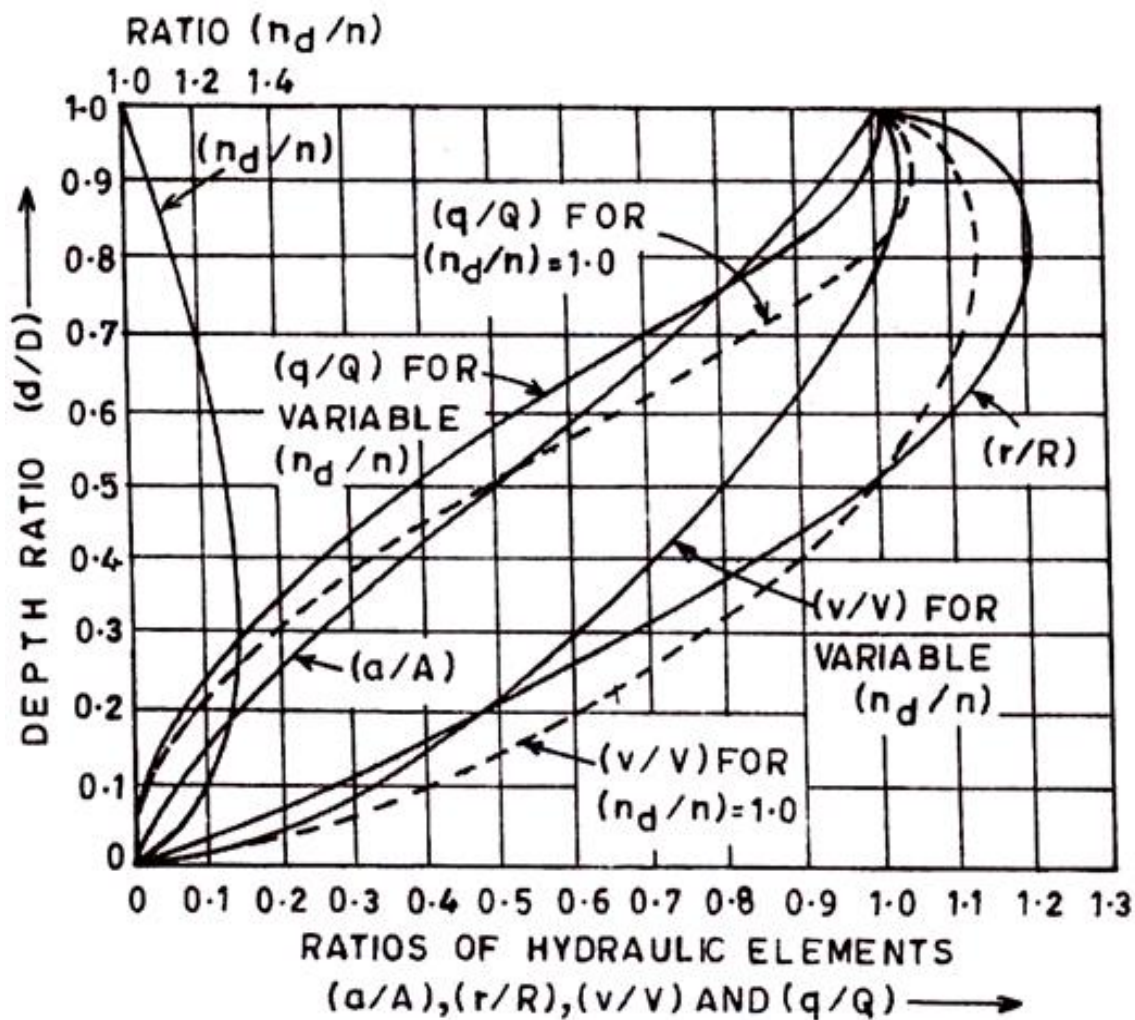
- Design discharge (Q) = area (A) x velocity (V)
 = A x C x (m x i)^{1/2}
 = A x (1/N) x m^{1/6} x (m x i)^{1/2}
 = A x (1/N) x m^{1/6} x m^{1/2} x i^{1/2}
 Q = (A X m^{2/3} x i^{1/2}) / N
 = 0.311 x D^{8/3} x i^{1/2} / N

(Area of circular sewer = 0.78539 D², Hydraulic radius = 0.25D)

- Diameter of sewer(D) from discharge= $(QN/0.311 \times i^{1/2})^{3/8}$
- Velocity of sewer $= Q/A$

Check for lone sewage discharge:

When maximum sewage passing (once a day) in non monsoon periods:



Standard chart for proportionate hydraulic sewer elements for circular sewers

We know q/Q ratio from this obtain v/V from graph shown above.

When maximum sewage passing (once a day) in non monsoon periods:

v/V = value from the graph

v = velocity in non monsoon period

V = velocity of sewage in monsoon period

v = value from the graph $\times V$

- $v >$ self cleaning velocity of sewer (0.45m/sec)

It is sufficient for non-silting.

Equivalent egg shaped velocity:

Top width of the egg shaped section (D') = $0.84D$

Bottom width of the egg shaped section (D'') = $D/2$

Height of the egg shaped section = $1.5 D'$

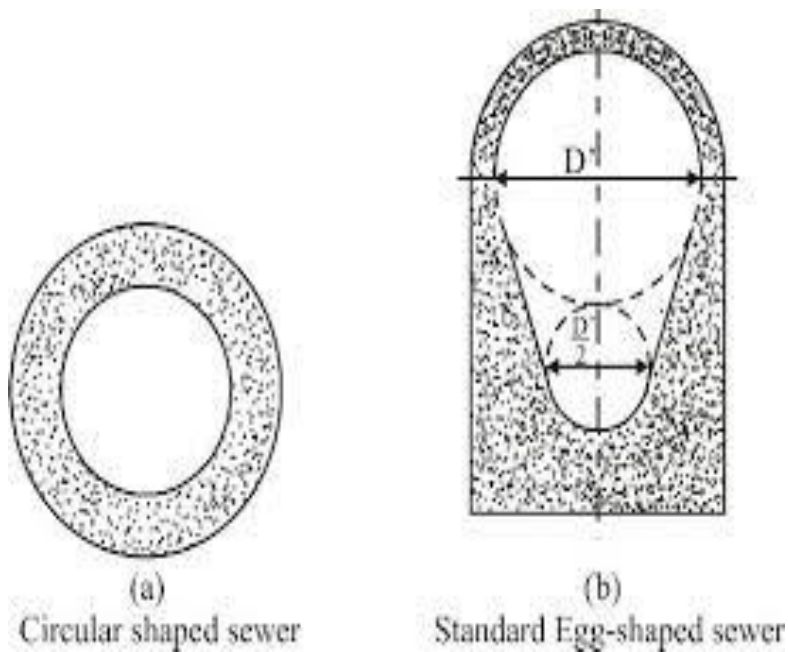


Figure 13

9. Design a circular sewer so as to cater to a residential colony in town, having the following data:

Area of the colony = 500000m²

Population =20000

Per capita demand =200 L/h/d

Critical design rainfall intensity =5.59cm/hr

General available ground slope = 1 in 600

Assume suitable data wherever necessary.

(ii) What will be the dimensions of an equivalent egg shaped sewer if adopted in this case.

DESIGN OF CIRCULAR SEWER:

Sewage discharge computations:

$$\begin{aligned}\text{Average daily consumption} &= \text{population} \times \text{per capita demand} \\ &= 200 \times 20000 = 4 \times 10^6 \text{ liter/ day}\end{aligned}$$

Average daily consumption in m³/s

$$\begin{aligned}&= (\text{Average daily consumption}/ (1000 \times 24 \times 60 \times 60)) \\ &= (4 \times 10^6 / (1000 \times 24 \times 60 \times 60)) \\ &= 0.0462 \text{m}^3\end{aligned}$$

Assume 80 percentage of water consumed appears as sewage

(Average quantity of discharge)

$$\begin{aligned}&= (80/100) \times \text{Average daily consumption in m}^3/\text{s} \\ &= 0.8 \times 0.0462 = 0.03696 \text{ m}^3\end{aligned}$$

Maximum rate of sewage produced (q) = 3 x average quantity of discharge
= 3 x 0.3696 = 0.1108 m³/s

Storm water run-off:

$$Q_p = \frac{1}{36} \times K \times P_c \times A$$

Q_p = Peak storm run off

K = coefficient of runoff (Areas with 50% attached house) = 0.65

P_c = critical design runoff = 5.59 cm/hr

A = area of colony = 500000 m² = 50 hectare
(1 hectare = 10000 m²)

$$Q_p = \frac{1}{36} \times 0.65 \times 5.59 \times 50$$
$$= 5.046 \text{ m}^3/\text{sec}$$

Combined maximum discharge = 5.046 + 0.11088 = 5.156 m³/s

Now, assume that the sewer while carrying this combined peak discharge possesses 10% extra capacity, we have

The design discharge which the sewer should carry while flowing full =

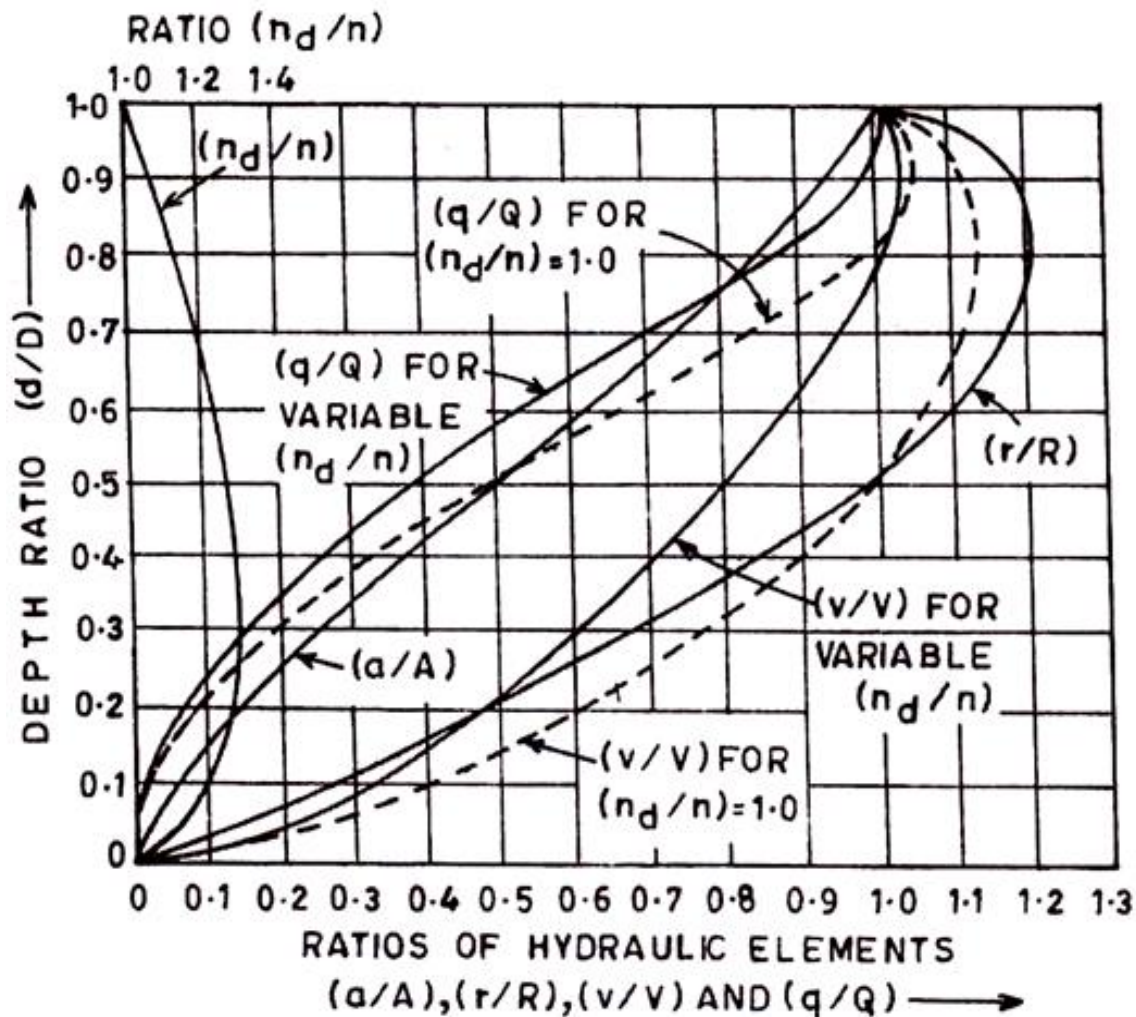
$$(10/100) \times 5.156 + 5.156 = 5.67 \text{ m}^3/\text{s}$$

Diameter of pipe (D) = (Q N / 0.3114 X i^{1/2})^{3/8}
= (5.67 x 0.013 / 0.3114 x ((1/600)^{0.5})^{3/8}
= 1.93 m approximately 2.0m

Velocity of sewage in pipe (V) = Q/A = 5.67/0.78539 X 2.0²
= 1.804 m/s > 0.9 m/s

The designer should also ensure that a velocity of 0.9m/s is developed at least at the time of the maximum flow and preferably during the average flow periods also.

Check:



We know q/Q ratio from this obtain v/V from graph shown above.

$$q/Q = (0.1108/5.67) = 0.0195$$

$$v/V = 0.28$$

$$\begin{aligned}
 v &= 0.28 \times V \\
 &= 0.28 \times 1.80 \\
 &= 0.505 \text{ m/s} > 0.45 \text{ m/sec}
 \end{aligned}$$

Hence ok

v = velocity in non monsoon period

$v >$ self cleaning velocity of sewer (0.45m/sec)

EQUIVALENT EGG SHAPED SEWER:

$$\begin{aligned}
 \text{Top width of the egg shaped section (D')} &= 0.84D \\
 &= 0.84 \times 2 \\
 &= 1.68\text{m approximately } 1.7\text{m}
 \end{aligned}$$

$$\begin{aligned}
 \text{Bottom width of the egg shaped section (D'')} &= D/2 \\
 &= 1.7/2 = 0.85\text{m}
 \end{aligned}$$

$$\begin{aligned}
 \text{Height of the egg shaped section} &= 1.5 D' \\
 &= 1.5 \times 1.7 \\
 &= 2.55\text{m say } 2.6\text{m}
 \end{aligned}$$

Hence ok

1	Cubic Meter	=	35.3146	Cubic Feet
1	Cubic Inch	=	0.01639	Liters
1	Cubic Feet	=	28.32	Liters
1	Liter	=	0.21998	UK Gallons
1	Liter	=	0.26418	US Gallons

DESIGN OF RIVER INTAKE

- Average water demand = number of population x average daily demand
- Maximum water demand = 1.8 x average water demand
- Assuming the pumping to take place for T hours, the discharge to be pumped(Q) = (Maximum water demand/1000 x T x 60 x 60)

DESIGN OF INLET WELL:

1. Screening:

- This screen may be made of vertical iron bar screens. These screens may be made of vertical iron bars of 20mm diameter, placed vertically at 30mm to 50mm horizontal clear spacing and fitted to an angle iron frame, which may be fixed properly in the opening.
- Flow Velocity through the screens to a value of about 0.15m/sec to 0.20m/sec
- The total required area of openings can be provided one or more screened openings at one level. Such openings or ports are in fact usually provided at 2 to 3 levels in the well, thus providing one or more ports at each level, depending on the pumping rate.
- Area of opening required at each level = Q/Flow velocity through screen
- Number of opening required = Clear length of openings/spacing of bar
- Number of bars = Number of opening required – 1
- Length occupied by the bar = number of bars x diameter of bars
- Total length of screen = clear length + length occupied by the bar

2. Design of gravity pipe connecting intake well and jack well:

- Diameter of intake pipe = $(Q/\text{Velocity of water in intake pipe} \times 0.78539)^{1/2}$
- Assume Velocity of water in intake pipe = 1.2m/sec
- Velocity of pipe = $(Q/0.78539 \times D^2)$
- Velocity of pipe < 1.2m/sec
- Using manning's formula in chezy's formula for find out slope(i)
$$= ((QN / Am^{2/3}))^2.$$

10. Design a river intake with respect to (i) number and size of the openings in the intake well (ii) the size, shape and the height of the intake well and (iii) the gravity pipe for raw water connecting the intake well and jack well. The data supplied is:

Reduced level of river bed	= +150.000m
Reduced level of lowest water level	= +158.000m
Reduced level of normal water level	= +160.000m
Reduced level of high flood level	= +167.500m
Population to be served	= 1, 05,000
Average water demand	= 200lpd

Assume suitable data wherever necessary.

Design :

Average water demand to be pumped = No.of.population x avg daily demand

$$= 105000 \times 200$$

$$= 21 \times 10^6 \text{ liters}$$

Maximum water demand = 1.8 x avg water demand to be pumped

$$= 1.8 \times 21 \times 10^6$$

$$= 37.8 \times 10^6$$

Assuming the pumping to take place for 16 hours, the discharge to be pumped

$$= \text{maximum water demand} / (1000 \times 16 \times 60 \times 60)$$

$$= (37.8 \times 10^6 / 1000 \times 16 \times 60 \times 60)$$

$$= 0.656 \text{ say } 0.66 \text{ m}^3/\text{sec}$$

1. Design of inlet well:

Let us provide ablong shaped inlet well, with openings provided at 3 layers:

First layer:

First layer of openings shall be kept below RL of lowest water level +158.000m

Second layer:

Second layer of openings shall be kept below RL of normal water level
+160.000m

Third layer:

Third layer of opening shall be kept below RL of high flood level +167.500m

These shall be fitted with bar screens made of 20mm (0.02m)dia steel bars of say 50mm (0.05m)clear opening. Let the velocity through the bar screens be limited to 0.15m/sec

The area of openings required at each level:

$$\begin{aligned}Q &= A \times V \\A &= Q/V \\&= 0.66/0.15 \\&= 4.4\text{m}^2\end{aligned}$$

Let us provide 1.5m height of screen openings, then the clear length of the openings required 2.95m

$$\begin{aligned}\text{Number of opening required} &= \text{Clear length of openings/spacing of bar} \\&= 2.95/0.05 \\&= 59\end{aligned}$$

Number of bars = Number of opening required – 1
 =59-1
 =58

Length occupied by 20mm diameter bars =58 x 0.02 = 1.16m

Total length of the screen =2.95 +1.16
 =4.11m says 4.2m

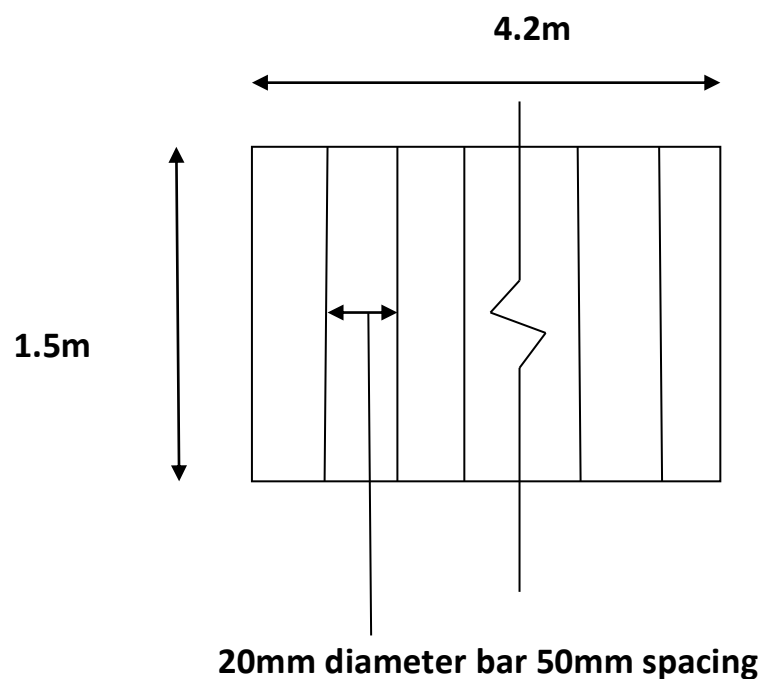


Figure 14

Bar screen

Let us provide 2 parts at each level

The size of each port will then be 1.5m x 2.1 length. In all, there will be 6 screened ports; 2 at each of three levels 2 screened ports will thus be provided with in the well steining between RL +158.000m to 156.500m each have 2.1m length. 2 other screened ports shall be shall similarly be provided between RL

+160.000m to +158.500m and 2 others between RL +167.500m to 157.000m as shown in figure.

These ports can be fitted in an ablong well, consisting of rectangular length of 4.6m (sufficient to fix two bar screens each of length 2.1m) and provided with circular ends. The well can have a width of 2.5m

This inlet well can be sunk into the river bed by say 3m below the river bed, as to provide space for accumulation of sand and silt. So let us keep the bottom of inlet well at RL +147.000m

Also let us provide a free board of 2m above the river high flood level to fix the bottom level of the roof of the well. Hence provide invert level of roof at RL +169.500m. The height of inlet well will then be 169.500m to 147.000m (22.500m)

2. Design of gravity pipe connecting intake well and jack well:

The intake pipe shall be designed to flow by gravity at maximum velocity of say 1.2m/s

$$Q = A \times V$$

$$D = (Q/V \times 0.78539)^{1/2}$$

$$= (0.66/1.2 \times 0.78539)^{1/2}$$

$$= 0.836\text{m says } 0.9\text{m diameter pipe}$$

Hence using 0.9m diameter R.C.C. pipe giving a velocity of 1.037m/sec

$$1.037\text{m/s} < 1.2\text{m/s}$$

Slope(i):

$$I = ((QN / Am^{2/3}))^2.$$

$$= (0.66 \times 0.017 / 0.78539 \times 0.9^2 \times 0.225^{2/3})^2$$

$$= 2.273 \times 10^{-3}$$

$$I = (1 / 1/ 0.002273)$$

$$= 1 \text{ in } 439.947 \text{ says } 1 \text{ in } 440$$

m = hydraulic mean depth = $D/4$ for circular radius

N = manning coefficient for R.C.C Pipes = 0.017

A = area of circular pipe = $0.78539 \times D^2$

Lay 0.9m diameter intake pipe at a gradient of 1 in 440

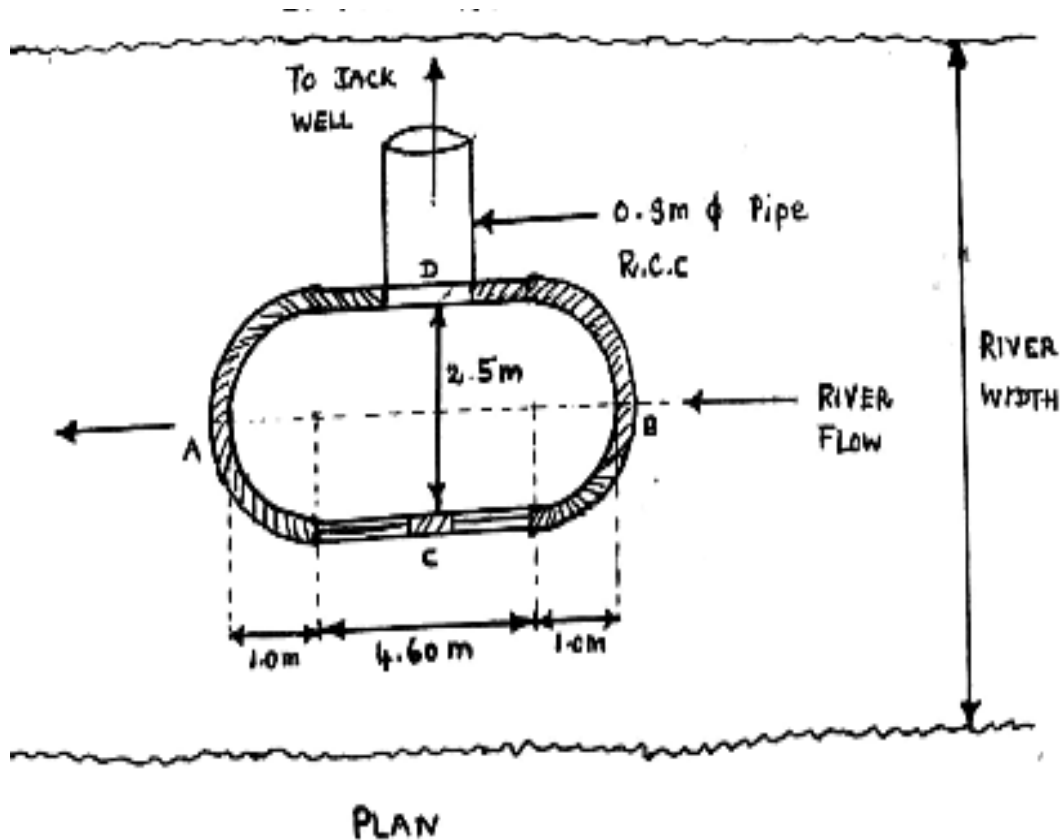


Figure 15

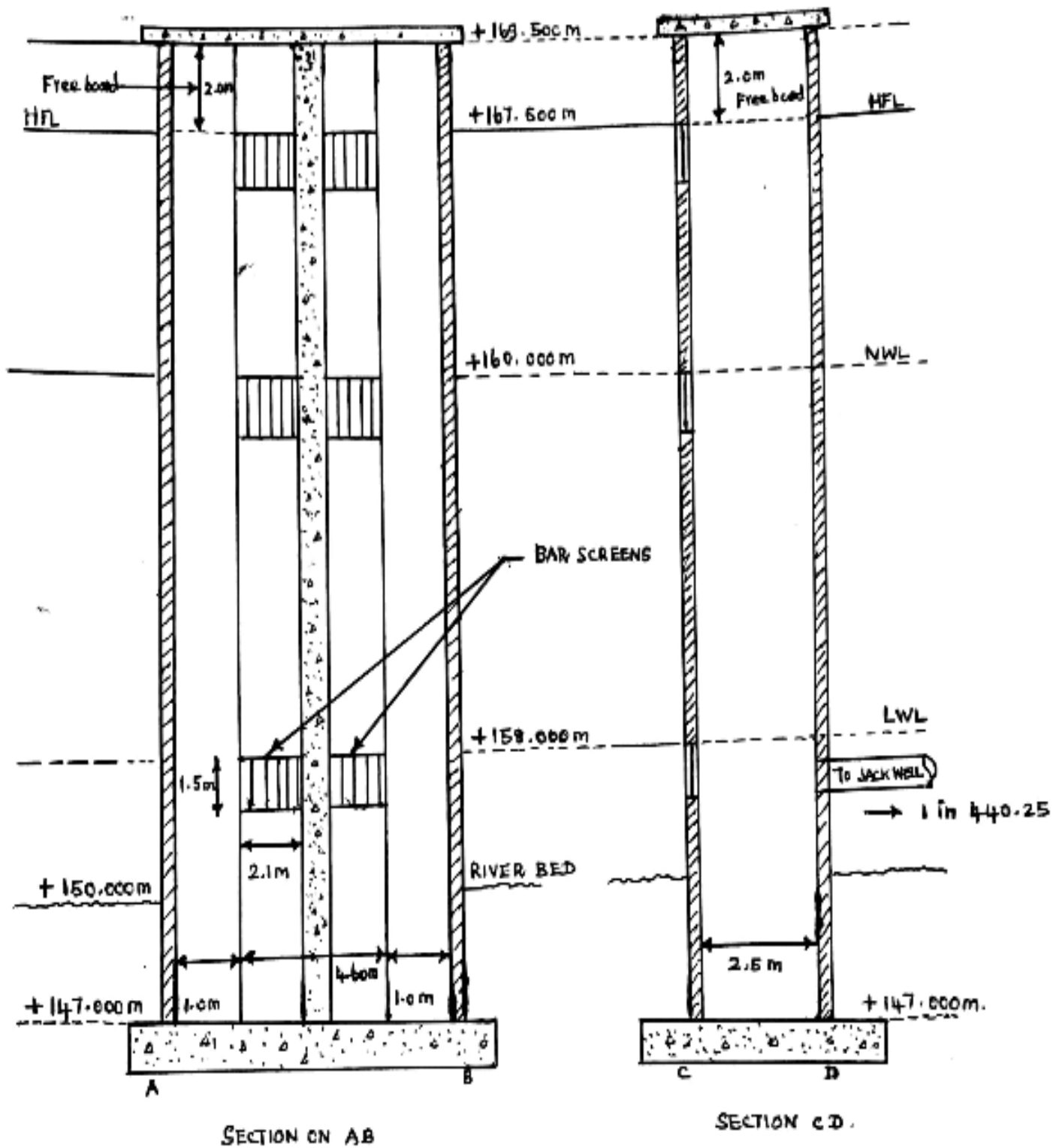


Figure 16

DESIGN OF CANAL INTAKE

- Discharge required by the city/town
= average consumption per person x total number of person in city/town
- Assuming the pumping to take place for T hours, the discharge to be pumped(Q) = (Maximum water demand/1000 x T x 60 x 60)

1. DESIGN OF COARSE SCREEN:

- This screen may be made of vertical iron bar screens. These screens may be made of vertical iron bars of 20mm diameter, placed vertically at 30mm to 50mm horizontal clear spacing and fitted to an angle iron frame, which may be fixed properly in the opening.
- Flow Velocity through the screens to a value of about 0.15m/sec to 0.20m/sec
- Area of opening required at each level = Q/Flow velocity through screen
- The entry of water in the intake well takes through a coarse screen, the top of which is generally provided at minimum water level in canal, and bottom is about 150mm (6'')above the canal bed to avoid entry of bed level
- Number of opening required = Clear length of openings/spacing of bar
- Number of bars = Number of opening required – 1
- Length occupied by the bar = number of bars x diameter of bars
- Total length of screen = clear length + length occupied by the bar

2. DESIGN OF BELL MOUTH ENTRY:

- The flow velocity of bell mouth inlet is limited to about 0.3m/sec or so
- Area of bell mouth entry(A_b) = Q / velocity through bell mouth
- Diameter of bell mouth (d_b) = ($A_b / 0.78539$)^{0.5}

3. DESIGN OF INTAKE CONDUIT:

- The flow velocity through the outlet of conduit is generally kept at about 1.5m/sec

- Area of conduit(A_c) = Q / velocity through conduit
- Diameter of conduit (d_c) = $(A_c / 0.78539)^{0.5}$
- The head loss in the intake of conduit up to treatment works, can be determined by using hazens william formula:

$$V = 0.85 C_H m^{0.63} i^{0.54}$$

$$i = H_L / L$$

V = Flow velocity

m = hydraulic radius = $d_c/4$

C_H = Coefficient of the pipe material

Table 3

Description of the Pipe	Values of C
Extremely smooth and straight	140
Cast iron:	
New	130
5 years old	120
10 years old	110
20 years old	90–100
30 years old	75–90
Concrete or cement lined	120–140
Welded steel, as for cast-iron pipe, 5 years older	
Riveted steel, as for cast-iron pipe, 10 years older	
Plastic	150
Asbestos cement	120–140

i = slope

L = Length

$$H_L = (2.395 V L^{0.54} / 0.85 C_H d^{0.63})^{1.852}$$

OR

$$H_L = (V L^{0.54} / 0.355 C_H d^{0.63})^{1.852}$$

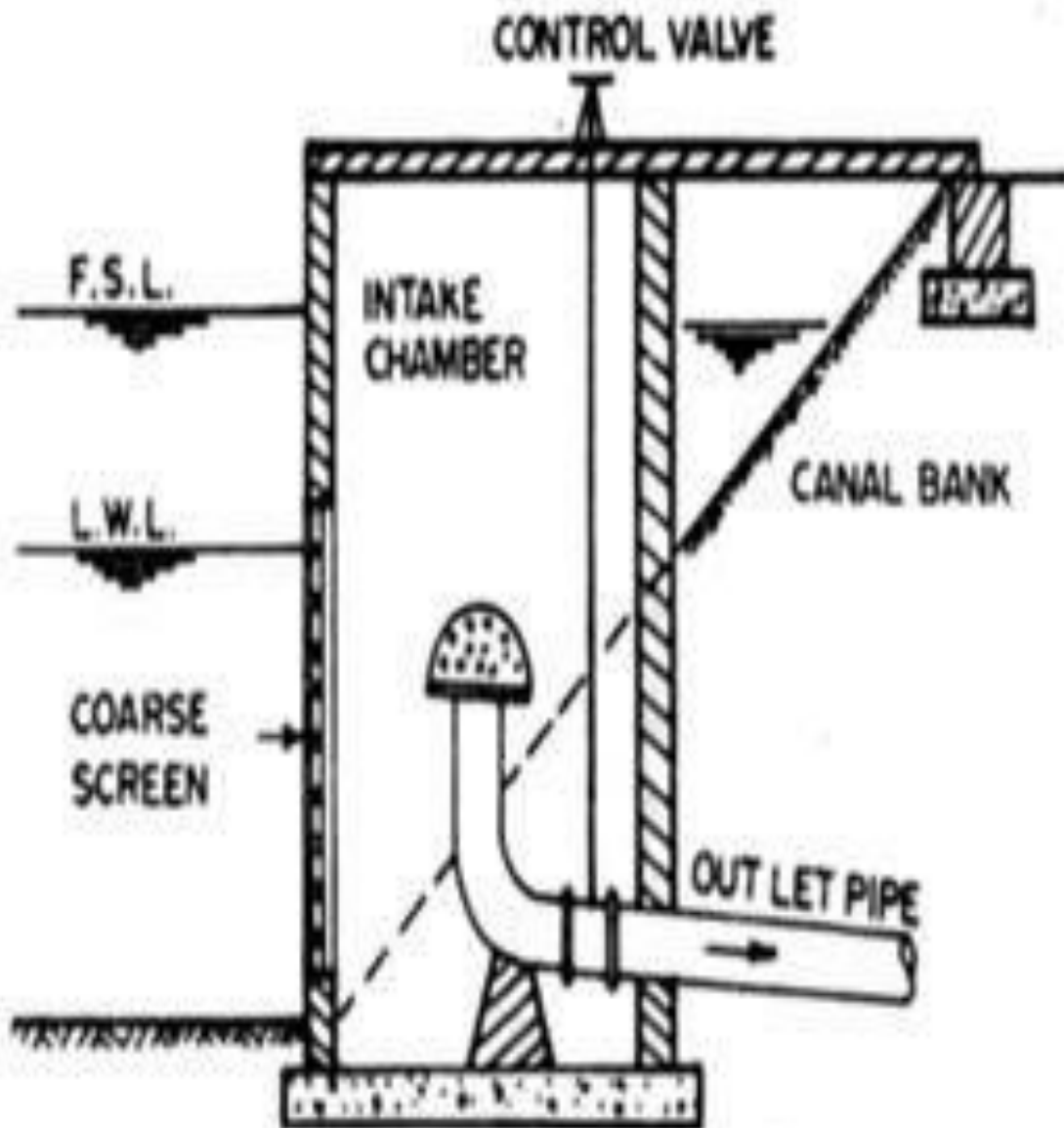


Figure 17

11. Design bell mouth canal intake for a city of 1,05,000 persons drawing water from a canal which runs only for 12 hours a day with a depth of 2.5m. Also calculate the head loss in the intake conduit if the treatment works are 750m away. Draw a neat sketch of anal intake. Given average consumption per persons is 200 liter per day. Assume the velocity through the screen and bell mouth to be less than 0.15m/se and 0.3m/sec respectively.

Design:

Discharge required by the city/town

$$\begin{aligned}
 &= \text{average consumption per person} \times \text{total number of person in city/town} \\
 &= 200 \times 1,05,000 \\
 &= 21 \times 10^6 \text{ LPD}
 \end{aligned}$$

Assuming the pumping to take place for T hours, the discharge to be pumped (Q) = (Maximum water demand/1000 x T x 60 x 60)

$$\begin{aligned}
 &= (21 \times 10^6 / 1000 \times 60 \times 60 \times 12) \\
 &= 0.486 \text{ m}^3/\text{sec}
 \end{aligned}$$

1. DESIGN OF COARSE SCREEN:

These shall be fitted with bar screens made of 20mm (0.02m)dia steel bars of say 50mm (0.05m)clear opening. Let the velocity through the bar screens be limited to 0.15m/sec

The area of openings required:

$$Q = A \times V$$

$$A = Q/V$$

$$= 0.486 / 0.15$$

$$= 3.24 \text{ m}^2$$

$$\text{Height of screen provided} = 2.5 \text{ m}$$

***The entry of water in the intake well takes through a coarse screen, the top of which is generally provided at minimum water level in canal, and bottom is about 150mm (6") above the canal bed to avoid entry of bed level**

****Assuming the minimum water level in canal to be 0.35m below Normal Water Level (NWL)**

$$= 2.5 - 0.15 - 0.35^{**}$$

$$H = 2.10\text{m says } 2\text{m}$$

Minimum length of screen provided (L) = Area of the screen/Height of the screen

$$= 3.24/2$$

$$= 1.62\text{m}$$

Now assuming the clear opening width between vertical bars to 30mm each

We have

$$\text{Number of opening required} = \text{Clear length of openings/spacing of bar}$$

$$= 1.62/0.03$$

$$= 54$$

$$\text{Number of bars} = \text{Number of opening required} - 1$$

$$= 54 - 1$$

$$= 53$$

$$\text{Length occupied by 20mm diameter bars} = 53 \times 0.02 = 1.06\text{m}$$

$$\text{Total length of the screen} = 1.62 + 1.06$$

$$= 2.68\text{m say } 2.7\text{m}$$

Hence provide 2.7m length of coarse screen of height 2m in the rectangular intake well as shown in figure.

2.DESIGN OF BELL MOUTH ENTRY:

The flow velocity of bell mouth inlet is limited to about 0.3m/sec or so

$$\begin{aligned}\text{Area of bell mouth entry (A}_b\text{)} &= Q / \text{velocity through bell mouth} \\ &= 0.486 / 0.3 \\ &= 1.62 \text{m}^2\end{aligned}$$

$$\begin{aligned}\text{Diameter of bell mouth (d}_b\text{)} &= (\text{A}_b / 0.78539)^{0.5} \\ &= (1.62 / 0.78539)^{0.5} \\ &= 1.436 \text{m says } 1.5 \text{m}\end{aligned}$$

Hence , use 1.5m diameter bell mouth provided with perforations for fine screen

3.DESIGN OF INTAKE CONDUIT:

The flow velocity through the outlet of conduit is generally kept at about 1.5m/sec

$$\begin{aligned}\text{Area of conduit (A}_c\text{)} &= Q / \text{velocity through conduit} \\ &= 0.486 / 1.5 \\ &= 0.324 \text{ m}^2\end{aligned}$$

$$\begin{aligned}\text{Diameter of conduit (d}_c\text{)} &= (\text{A}_c / 0.78539)^{0.5} \\ &= (0.324 / 0.78539)^{0.5} \\ &= 0.642 \text{m says } 0.65 \text{m}\end{aligned}$$

We may hence use 650mm diameter cast iron conduit

Flow velocity through this 650mm diameter conduit = Q / A_c

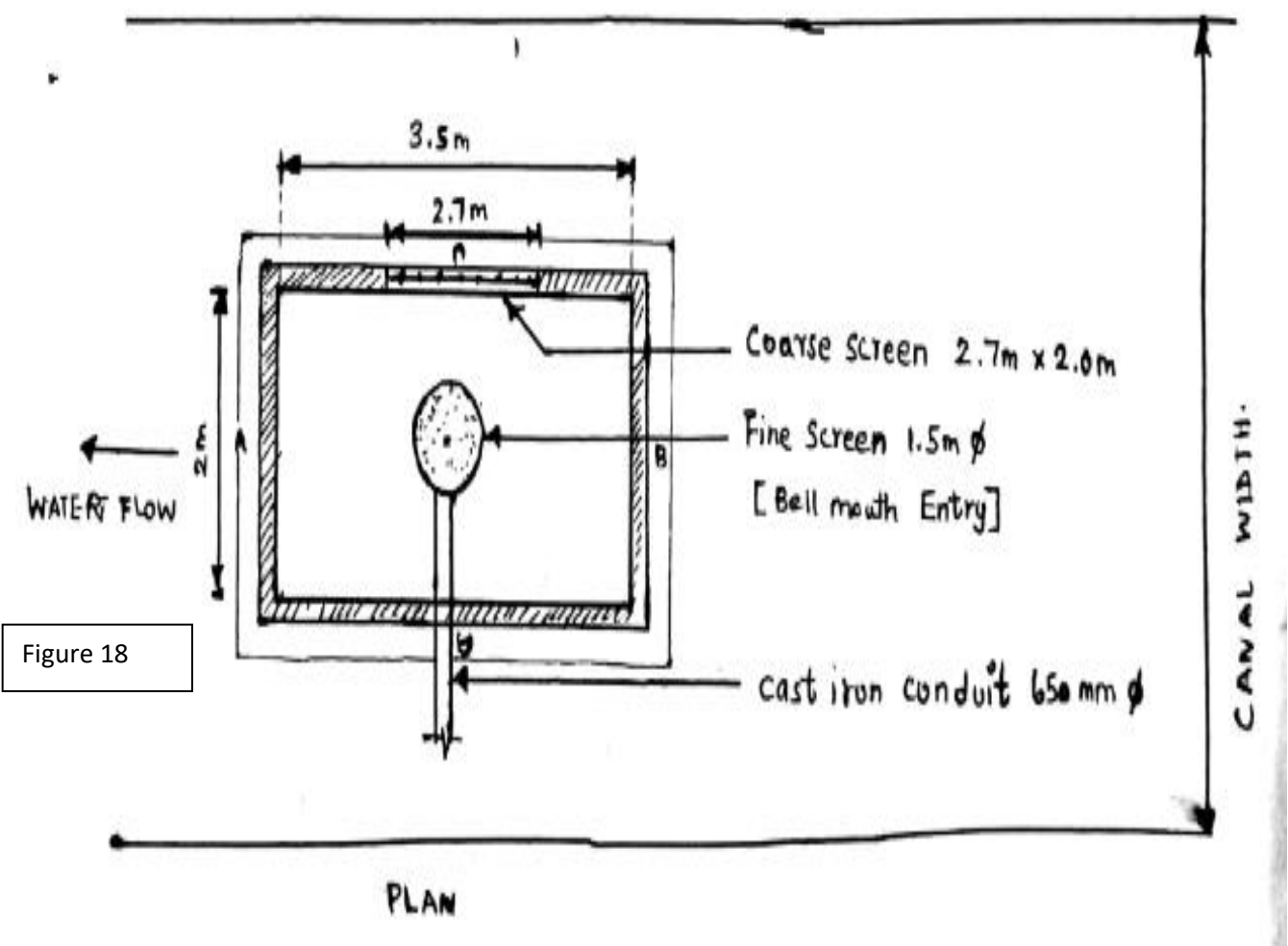
$$\begin{aligned}&= 0.486 / 0.78539 \times 0.65^2 \\ &= 1.465 \text{m/sec}\end{aligned}$$

The head loss in the intake of conduit up to treatment works can be determined by using hazens william formula:

$$\begin{aligned}
 H_L &= (V L^{0.54} / 0.355 C_H d^{0.63})^{1.852} \\
 &= (1.465 \times 750^{0.54} / 0.355 \times 130 \times 0.65^{0.63})^{1.852} \\
 &= 2.082\text{m}
 \end{aligned}$$

$$\begin{aligned}
 \text{Slope} &= H_L / L \\
 &= 2.082 / 750 \\
 &= 2.777 \times 10^{-3}
 \end{aligned}$$

Slope 1 in 360.08 says 1 in 360



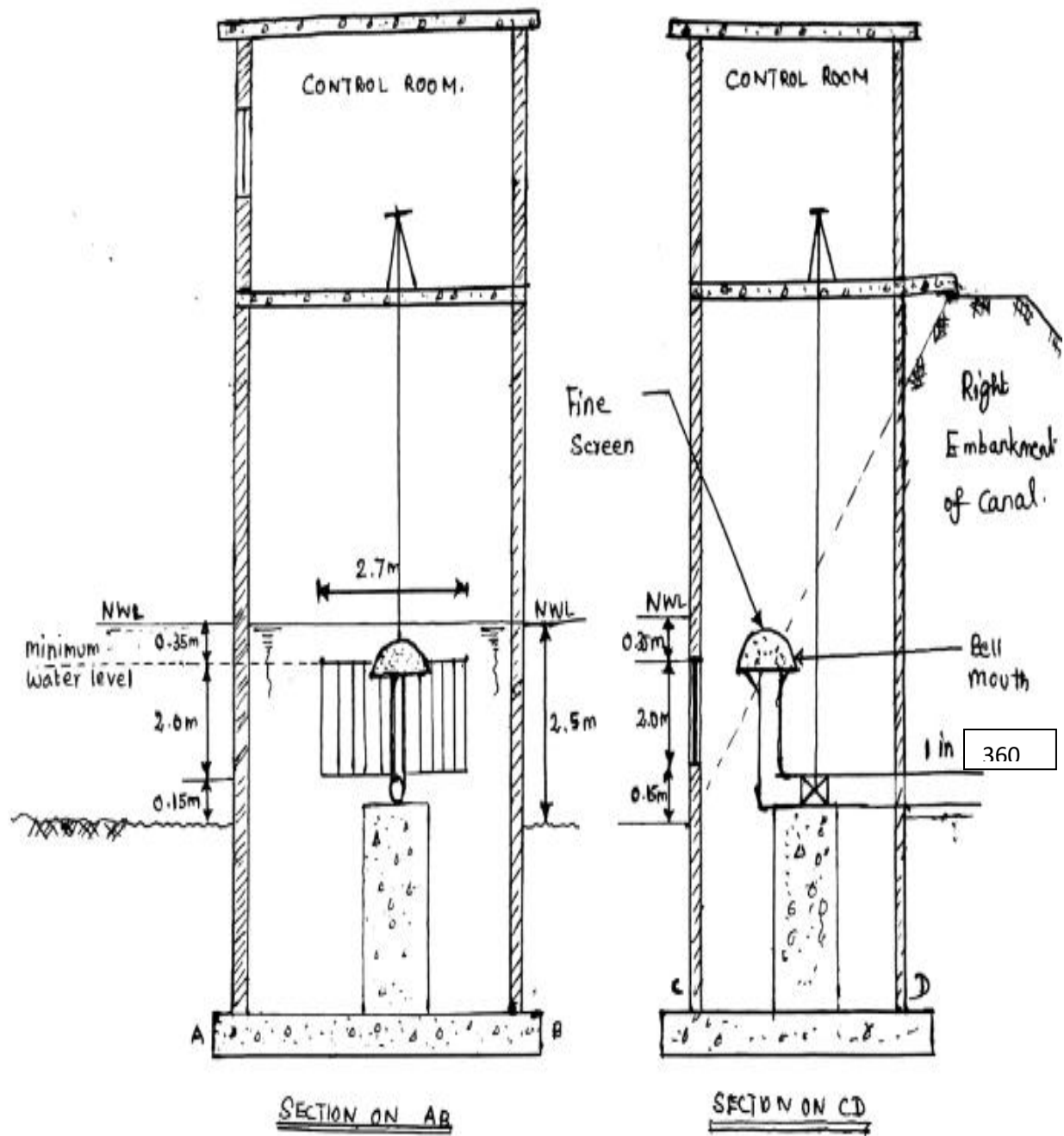


Figure 19

DESIGN OF TRICKLING FILTER

- Total BOD present in sewage to be treated per day (avg flow)=
Sewage capacity x BOD of sewage
- Organic loading = 900 kg/hectare –m / day to 2200 kg/hectare-m/day
- Volume of filter media
= Total BOD present in sewage to be treated per day / organic loading
- Effective depth = 2m to 3m
- Area = volume / depth
- Maximum Diameter of trickling filter = 60m
- Number of units required = area / area of single unit
- Check for hydraulic loading:
Hydraulic loading = 22 to 44 million liter / hectare / day
Surface area required
= total sewage to be treated per day/ hydraulic loading per day
Area > surface area required

1. DESIGN OF ROTARY DISTRIBUTOR:

- peak flow = 2.25 x average flow
- flow through each unit at peak flow = peak flow/2
- assume velocity of peak flow is 2m/sec through the central column of the distributor
- diameter of the central column (D_{cc}) =
 $(\text{flow through each unit of flow}/0.78539)^{1/2}$

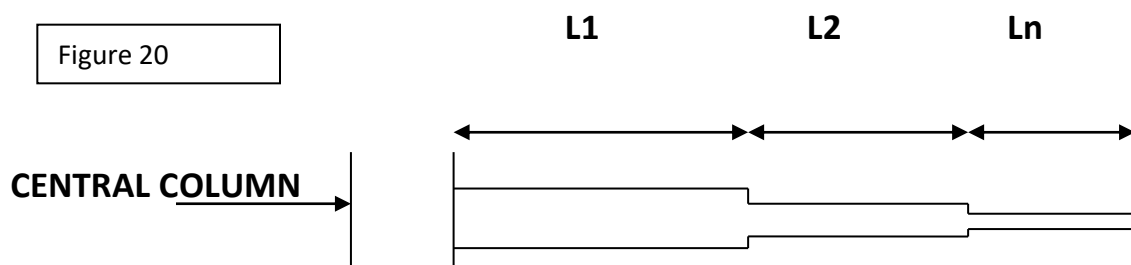
Check for flow velocity through the central column at average flow:

- discharge through each unit at average flow =
average flow/2
- velocity at average flow =
Discharge through each unit at average flow /area of central column
- velocity at average flow > 1.000m/sec

- **velocity of peak flow = flow through each unit at peak flow/area of central column**

2. DESIGN OF ARMS:

- **Assume the number of arms**
- **Discharge per arm = flow through each unit at peak flow/number of arms**
- **Arm length = (diameter of filter – 2)/2**
- **We can use each arm of length with its size reducing from near the central column towards the end**
- **Divided in to different length of sections Shown in figure**



Flow in the arm has to be adjusted in the proportion of the filter area covered by these lengths of arm. Therefore the area covered by the different length of arm are calculated by A_1, A_2 to A_n

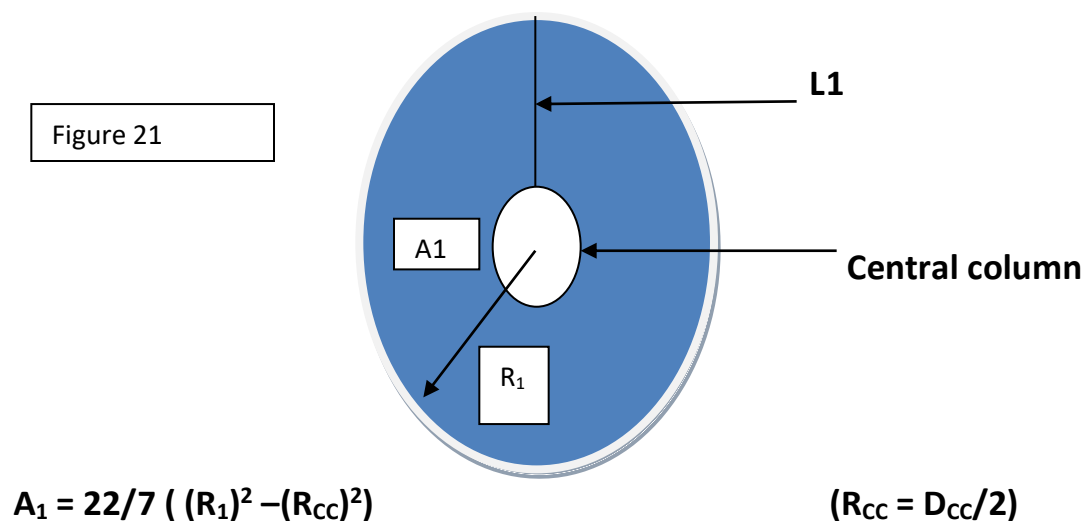
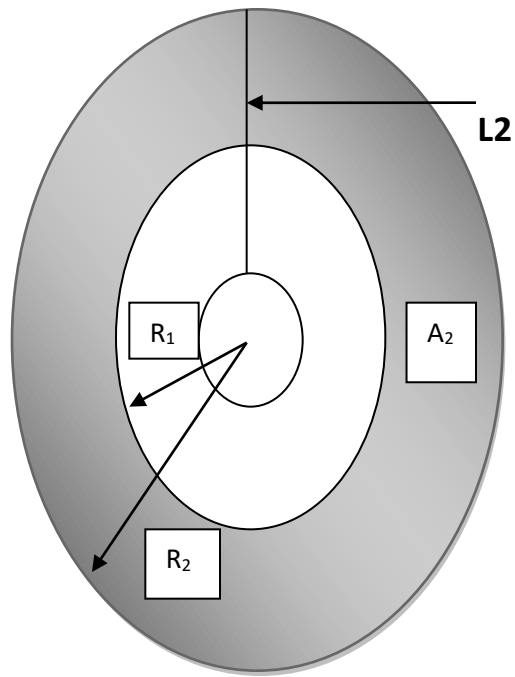
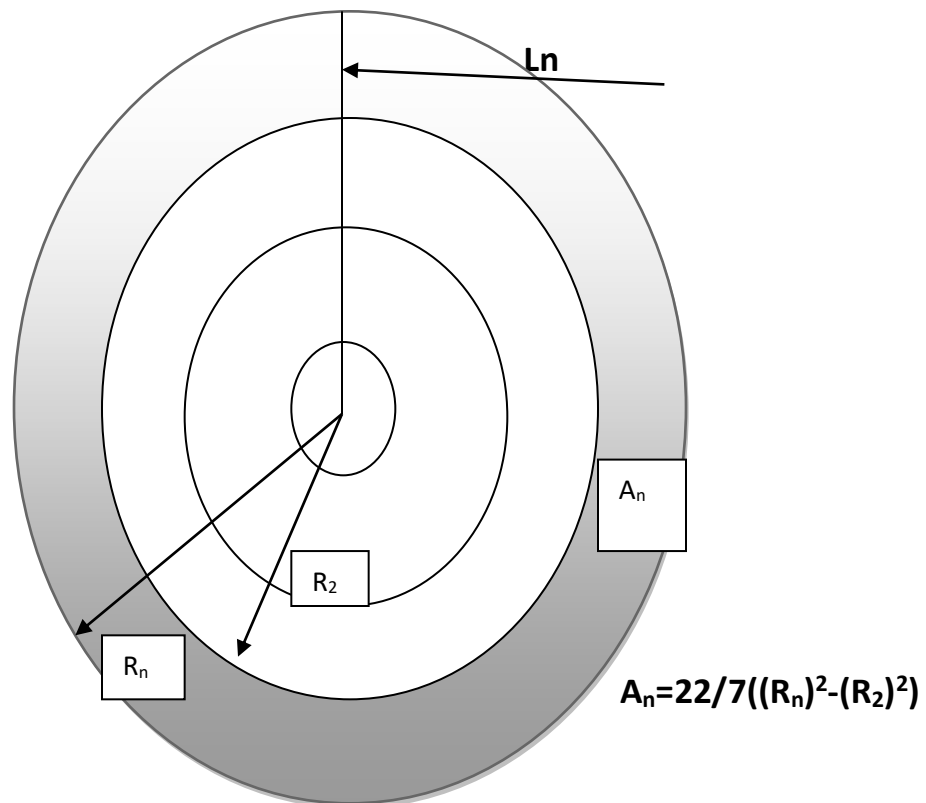


Figure 22



$$A_1 = 22/7 ((R_2)^2 - (R_1)^2)$$

Figure 23



$$A_n = 22/7 ((R_n)^2 - (R_2)^2)$$

- **Proportionate areas served by each sections of arms:**

$$((A_1/A) \times 100) + (A_2/A) \times 100 + (A_n/A) \times 100 = 100.00. \%$$

3. Design of orifice:

- Assume 10mm diameter orifice, coefficient of discharge(C_d)=0.65
- Assume water head causing flow as (h) 1.5m
- Discharge from orifice (Q) = $C_d \times a \times (19.62 h)^{1/2}$
- Area of orifice (a) = $0.78539 \times d^2$
- Total number of orifice through each arm

$$= \text{Discharge per arm} / \text{discharge through the orifice}$$
- Number of orifice through the L_1 section

$$= ((A_1/A) \times \text{Total number of orifice through each arm})$$
- Number of orifice through the L_2 section

$$= ((A_2/A) \times \text{Total number of orifice through each arm})$$
- Number of orifice through the L_n section

$$= ((A_n/A) \times \text{Total number of orifice through each arm})$$
- Spacing of first section (L_1) = $L_1 / \text{Number of orifice through the } L_1 \text{ section}$
- Spacing of second section (L_2)

$$= L_2 / \text{Number of orifice through the } L_1 \text{ section}$$
- Spacing of last section (L_n) = $L_n / \text{Number of orifice through the } L_1 \text{ section}$

4. Design of under drainage system:

- Design the under drainage system with a central rectangular channel, fed by radial laterals discharging into the channel. The under drain block lengths, containing semi elliptical openings can be used as laterals.
- The size and slope of the rectangular effluent channel should be such as to allow , say a velocity of 900mm/sec
- Area of channel = flow through each unit at peak flow/ velocity
- Assume the width of the channel
- Depth = area of channel / width of the channel
- Slope (i) = $I = ((QN / Am^{2/3}))^2$. (slope = 1/200 to 1/40)

- The under drain block lengths can be placed in rows, discharging into the effluent channel. The under drains may be laid circumferentially at a slope of 1/40 if the main effluent channel is provided diagonally.
- Let us use 10mm diameter, semi circular under drain blocks. These laterals should be designed to run less than half, so as to ensure proper ventilation. Let us assume that laterals run at a depth of say 0.3D, Where D is the diameter of circle of which the lateral section in a semi circle.
- Find Proportionate discharge q/Q and proportionate area a/A in tabular column given below
- Discharge through a circular sewer of diameter D $(Q) = A C (m i)^{0.5}$

$$C = 1/ N \times m^{1/6}$$

- Find q from the table
- Discharge through the filter
- Discharge through the each laterals (q)
- Number of laterals required
= Discharge through the filter/ q
- Velocity through the laterals at peak flow = $q / a > 0.75$ m/sec (req)
- Velocity at average flow ($q_{avg \text{ flow}} \text{)} = q / 2.25$
- Find $q_{avg \text{ flow}} / Q$
- For $q_{avg \text{ flow}} / Q$ of value , find d/D and a/A from table
- $V_{avg} = q_{avg \text{ flow}} / a_{avg} > 0.5$ m /sec

Proportionate values of hydraulic elements for circular sewers when flowing partially full (without being corrected for variations of roughness with depth)

Proportionate depth d/D	a/A	q/Q	Proportionate depth d/D	a/A	q/Q
0.1	0.052	0.021	0.6	0.626	0.671
0.2	0.143	0.088	0.7	0.748	0.838
0.3	0.252	0.196	0.8	0.858	0.988
0.4	0.373	0.337	0.9	0.949	1.066
0.5	0.500	0.500	1.0	1.000	1.000

Table 4

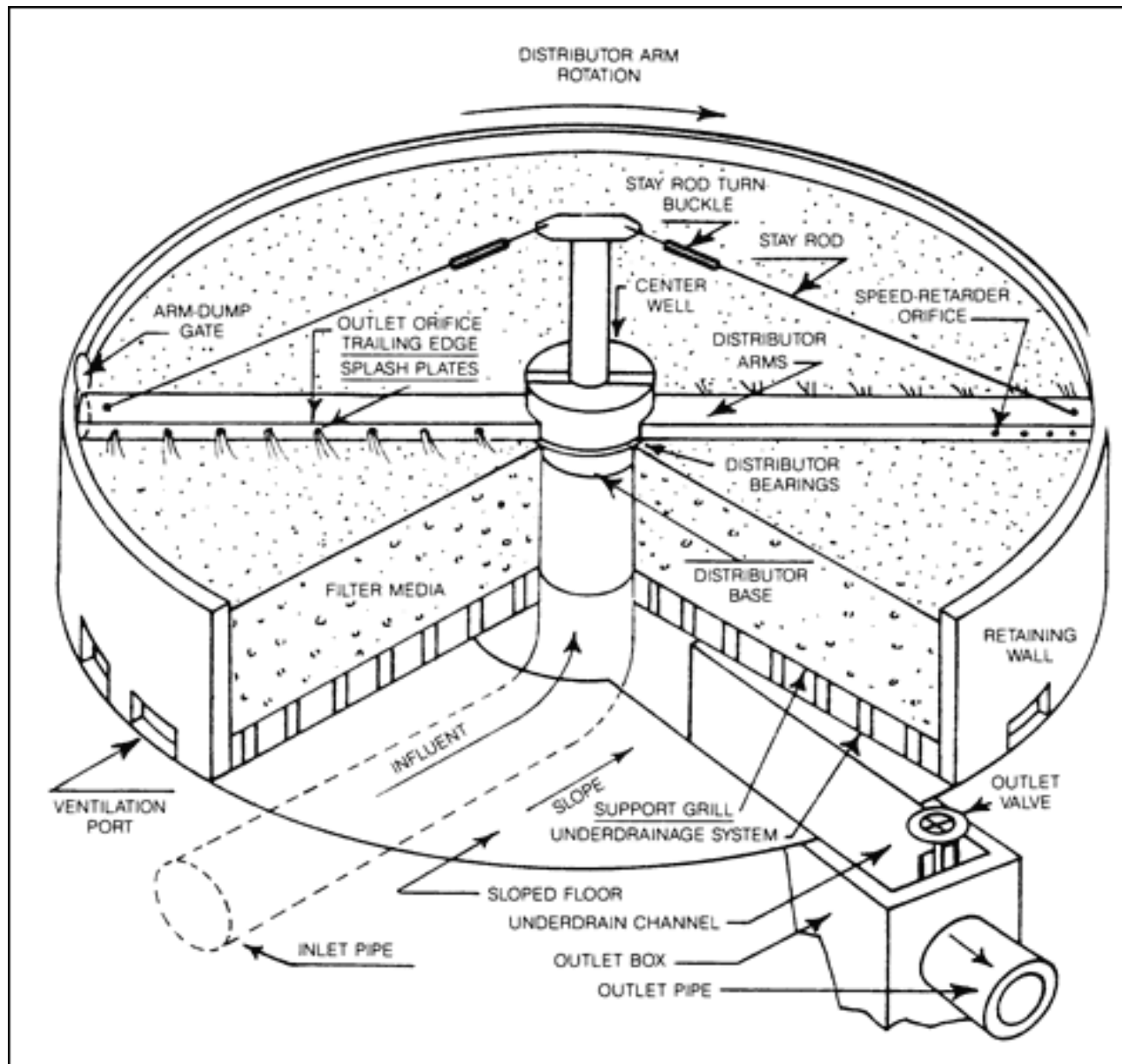


Figure 24

12. Design suitable dimensions of circular trickling filter units for treating 4500000 liter of sewage per Day. The B.O.D of sewage is 175 mg/l and also design suitable dimensions for its rotary distribution system, as well as the under drainage system.

Design:

Total BOD present in sewage to be treated per day (avg flow) =

$$\begin{aligned} & \text{Sewage capacity} \times \text{BOD of sewage} \\ &= 4.5 \times 10^6 \text{ L} \times 175 \text{ mg / L} \\ &= 4.5 \times 10^6 \text{ L} \times (175 \text{ Kg} / (1000 \times 1000 / \text{L})) \\ &= 4.5 \times 10^6 \text{ L} \times (1.75 \times 10^{-4} / \text{L}) \\ &= 787.5 \text{ Kg} \end{aligned}$$

(1 kg = 1000gm and 1 g = 1000mg)

Organic loading = 900 kg/hectare –m / day to 2200 kg/hectare-m/day

Assume Organic loading = 1000 kg/hectare-m

Volume of filter media

$$\begin{aligned} &= \text{Total BOD present in sewage to be treated per day} / \text{organic loading} \\ &= (787.5 / 1000) \text{ hectare – meter} \\ &= 0.7875 \times 10000 \text{ m}^2 \times \text{m} \\ &= 7875 \text{ m}^3 \qquad \qquad \qquad (1 \text{ hectare} = 10000\text{m}^2) \end{aligned}$$

Assume Effective depth of filter as say 2.5m

$$\begin{aligned} \text{Area} &= \text{volume} / \text{depth} \\ &= 7875 / 2.5 \\ &= 3150 \text{ m}^2 \end{aligned}$$

Using a circular trickling filter of diameter 45m

Number of units required = area / area of single unit

$$= 3150 / 0.78539 \times 45^2$$

$$= 1.98 \text{ says 2 numbers}$$

Check for hydraulic loading:

Hydraulic loading = 22 to 44 million liter / hectare / day

Surface area required

= total sewage to be treated per day/ hydraulic loading per day

$$= 4.5 \times 10^6 \text{ L/day} / 30 \times 10^6 \text{ L/hectare/day}$$

$$= 0.15 \text{ hectare}$$

$$= 0.15 \times 10000$$

$$= 1500\text{m}^2$$

$$3150 > 1500$$

Area > surface area required

Hence safe

Hence two units of each 45m diameter and 2.5 m effective depth (i.e. 3m overall depth) can be adopted

Extra third unit as standby may also be constructed.

1. DESIGN OF ROTARY DISTRIBUTOR:

$$\text{Peak flow} = 2.25 \times \text{average flow}$$

$$= 2.25 \times \text{average flow}$$

$$= 2.25 \times 4.5 \times 10^6 \text{ L/ Day}$$

$$= 10.125 \times 10^6 \text{ L/ Day}$$

$$= (10.125 \times 10^6 \text{ L/ Day} / 1000 \times 24 \times 60 \times 60)$$

$$= 0.117 \text{ m}^3/\text{sec}$$

Flow through each unit at peak flow = peak flow/2

$$= 0.117/2$$

$$= 0.0585 \text{ Says } 0.059 \text{ m}^3/\text{sec}$$

Assume velocity of peak flow is 2m/sec through the central column of the distributor

Diameter of the central column (D_{cc}) =

$$(\text{Flow through each unit of flow}/0.78539)^{1/2}$$

$$= (0.059/0.78539)^{1/2}$$

$$= 0.274\text{m say } 0.275\text{m}$$

Provide a central column .275 m in diameter, but check the velocity through the column at average flow, as it should not be less than 1m/sec or so

Check for flow velocity through the central column at average flow:

Discharge through each unit at average flow = average flow/2

$$= 4.5 \times 10^6 \text{ L/day} / 2$$

$$= 2.25 \times 10^6 \text{ L/day}$$

$$= 2.25 \times 10^6 / 24 \times 60 \times 60 \times 1000$$

$$= 0.0260 \text{ m}^3/\text{sec}$$

Velocity at average flow =

Discharge through each unit at average flow /area of central column

$$= 0.0260 / 0.78539 \times 0.275^2$$

$$= 0.437 \text{ m/sec}$$

Strictly the velocity at average flow becomes slightly less than the maximum permissible limits of 1m/sec. we should reduce the adopted diameter slightly

Let us use 0.18m diameter, then

Velocity at average flow =

$$\begin{aligned}\text{Discharge through each unit at average flow /area of central column} \\ &= 0.0260 / 0.78539 \times 0.180^2 \\ &= 1.021 \text{ m/sec} \\ 1.021 &> 1.000\end{aligned}$$

Velocity at average flow > 1.000m/sec

Velocity of peak flow =

$$\begin{aligned}\text{Flow through each unit at peak flow/area of central column} \\ &= 0.059 / 0.78539 \times 0.18^2 \\ &= 2.3185 \text{ m/sec}\end{aligned}$$

Hence we provide central column diameter = 0.18m (7.2")

2. DESIGN OF ARMS:

Now let use rotary section spray type distributor with four arms

$$\begin{aligned}\text{Discharge per arm} &= \text{flow through each unit at peak flow/number of arms} \\ &= 0.059/4 \\ &= 0.0148 \text{ m}^3/\text{sec}\end{aligned}$$

$$\begin{aligned}\text{Arm length} &= (\text{diameter of filter} - 2)/2 \\ &= (45-2)/2 \\ &= 21.5\text{m}\end{aligned}$$

We can use each arm of 21.5m length with its size reducing from near the central column towards the end

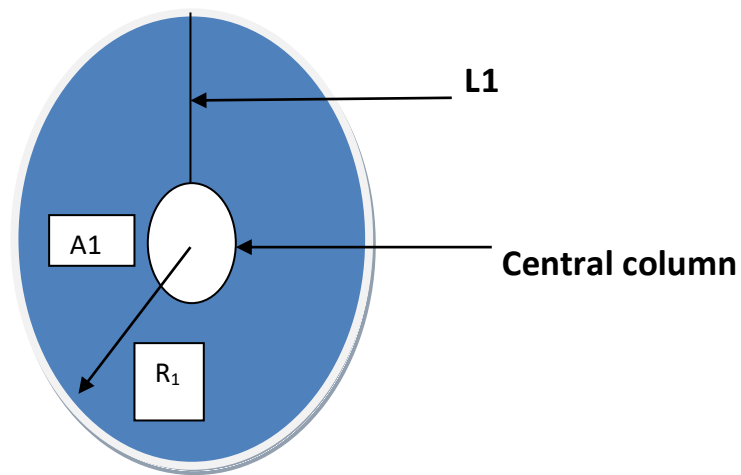
The first two sections, each of 7m length

The third section of 7.5m length

The flow in the arm has to be adjusted in the proportion of filter area covered by these lengths of arm. Therefore the area covered by the different length of arm calculated

A_1 , A_2 and A_3 – Circular filter areas covered by each length of arm

0.18m diameter in center to be used for central column



$$A_1 = \frac{22}{7} (R_1^2 - R_{cc}^2)$$

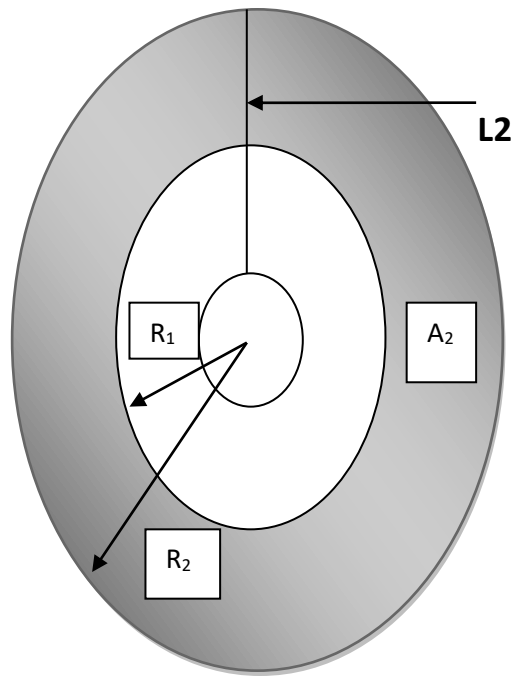
$$(R_{cc} = D_{cc}/2)$$

$$= \frac{22}{7} ((7.09)^2 - (0.09)^2)$$

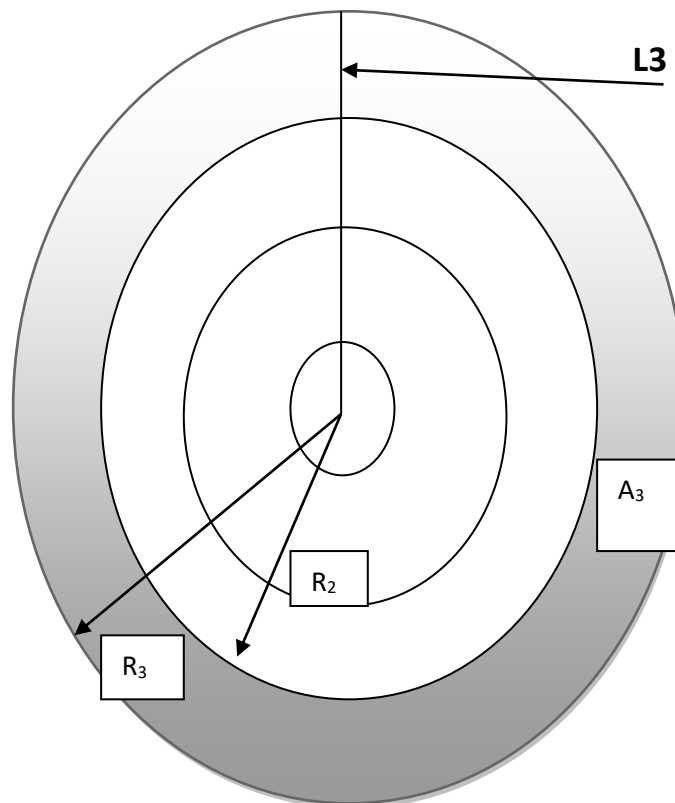
$$= 157.89\text{m}^2$$

$$(R_{cc} = D_{cc}/2)$$

$$(R_{cc} = 0.18/2 = 0.09 \text{ m})$$



$$A_1 = 22/7 (R_2)^2 - (R_1)^2 = 22/7 ((14.09)^2 - (7.09)^2) = 465.767\text{m}^2$$



$$A_3 = 22/7((R_3)^2 - (R_2)^2) = 22/7((22.5)^2 - (14.09)^2) = 966.726 \text{ m}^2$$

Proportionate areas served by each section of arms:

$$((A_1/A) \times 100) + (A_2/A) \times 100 + (A_n/A) \times 100 = 100.00 \%$$

$$(A_1/A) \times 100 = ((157.89/1590.389) \times 100) = 9.92\%$$

$$(A_2/A) \times 100 = ((465.767/1590.389) \times 100) = 29.29\%$$

$$(A_3/A) \times 100 = ((966.726/1590.389) \times 100) = 60.79\%$$

$$(9.92 + 29.29 + 60.79) = 100.00 \%$$

(i) Design the first section:

discharge = 0.0148 m³/sec

assuming the velocity through the arm as 1.2 m/sec

the area required = 0.0148/1.2 = 0.0123 m²

diameter required = (area / 0.78539)^{0.5}

$$= (0.0123/0.78539)^{0.5}$$

$$= 0.125 \text{ m (5")}$$

(ii) Design the first section:

discharge through the second section = (100 – 9.92) % x 0.0148

$$= 0.0133 \text{ m}^3/\text{sec}$$

the area required = 0.0133/1.2 = 0.0110 m²

diameter required = (area / 0.78539)^{0.5}

$$= (0.0110/0.78539)^{0.5}$$

$$= 0.118 \text{ m or 118 mm}$$

(iii) Design the first section:

discharge through the third section = (100 – 9.92 – 29.29) % x 0.0148

$$= 8.99 \times 10^{-3} \text{ m}^3/\text{sec}$$

the area required = 8.99 x 10⁻³ / 1.2 = 7.497 x 10⁻³ m²

diameter required = (area / 0.78539)^{0.5}

$$= (7.497 \times 10^{-3} / 0.78539)^{0.5}$$

$$= 0.0977 \text{ m or 97.7 mm}$$

if different size of pipes area difficult to join, then the entire arm length (avg =

((0.125 + 0.118 + 0.0977)/3 = 0.113 m say 0.120 m) may be kept of 120 mm diameter.

3. Design of orifice:

Assume 10mm diameter orifice, coefficient of discharge (C_d)=0.65

Assume water head causing flow as (h) 1.5m

$$\begin{aligned}\text{Discharge from orifice (Q)} &= C_d \times a \times (19.62 h)^{1/2} \\ &= 0.65 \times 0.78539 \times 0.01^2 \times (19.62 \times 1.5)^{1/2} \\ &= 2.769 \times 10^{-4} \text{ m}^3/\text{sec}\end{aligned}$$

Area of orifice (a) = $0.78539 \times d^2$

Total number of orifice through each arm

$$\begin{aligned}&= \text{Discharge per arm/discharge through the orifice} \\ &= 0.0148 / 2.769 \times 10^{-4} \\ &= 53.44 \text{ say } 54 \text{ numbers}\end{aligned}$$

Number of orifice through the L_1 section

$$\begin{aligned}&= (((A_1/A) \times 100)/100) \times \text{Total number of orifice through each arm} \\ &= ((9.92/100) \times 54) \\ &= 5.35 \text{ say } 5\end{aligned}$$

Number of orifice through the L_2 section

$$\begin{aligned}&= (((A_2/A) \times 100)/100) \times \text{Total number of orifice through each} \\ &= ((29.29/100) \times 54) \\ &= 15.81 \text{ say } 16\end{aligned}$$

Number of orifice through the L_3 section

$$\begin{aligned}&= (((A_3/A) \times 100)/100) \times \text{Total number of orifice through each} \\ &= ((60.79/100) \times 54) \\ &= 32.82 \text{ say } 33\end{aligned}$$

Spacing of first section (L_1) = $L_1 / \text{Number of orifice through the } L_1 \text{ section}$

$$\begin{aligned}&= 7/5 \\ &= 1.4 \text{ m c/c}\end{aligned}$$

$$\begin{aligned}\text{Spacing of second section (L}_2\text{)} &= L_2 / \text{Number of orifice through the L}_2\text{ section} \\ &= 7/16 \\ &= 0.437 \text{ say } 0.43 \text{ m c/c}\end{aligned}$$

$$\begin{aligned}\text{Spacing of last section (L}_3\text{)} &= L_3 / \text{Number of orifice through the L}_3\text{ section} \\ &= 7.5/33 \\ &= 0.227\text{m say } 0.225\text{m c/c}\end{aligned}$$

Design of under drainage system:

Total discharge through each filter unit at peak flow = 0.059m³/sec

Let us Design the under drainage system with a central rectangular channel, fed by radial laterals discharging into the channel. The under drain block lengths, containing semi elliptical openings can be used as laterals.

The size and slope of the rectangular effluent channel should be such as to allow, say a velocity of 900mm/sec (0.9m/sec)

$$\begin{aligned}\text{Area of channel} &= \text{flow through each unit at peak flow/ velocity} \\ &= 0.059/0.9 \\ &= 0.0656 \text{ say } 0.066\text{m}^2\end{aligned}$$

Assume the width of the channel = 0.25m wide

$$\begin{aligned}\text{Depth} &= \text{area of channel / width of the channel} \\ &= 0.066/0.25 = 0.33\text{m say } 0.35\text{m}\end{aligned}$$

$$\text{Slope (i) = I} = ((QN / Am^{2/3}))^2 \quad (\text{slope} = 1/200 \text{ to } 1/40)$$

$$\begin{aligned}m &= A / P \\ &= (0.25 \times 0.35) / (0.35 + 0.25 + 0.35) \\ &= 0.0657\end{aligned}$$

m = hydraulic radius

A = Wetted area

P = Wetted perimeter

$$I = ((0.059 \times 0.018 / (0.25 \times 0.35) \times 0.0657^{2/3}))^2$$

$$= 5.5566 \times 10^{-3}$$

Slope 1 in 180

Hence use a central effluent channel, 0.25m (10”) in width and 0.35m (14”) in depth below the bottom of the laterals

The channel may be laid at a slope of 1 in 180

The under drain block lengths can be placed in rows, discharging into the effluent channel. The under drains may be laid circumferentially at a slope of 1/40 if the main effluent channel is provided diagonally.

Let us use 10mm diameter, semi circular under drain blocks. These laterals should be designed to run less than half, so as to ensure proper ventilation.

Let us assume that laterals run at a depth of say 0.3D, Where D is the diameter of circle of which the lateral section in a semi circle.

Find Proportionate discharge q/Q and proportionate area a/A in tabular column given below

From the tabular column q/Q = 0.196 and a/A = 0.252

Discharge through a circular sewer of diameter D (Q) = A C (m i)^{0.5}

$$C = 1 / N \times m^{1/6}$$

$$= (1/0.013) \times (0.1/4)^{1/6}$$

$$= 41.59$$

$$(Q) = 0.78539 \times 0.1^2 \times 41.59 ((0.1/4) \times (1/40))^{0.5}$$

$$= 0.008125 \text{ m}^3/\text{sec}$$

Discharge through each lateral = 0.008125 m³/sec

Discharge through the filter = 0.059 m³/sec

Find q :

$$q/Q = 0.196$$

$$\begin{aligned} q &= 0.196 \times Q \\ &= 0.196 \times 0.008125 \\ &= 0.00159 \text{ m}^3/\text{sec} \end{aligned}$$

$$\begin{aligned} \text{Number of laterals required} &= \text{Discharge through the filter}/q \\ &= 0.059/0.00159 \\ &= 37.04 \text{ say } 38 \end{aligned}$$

So use 38 laterals in all, laid circumferentially in the circular filter tank of 45m diameter @ about 1.0m spacing to discharge in the central effluent channel below.

Velocity through the laterals at peak flow = $q / a > 0.75 \text{ m/sec}$ (req)

$$\begin{aligned} q/a &= 0.0020475 / 0.78539 \times 0.1^2 \times 0.252 \\ &= 1.0345 \text{ m/sec} > 0.75 \text{ m/sec} \\ (a &= A \times 0.252 = 0.78539 \times 0.1^2 \times 0.252, a/A = 0.252) \end{aligned}$$

Hence ok

$$\begin{aligned} \text{Velocity at average flow (} q_{\text{avg flow}} \text{)} &= q / 2.25 \\ &= 0.00159/2.25 \\ &= 0.0007067 \text{ m}^3/\text{sec} \end{aligned}$$

Find $q_{\text{avg flow}} / Q$:

$$\begin{aligned} &= q_{\text{avg flow}} / Q \\ &= 0.0007067/0.008125 \\ &= 0.0869 \end{aligned}$$

For $q_{\text{avg flow}} / Q$ of 0.087 , from the table $d/D = 0.2$ and $a/A = 0.143$

$$\begin{aligned} a_{\text{avg}} &= 0.143 A \\ &= 0.143 \times 0.78539 \times 0.1^2 \\ &= 0.001123 \text{ m}^2 \end{aligned}$$

$V_{\text{avg}} = q_{\text{avg flow}} / a_{\text{avg}} > 0.5 \text{ m/sec}$:

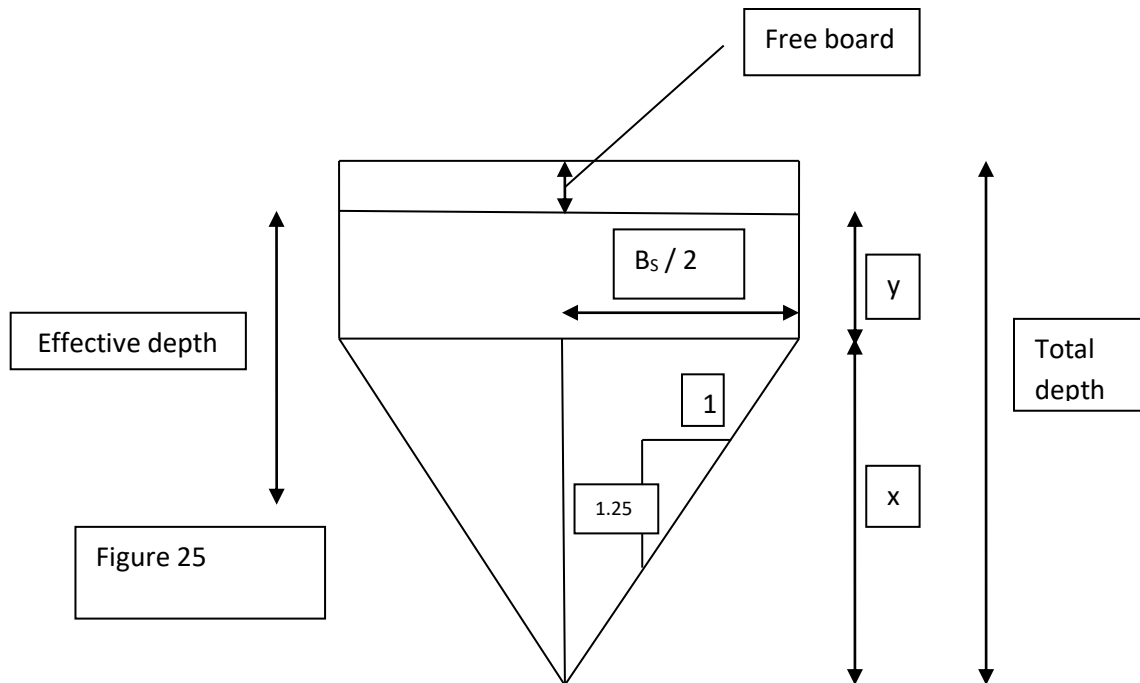
$$\begin{aligned} V_{\text{avg}} &= q_{\text{avg flow}} / a_{\text{avg}} > 0.5 \text{ m/sec} \\ &= 0.0007067/0.001123 \\ &= 0.629 \text{ m/sec} > 0.5 \text{ m/sec} \text{ HENCE O.K.} \end{aligned}$$

Hence , use 38 semi circular laterals of 100mm diameter (4'') each, discharging into the effluent channel and laid at a slope of 1 in 40.

DESIGN OF IMHOFF TANK

1. Design of sedimentation chamber:

- Sewage discharge per day
= population x rate of sewage liter per head per day
- Detention period = 2 to 4 hours
- Volume of sewage in detention period
= sewage discharge per day x (detention period / 24)
- Flow through velocity = should not be more than 0.3m/Minute
- Surface loading = should not be more than 30000 liters/m²/area/day
= should not be more than 45000 liters/ m²/area/day for effluent coming from activated sludge plant or recirculation adopted.
- Length of imhoff tank = not exceed 30m
- Length of the sedimentation chamber =
(Volume of sewage in detention period/ effective depth x width)
- Length = 3 x width of imhoff tank (L/B = 3 TO 5)
- Discharge passing through single unit = (1/number of units) x total discharge
- Total depth = 9m to 11m
- Depth of sedimentation chamber = 3 to 3.5m
- free board = 0.45m
- Check for velocity : velocity = length of tank /detention time < 0.3 m/ min
- Check for surface loading = $Q/BL < 30000 \text{ L/m}^2/\text{day}$
- Assume the effective depth = width of sedimentation tank / 2
- Bottom side slope 1 H :1.25 V
- From figure above (x) = 1.25 x(width of sedimentation chamber(B_s)/2)
- Y = effective depth - (x/2)
- Total depth of sedimentation tank = free board + y + x



Sedimentation chamber

- Check for velocity : velocity = length of tank /detention time < 0.3 m/ min
- Check for surface loading = $Q/BL < 30000 \text{ L/m}^2/\text{day}$
- Assume the effective depth = width of sedimentation tank / 2
- Bottom side slope 1 H :1.25 V
- From figure above $(x) = 1.25 \times (\text{width of sedimentation chamber}(B_s)/2)$
- $Y = \text{effective depth} - (x/2)$
- Total depth of sedimentation tank = free board + y + x

2. Design of gas vent and neutral zone :

- Provide a neutral zone of free board below this total length. the tank , in general, is of total length, but below this total depth, it shall be divided into a number of compartments

Length of each compartment = total length / number of compartment

- The area of gas vent has now to be provided on both sides of the sedimentation chamber. This width should be about 25 to 30% of the total width of the tank. The total width of the gas vent (that is both sides of sedimentation chamber), assuming 0.15m thickness of chamber walls

**(length of each compartment) +(length of each compartment x 30/100) –
B_s – (2 x thickness of chamber wall)**

3. Design of digestion chamber :

- Assuming the capacity of the digestion chamber at 40 liters/capita
- the capacity of the digestion chamber
= population x capacity of digestion chamber in liters/capita
- the capacity of each unit
= the capacity of the digestion chamber/ number of compartments
- now assume the depth of each hopper and side slopes 1:1 and bottom section as
**((length of each compartment) +(length of each compartment x 30/100) –
2 x depth of hopper)) x (length of each compartment - 2 x depth of hopper)**
- Length of single digester = (L_R) = Length of each compartment
- Overall width = (B_R) = ((length of each compartment) +(length of each compartment x 30/100))
- Capacity of each hopper = $h/3 (A_1 + A_2 + (A_1 A_2)^{1/2})$
- $A_1 = L_R \times B_R$
- $A_2 = L_H \times B_H$

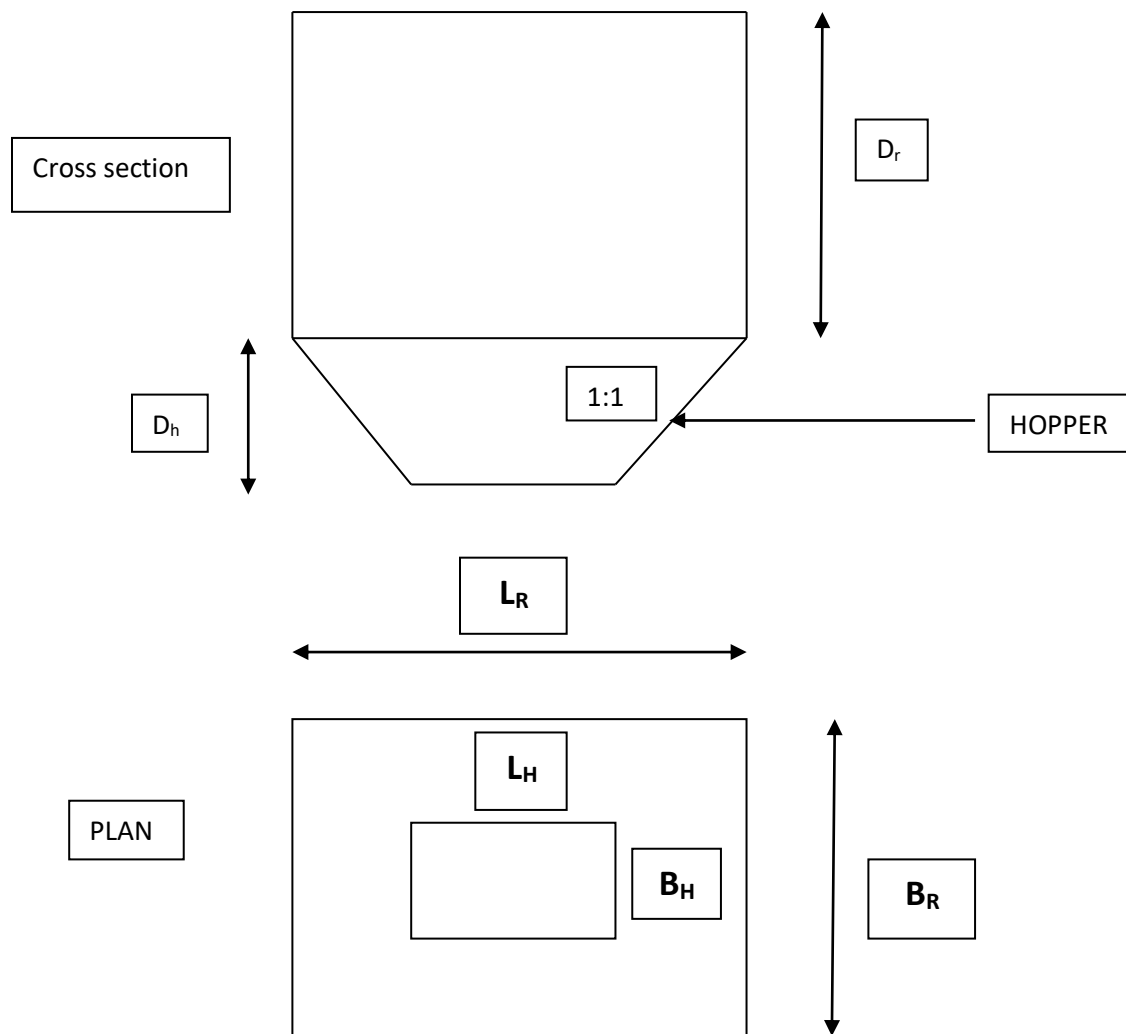


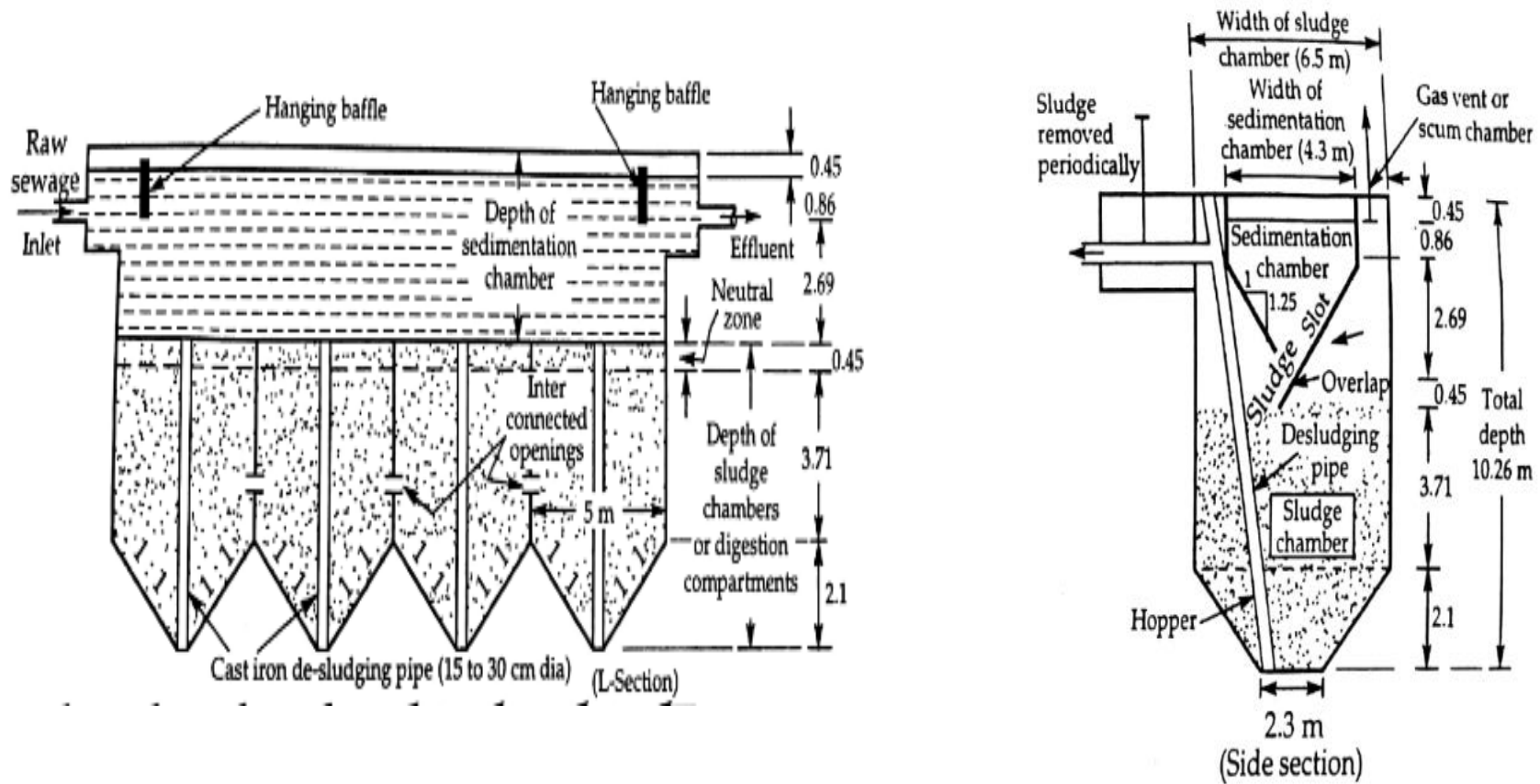
Figure 26

- **Balance capacity to be provided by rectangular portion of section**
= the capacity of each unit – capacity of each hopper
- **Height of this portion (D_r)**
= Balance capacity to be provided by rectangular portion of section/ A_1
- **Total height of digestion chamber = Free board + D_r + D_h**
- **Total height of tank from top to bottom**
= height of sedimentation chamber + height of sludge chamber
- **Total height = 9m to 11m**

Area

1 km ²	=	100 hectares (ha)
1 km ²	=	0.3861 square mile
1 km ²	=	247.105 acres
1m ²	=	10.7639 square feet
1 ha	=	10,000 m ²
1 ha	=	2.4711 acres
1 square mile	=	2.591 km ²
1 acre	=	0.4047 ha
1 acre	=	4046.86 m ²
1 acre	=	4840 square yard
1 square yard	=	9 square feet
1 square yard	=	0.8361 m ²
1 square foot	=	0.0929 m ²

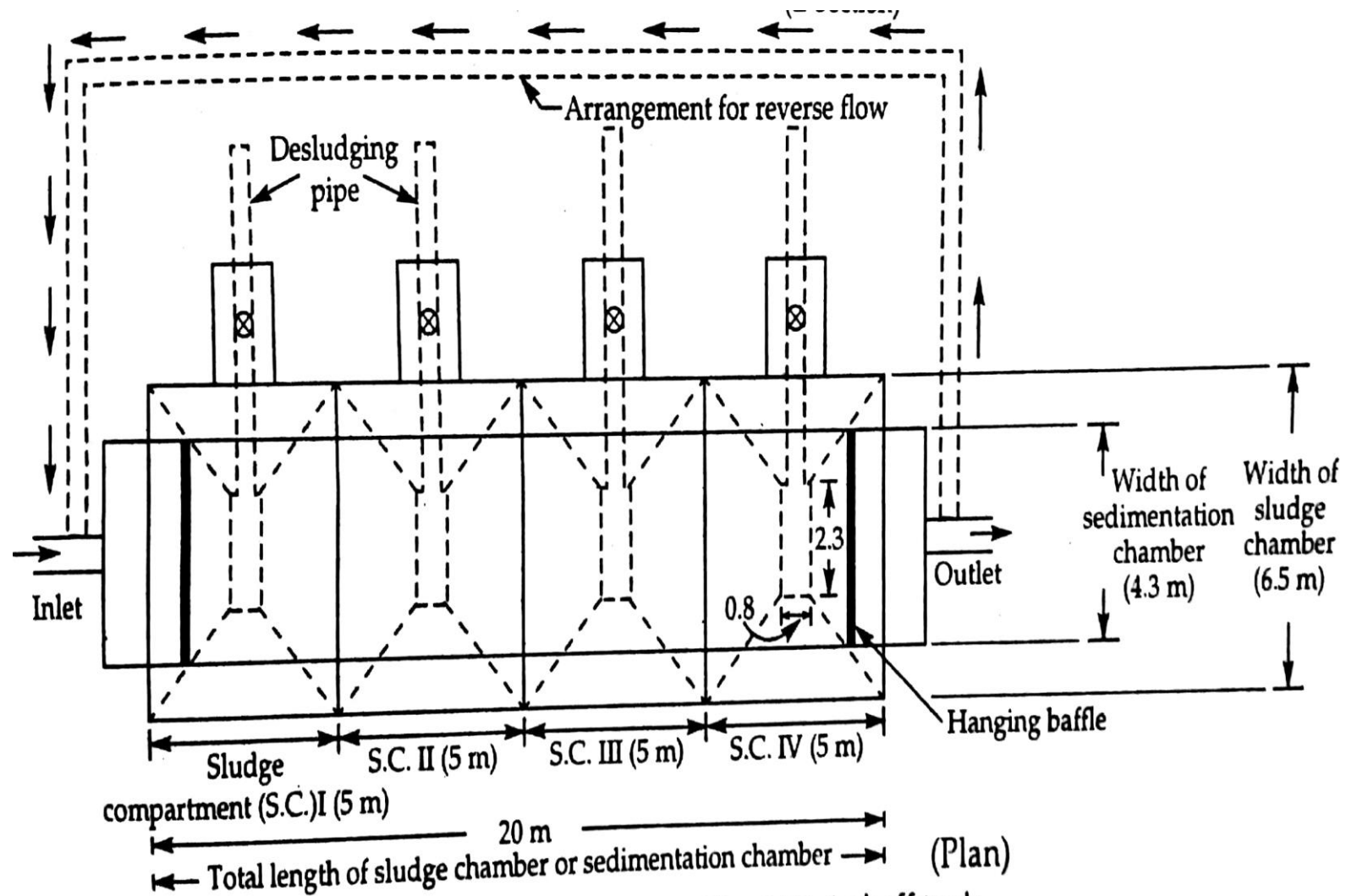
Figure 27



Don't match the measurement from design

Refer the drawing for understand

Figure 28



13. Design an imhoff tank to treat the sewage from a town with 62,232 populations. The rate of sewage may be assumed as 150 liters per head per day. Assume suitable data wherever necessary.

1.Design of sedimentation chamber:

Sewage discharge per day = population x rate of sewage liter per head per day

$$= 62232 \times 150$$

$$= 9.3348 \times 10^6 \text{ liters/day}$$

$$= 9.3348 \times 10^6 / 1000$$

$$= 9334.8 \text{ m}^3/\text{day}$$

Assume Detention period = 3 hours

Volume of sewage in detention period

$$= \text{sewage discharge per day} \times (\text{detention period} / 24)$$

$$= 9334.8 \times (3 / 24)$$

$$= 1166.85 \text{ m}^3$$

Assume an effective depth of 2.0 m and a width of 4.5m

Length of the sedimentation chamber =

$$(\text{Volume of sewage in detention period} / \text{effective depth} \times \text{width})$$

$$= (1166.85 / 2.0 \times 4.5)$$

$$= 129.65\text{m say } 130\text{m}$$

This length is very large for a single unit of sedimentation chamber. So Let us adopt 22m and width 4.5m,Then

$$L/B = 22/4.5 = 4.89$$

$$L/B = 3 \text{ TO } 5$$

Hence ok , it's within the limit.

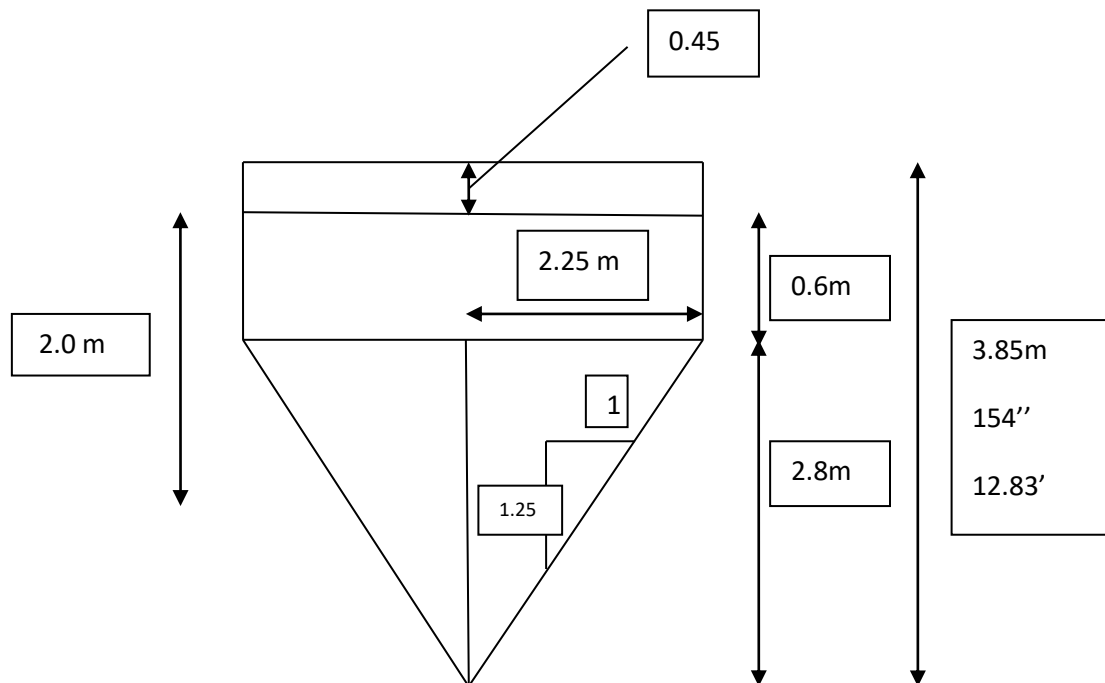
Discharge passing through single unit = $(1/6) \times \text{total discharge}$
 $= (1/6) \times 9334.8$
 $= 1555.8 \text{ m}^3/\text{day}$

Check for velocity : velocity = length of tank / detention time < 0.3 m/ min
 $= 22 / 2 \times 60$
 $= 0.183 \text{ m/min} < 0.3 \text{ m/minute}$

Hence safe

Check for surface loading $= Q/BL < 30000 \text{ L/m}^2/\text{day}$
 $= 1555.8 \times 1000 / 22 \times 4.5$
 $= 15715.15 < 30000 \text{ L/m}^2/\text{day}$

Hence safe



Assume the effective depth = width of sedimentation tank / 2

$$= 4.5/2 = 2.25\text{m}$$

Bottom side slope 1 H :1.25 V

From figure above (x) = 1.25 x(width of sedimentation chamber(B_s)/2)

$$= 1.25 \times (4.5/2)$$

$$= 2.81\text{m say } 2.8\text{m}$$

Y = effective depth - (x/2)

$$= 2 - (2.8/2)$$

$$= 0.6\text{m}$$

Free board = 0.45m

Total depth of sedimentation tank = free board + y + x

$$= 0.45 + 0.6 + 2.8$$

$$= 3.85\text{m}$$

Design of gas vent and neutral zone:

Provide a neutral zone of 0.45m below this depth of 3.85m. The tank , in general, is of 22m length, but below this 3.85m depth, it shall be divided into a number of compartments, say 4, each of length

Length of each compartment = total length / number of compartment

$$= 22 / 4$$

$$= 5.5\text{m}$$

The area of gas vent has now to be provided on both sides of the sedimentation chamber. This width should be about 25 to 30% of the total width of the tank. The total width of the gas vent (that is both sides of sedimentation chamber), assuming 0.3m thickness of chamber walls

$$= (\text{length of each compartment}) + (\text{length of each compartment} \times 30/100) -$$

$$B_s - (2 \times \text{thickness of chamber wall})$$

$$\begin{aligned}
 &= (5.5 + (5.5 \times (30/100)) - 4.5 - (2 \times 0.3)) \\
 &= (7.15 - 4.5 - 0.6) \\
 &= 2.05\text{m}
 \end{aligned}$$

This is about $(2.05/7.15) \times 100 = 28.67\%$ of the total width, and therefore, ok (between 25 to 30%)

Provide 0.75m width of gas vent on either side of sedimentation chamber.

Design of digestion chamber:

Assuming the capacity of the digestion chamber at 40 liters/capita

The capacity of the digestion chamber

$$\begin{aligned}
 &= \text{population} \times \text{capacity of digestion chamber in liters/capita} \\
 &= 62232 \times 40 \\
 &= 2.489 \times 10^6 \text{ liters} \\
 &= 2489 \text{ m}^3
 \end{aligned}$$

Now considering four compartments or 24 units in 6 tank with 5.5m width

The capacity of each unit

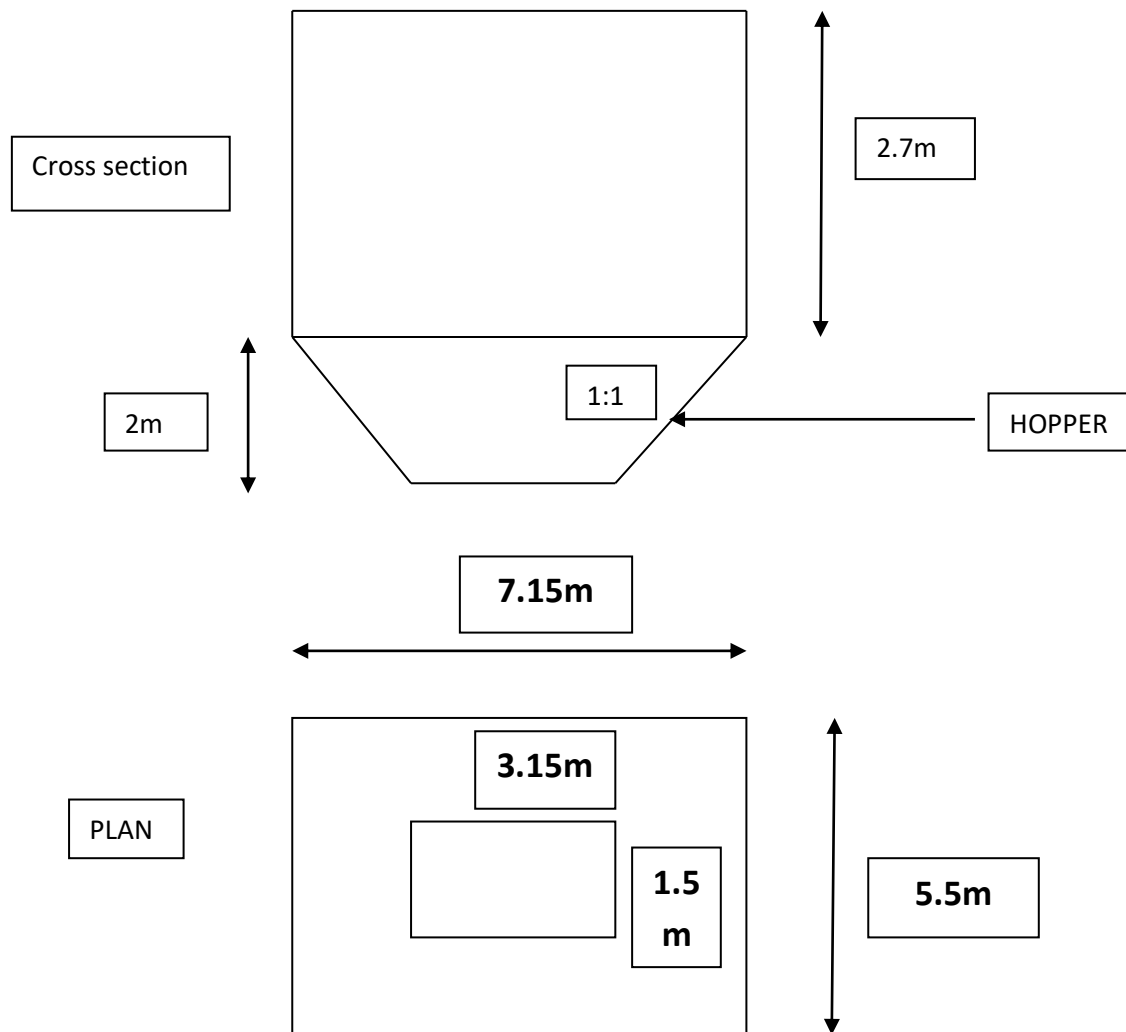
$$\begin{aligned}
 &= \text{the capacity of the digestion chamber} / \text{number of compartments} \\
 &= 2489 / 24 \\
 &= 103.71 \text{ m}^3
 \end{aligned}$$

Now assume the depth of hopper = 2.0m and side slopes 1:1 and bottom section as

$$= ((\text{length of each compartment}) + (\text{length of each compartment} \times 30/100) - 2 \times \text{depth of hopper}) \times (\text{length of each compartment} - 2 \times \text{depth of hopper})$$

$$\begin{aligned}
 &= ((5.5 + (5.5 \times 0.3)) - (2 \times 2)) \times (5.5 - (2 \times 2)) \\
 &= 3.15\text{m} \times 1.5\text{m} \\
 &= 4.725 \text{ m}^2
 \end{aligned}$$

- Length of single digester = (L_R) = Length of each compartment
- Overall width $= (B_R) = ((\text{length of each compartment}) + (\text{length of each compartment} \times 30/100))$



$$\begin{aligned}
 \text{Capacity of each hopper} &= \frac{h}{3} (A_1 + A_2 + (A_1 A_2)^{1/2}) \\
 &= \frac{2}{3} (39.325 + 4.725 + (39.325 \times 4.725)^{1/2}) \\
 &= 38.45 \text{ m}^3
 \end{aligned}$$

$$A_1 = L_R \times B_R = 7.15 \times 5.5 = 39.325 \text{ m}^2$$

$$A_2 = L_H \times B_H = 3.15 \times 1.5 = 4.725 \text{ m}^2$$

Balance capacity to be provided by rectangular portion of section

$$\begin{aligned}
 &= \text{the capacity of each unit} - \text{capacity of each hopper} \\
 &= 103.71 - 38.45 \\
 &= 65.26 \text{m}^3
 \end{aligned}$$

Height of this portion (D_r)

$$\begin{aligned}
 &= \text{Balance capacity to be provided by rectangular portion of section/ } A_1 \\
 &= 65.26/39.325 \\
 &= 1.659 \text{m says } 2.7 \text{m}
 \end{aligned}$$

Total height of digestion chamber = Free board + D_r + D_h

$$\begin{aligned}
 &= 0.45 + 2.7 + 2 \\
 &= 5.15 \text{m}
 \end{aligned}$$

Total height of tank from top to bottom

$$\begin{aligned}
 &= \text{height of sedimentation chamber} + \text{height of sludge chamber} \\
 &= 3.85 + 5.15 \\
 &= 9 \text{m}
 \end{aligned}$$

The height is well within practical limits (9m to 11m) and hence the design is O.K.

Mass

1	Pound	=	0.4536	Kilograms
1	Kilogram	=	2.20462	Pounds
1	Pound	=	453.6	Grams
1	Gram	=	0.00221	Pounds
1	Ton	=	1016.05	Kilograms
1	Kilogram	=	0.00098	Tons
1	Ounce	=	28.3495	Grams
1	Gram	=	0.03527	Ounces
1	Ton	=	1000000	Grams
1	Kilogram	=	0.01968	hundred weights

DESIGN OF HUMUS TANK

(Secondary settling tank)

1. Design of conventional trickling filter:

- Hydraulic loading = 22 to 44 million liters/hectare/day
- Surface area of the filter bed required
= sewage treated per day/hydraulic loading
- If you are need , Provide Number of units
- Surface area of single each unit
= Surface area of the filter bed required / Number of units
- Diameter of single unit = $(\text{area} / 0.78539)^{1/2}$
- Provide extra unit as stand by unit
- The depth of the filter provide 2m or greater than 2m.

2. Design of conical humus tank/ Dortmund tank:

- Detention period = 0.5 to 2 hours
- Over flow rating = 40,000 to 70,000 liters/ m²
- Capacity of tank = (sewage treated per day / 24) x detention period
- Volume of conical tank = $(1/3) \times (0.78539 \times d^2) \times \text{depth}$
- If you are need , Provide Number of tank
- Volume of single tank = (Capacity of tank / number of tank)
- Surface area of single tank = Volume of single tank / depth of tank
- Depth and diameter of tank = 7.5m to 9m
- Diameter of tank = $(\text{area} / 0.26179)^{1/2}$

SYSTEM TO SYSTEM CONVERSIONS FOR TEMPERATURE

$$^{\circ}\text{F} \rightarrow ^{\circ}\text{C} \quad C = \frac{5}{9} (F - 32)$$

$$^{\circ}\text{C} \rightarrow ^{\circ}\text{F} \quad F = \frac{9}{5} C + 32$$

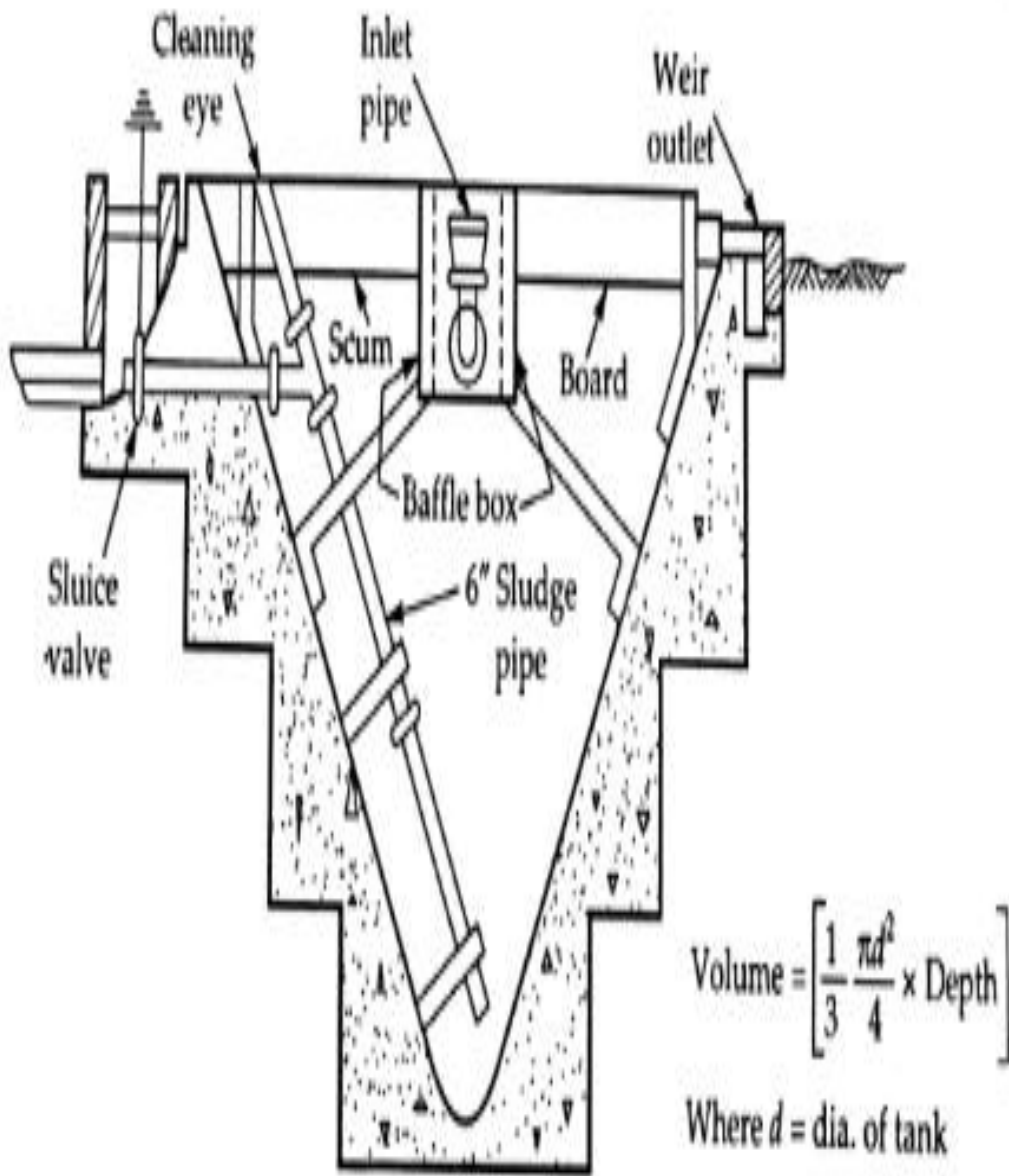


FIGURE 29

14. Design percolating filter with Dortmund tank for treating 9.5 million liter of sewage per day.

1. Design of conventional trickling filter:

Assume Hydraulic loading = 22 to 44 million liters/hectare/day

Surface area of the filter bed required

$$\begin{aligned} &= \text{sewage treated per day/hydraulic loading} \\ &= 9500000 \text{ liter per day} / 30000000/10000/\text{day} \\ &= 3166.67 \text{ m}^2 \qquad \qquad \qquad (1 \text{ hectare} = 100000\text{m}^2) \end{aligned}$$

Use three equal sized circular tanks

Surface area of single each unit

$$\begin{aligned} &= \text{Surface area of the filter bed required} / \text{Number of units} \\ &= 3166.67/3 \\ &= 1055.556 \text{ m}^2 \end{aligned}$$

$$\begin{aligned} \text{Diameter of single unit} &= (\text{area}/ 0.78539)^{1/2} \\ &= (1055.556/0.78539)^{1/2} \\ &= 36.66 \text{ say } 37\text{m} \end{aligned}$$

**Hence, use three trickling filters, each of 37m and Provide extra unit as stand by
The depth of the filter provide 2m or greater than 2m.**

2. Design of conical humus tank/ Dortmund tank:

Assume Detention period = 0.5 to 2 hours (0.0208 day to 0.0833 day)

Capacity of tank

$$\begin{aligned} &= ((\text{sewage} / 24 \text{ hrs}) \times \text{detention period}) \\ &= (9500000 \text{ liter} / 24) \times 2 \\ &= 79166.67 \text{ liter} \\ &= 79166.67 \times 0.001 \qquad \qquad \qquad (1 \text{ liter} = 0.001 \text{ m}^3) \\ &= 791.67 \text{ m}^3 \end{aligned}$$

Volume of conical tank = $(1/3) \times (0.78539 \times d^2) \times \text{depth}$

Use 14 Number of tank

Volume of single tank = (Capacity of tank / number of tank)
= $(791.67 / 5)$
= 158.334 m^3

Assume Depth of tank = 8.0m

Surface area of single tank = Volume of single tank / depth of tank
= $158.334 / 8$
= 19.79 m^2

Area of single tank = 14.72 m^2 to 21.20 m^2

Hence ok with in the limit

Diameter of tank = $(\text{area} / 0.26179)^{1/2}$
= $(19.79/0.26179)^{1/2}$
= 8.69 m say 8.7m

Hence, use 5 tanks with 8.7m diameter and depth 8.0m.

One addition tank as stand by also provided.

Using, totally 6 tanks.

UNITS OF WEIGHT IN THE METRIC SYSTEM

1 gram $\approx$ the weight of a paper clip

1 gram $\approx$ the weight of a raisin

1 gram = 1,000 milligrams (mg)

1 gram = 100 centigrams (cg)

1 kilogram (kg) = 1,000 grams

1 metric ton (t) = 1,000 kilograms

DESIGN OF AERATED GRIT CHAMBER

- Provide Detention period of about 3 minutes at the maximum rate of flow
- Aerator volume = peak flow rate x detention time
- Provided required number of chamber
- Volume of single aerated channel = aerated volume/number of chamber
- Width (B) / depth (d) ratio varying 1:1 to 5:1 (mostly 2:1)
- Depth = 2 to 5m
- Length = area / B x d
- Increase the length by about 20% to account for inlet and outlet condition
- Air supply requirement:
 - Air required = $0.0075 \text{ m}^3/\text{sec} - \text{m}$
- The daily great volume produced
= peak flow rate of sewage in m^3/day x grit produced per m^3 of sewage
- Distance of aerators from the bottom = 600mm to 1000mm
- Circumferential velocity above 150mm above waste water
= 0.6 - 0.75m/sec
- Mean velocity = 0.20m/sec – 0.30m/sec

4:Different Unit Weight of Engineering Materials.

<u>Bitumien</u>	=	<u>1340 kg/m³</u>
<u>Cement</u>	=	<u>1440 kg/m³</u>
<u>Cast Iron</u>	=	<u>7650 kg/m³</u>
<u>Steel</u>	=	<u>7850 kg/m³</u>
<u>Ice</u>	=	<u>913 kg/m³</u>
<u>Petrol</u>	=	<u>690 kg/m³</u>
<u>Water</u>	=	<u>1000 kg/m³</u>

15. Design of aerated grit chamber for treating municipal waste water with average flow rate of 50×10^6 liters .assume the peak flow rate to be 2.5 times the average.

Design:

Peak flow rate = 2.5 x average flow rate

$$= 2.5 \times 50 \times 10^6$$

$$= 125 \times 10^6 \text{ liters per day}$$

$$= (125 \times 10^6) / (24 \times 60 \times 60 \times 1000)$$

$$= 1.446 \text{ says } 1.45 \text{ m}^3/\text{sec}$$

Assume Detention period of about 3 minutes (180 sec) at the maximum rate of flow

Aerator volume = peak flow rate x detention time

$$= 1.45 \times 180$$

$$= 261 \text{ m}^3$$

Provided two number of chamber

Volume of single aerated channel = aerated volume/number of chamber

$$= 261 / 2$$

$$= 130.5 \text{ m}^3$$

Width (B) / depth (d) = 2/1 (2:1)

Assume depth = 2.75 m

B/d = 2

$$B = 2 \times d = 2 \times 2.75 = 5.5\text{m}$$

$$\begin{aligned}\text{Length} &= \text{volume} / B \times d \\ &= 130.5 / (2.75 \times 5.5) \\ &= 8.628\text{m}\end{aligned}$$

Increase the length by about 20% to account for inlet and outlet condition

$$\begin{aligned}\text{Length} &= ((20/100) \times 8.628 + 8.628) \\ &= 10.35\text{m}\end{aligned}$$

Air supply requirement:

$$\text{➤ Air required} = 0.0045 \text{ m}^3/\text{sec} - \text{m} \text{ to } 0.0125 \text{ m}^3/\text{sec}$$

$$\text{Typically} = 0.0075 \text{ m}^3/\text{sec} - \text{m}$$

$$\begin{aligned}\text{Air required} &= 0.0045 \text{ m}^3/\text{sec} - \text{m} \\ &= 0.0045 \times 10.35 \\ &= 0.07763 \text{ m}^3/\text{sec} \text{ or } 4.65 \text{ m}^3 / \text{minute}\end{aligned}$$

Volume of grid produced daily. Assume that 50m³/million cubic meter of grit is provided by incoming sewage, the daily grit volume produced

$$\begin{aligned}&= \text{peak flow rate of sewage in m}^3/24 \text{ hours} \times \text{grit produced per m}^3 \text{ of sewage} \\ &= 1.45 \text{ m}^3/\text{sec} (24 \times 3600) \times (50 / 10^6) \\ &= 6.26 \text{ m}^3\end{aligned}$$

Check for surface over flow rate:

$$\text{The settling velocity of smallest particle} = 0.024\text{m/sec}$$

$$\begin{aligned}\text{The actual surface velocity of chamber} &= \text{peak flow rate} / (L \times B \times d) \\ &= 1.45 / (10.35 \times 5.5 \times 2.5) = 0.0101\text{m/sec}\end{aligned}$$

$$0.024 > 0.0101 \text{ Hence safe.}$$

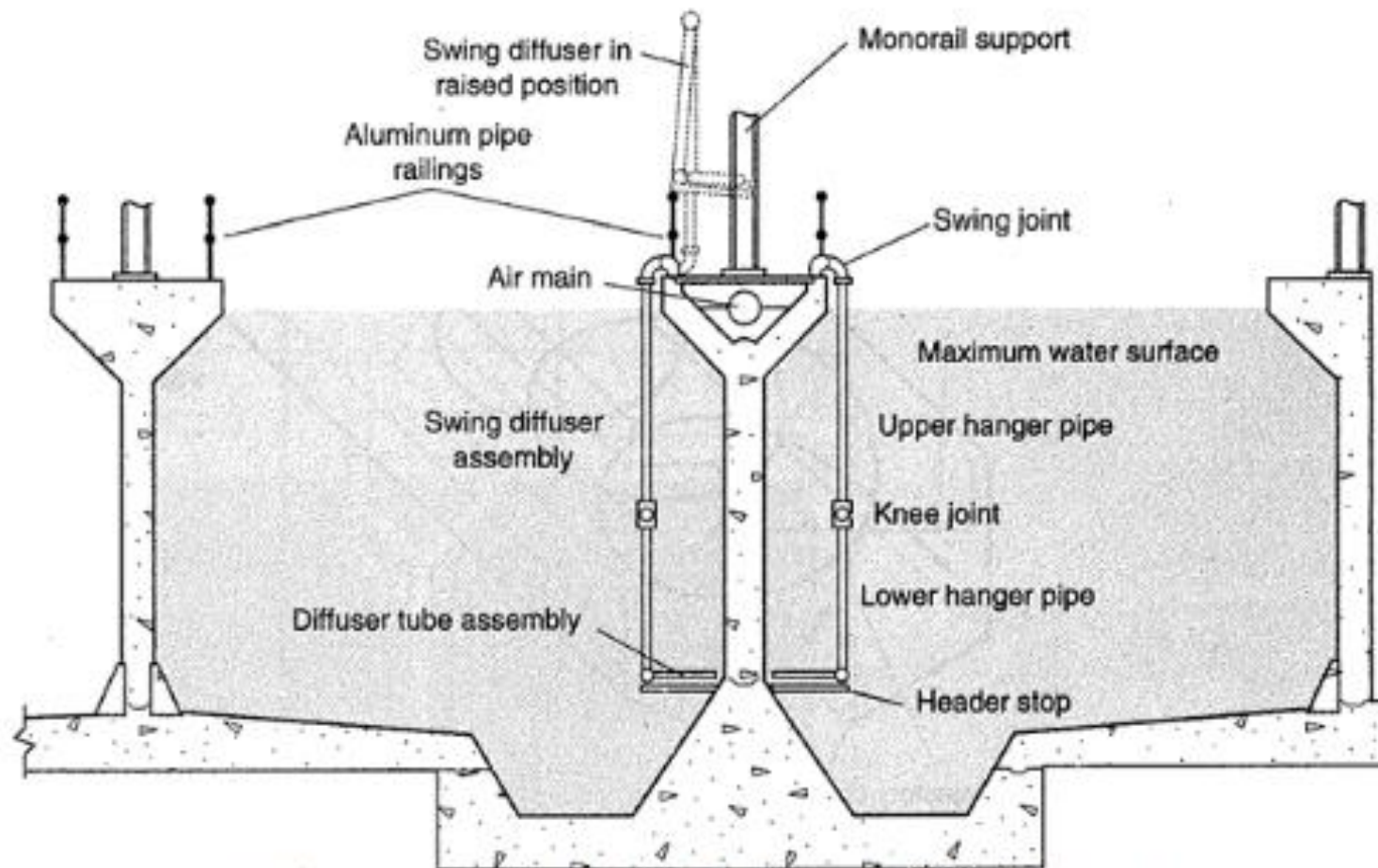
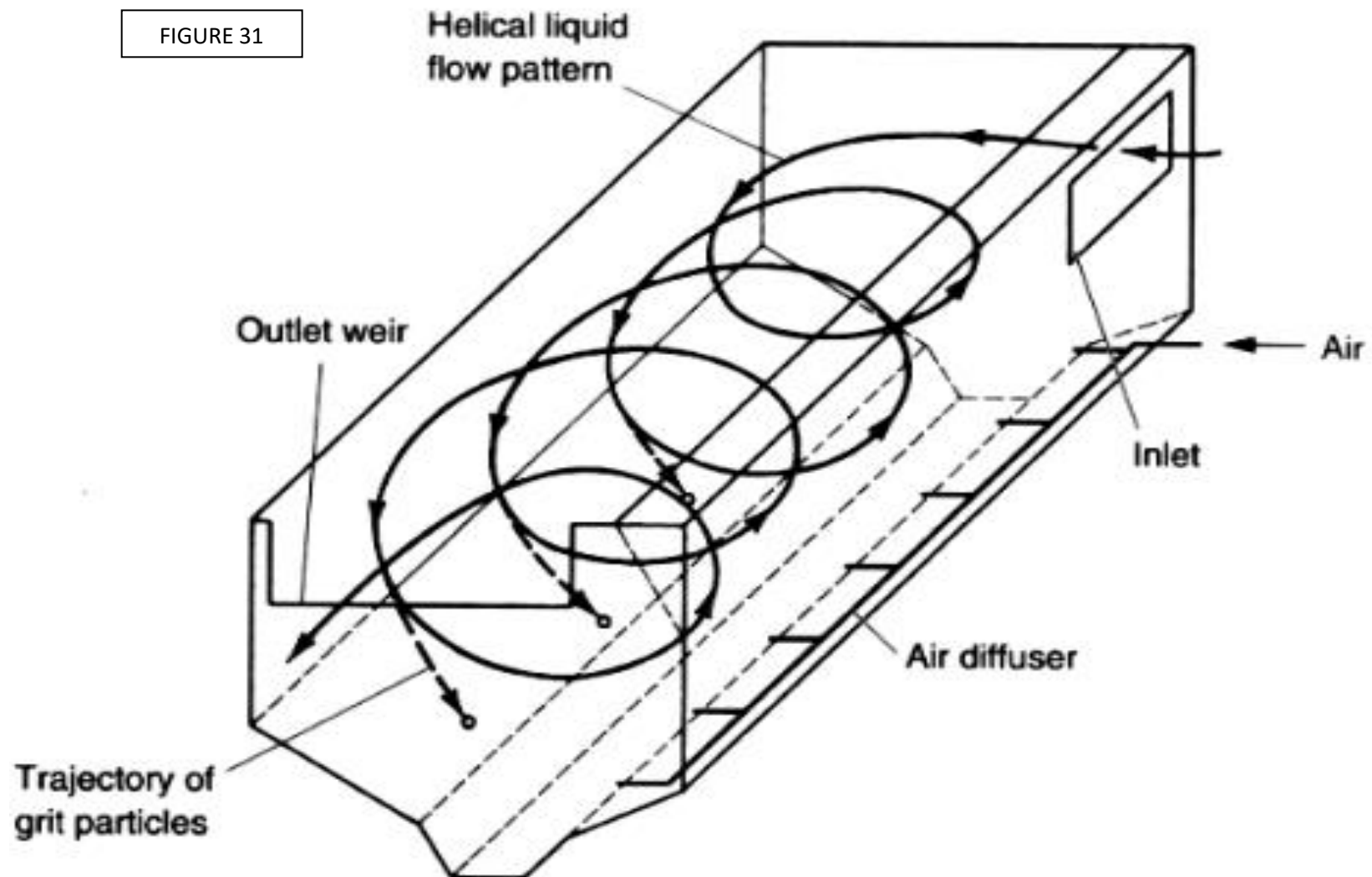


FIGURE 30

Typical Section Through Aerated Grit Chamber

TWO CHAMBERS

FIGURE 31



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CIVIL ENGINEERING

Simple designs

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