Hence provide outlet arrangement consisting of effluent launder with weirs on both sides of launder.

OUTLET ARRANGEMENT:

The outlet arrangement consists of effluent weir of V-notches, effluent launder, effluent box and a pressure outlet pipe.

i) **Effluent Weir:**

Length of effluent weir plate on each side of launder

$= \pi \times (57 - 1) = 175.93$ say 176 m

Provide 90° V-notches @ 20 cm centre to centre on both sides of the launder.

Total No. of notches $= 176 \times 5 = 880$

Average discharge per notch at average design flow

$$= \frac{50 x 10^6}{24 x 60 x 60 x 880 x 1000} = 6.58 \ x \ 10^{-4} cum/\sec.$$

The discharge through a V-notch is given by

$$Q = \frac{8}{15} \ C_d \sqrt{2g} \ \tan \ \frac{\theta}{2} \ H^{\frac{5}{2}}$$

for peak flow per notch, $Q = 6.58 \times 10^{-4} \times 2.25 = 1.48 \times 10^{-3}$ cum/s

for $C_d = 0.584$, $\Theta = 90$

Head over V-notch at peak flow =

$$\left(\frac{15 x 1.48 x 10^{-3}}{8 x 0.584 x \sqrt{2 x 9.81}}\right)^{\frac{2}{5}} = 0.065 \, m$$

Provide 8 cm deep 90 degree V-notches at 20 cm centre to centre.

(ii) **Effluent launder:**

Assume the width of effluent launder or channel to be 0.6 m. To compute depth of effluent launder, assume that the effluent launder discharges freely into the effluent box. Consequently the depth at the end of effluent channel may be assumed equal to critical depth of flow. Critical depth at the end of effluent launder, Y_2 is

$$Y_2=\left(\frac{(q' x L)^2}{(b^2 x g)}\right)^{\frac{1}{3}}$$

$$Y_2=\left[\frac{\left(\frac{50 x 10^3}{2 x 24 x 3600}\right)^2}{(0.6^2 x 9.81)}\right]^{\frac{1}{3}}=0.287\,m$$

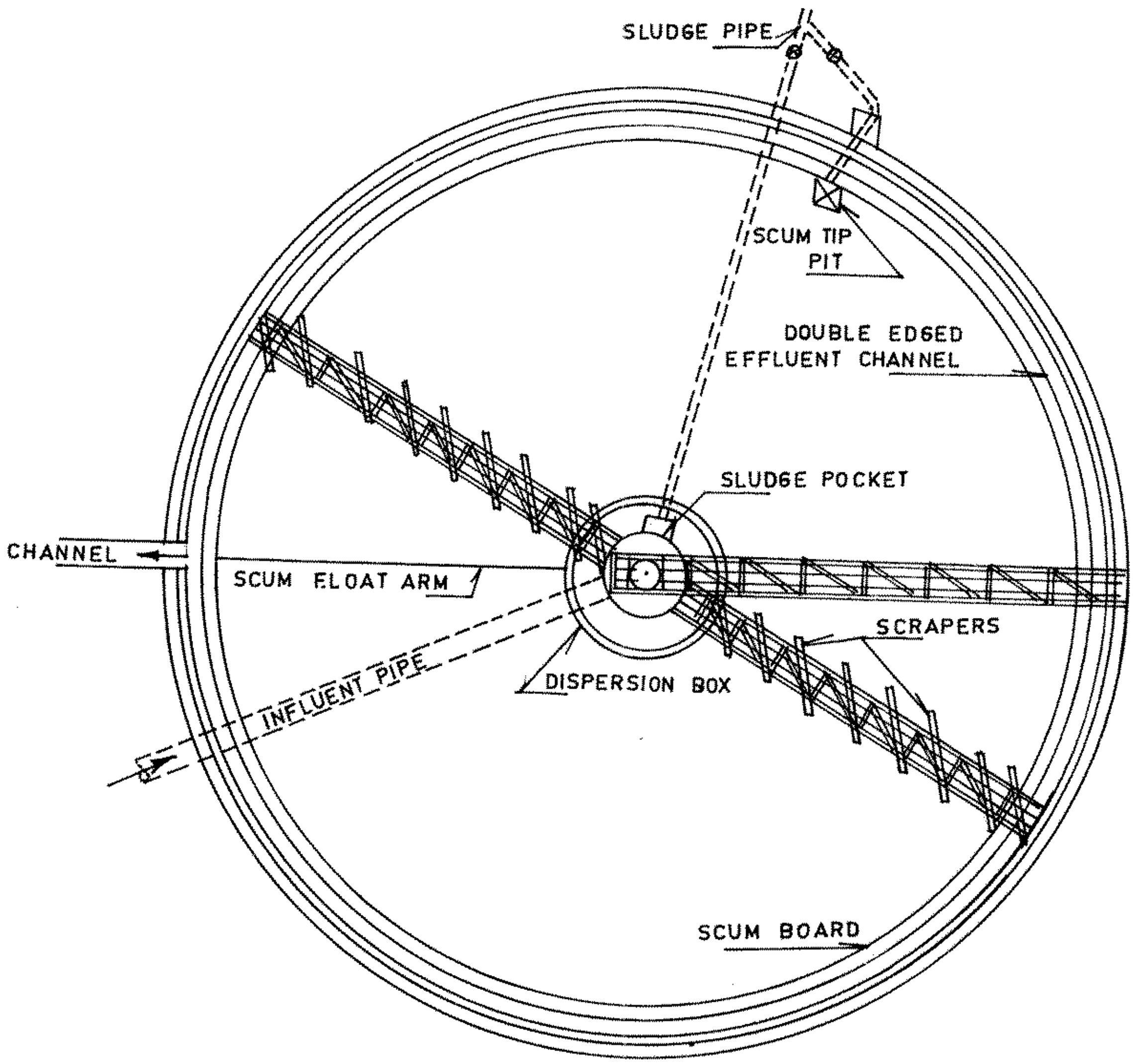
Depth of water at upper end of the trough, Y_1

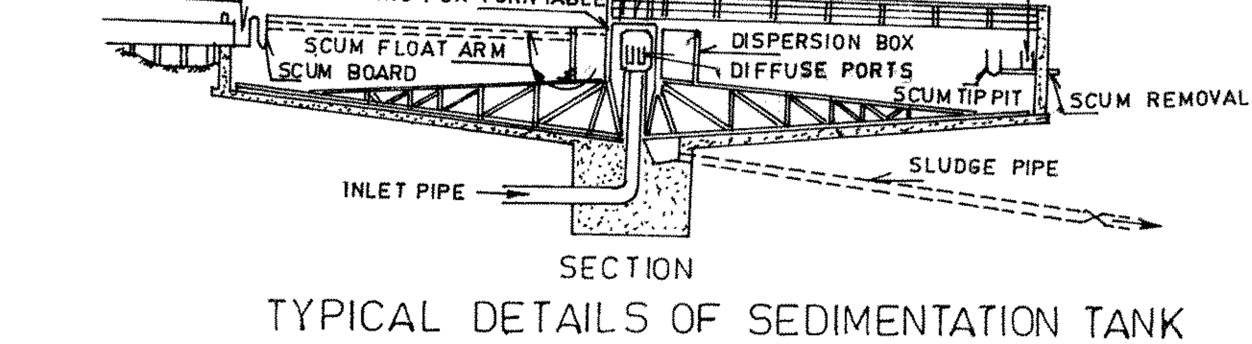
$$Y_1=\sqrt{Y_2^2+\frac{2x(q' x L x N)^2}{g x b^2 x Y_2}}$$

$$Y_1=\sqrt{0.287^2+\frac{2\left(\frac{50 x 1000}{2 x 24 x 3600} x 2\right)^2}{9.81 x 0.6^2 x 0.287}}=0.862\,m$$

Provide a depth of 0.95 m.

APPENDIX_12.1





SECTION

TYPICAL DETAILS OF SEDIMENTATION TANK

APPENDIX 13.1

DESIGN OF CONVENTIONAL ACTIVATED SLUDGE PROCESS

Given :

Flow = 50,000 m^3/d, Raw wastewater BOD_5 = 250 mg/l; SS = 400 mg/l; Minimum and maximum temperature = 18 and 32° C respectively; Primary Sedimentation efficiency for BOD and SS removal = 35% and 75% respectively; Primary and Secondary excess sludge SS concentration = 40 and 10 kg/m^3, Aeration equipment oxygen transfer efficiency under standard conditions = 1.8 kg O_2/kWh.

Aeration tank volume:

- BOD of influent to aeration tank = 250x65/100 = 162.5 mg/l
- For 90% BOD removal read Θ_c for 18° C from Fig.(13.3) = 6.5 days
- Adopt for conventional activated sludge MLSS = 2000 mg/l
- Assuming Y = 0.5 and k_d = 0.06/d from Eq.(13.7) calculate V = 8549 m^3
- HRT from Eq.(13.1) = 4.1 h, which is greater than 4 h, hence acceptable. However, if a larger HRT value is desired, repeat calculations assuming lower value of MLSS.
- The dimensions of the tank will be decided on the basis of aeration equipment requirements and conditions detailed in section 13.4.1.

Excess Sludge:

- Calculate $Q_w X_s$ from Eq.(13.3) = 2630461.5 g SS/d or 2630 kg/d.
- For 10 kg/m^3 SS concentration in secondary sludge, excess sludge volume = 263 m^3/d.

Sludge Recirculation:

- Calculate sludge recirculation ratio from Eq.(13.10) = 0.25, which is between 0.25 and 0.5, hence acceptable. However, provide for 0.33.
- Therefore sludge recirculation pump capacity
  = 0.33 x 50,000 = 16,500 m^3/d.

Oxygen Requirement:

- Calculate oxygen requirement from Eq.(13.8) assuming f = 0.68 ; = 7018420 g/d
- Calculate kg O_2 required/kg BOD removed = 0.96 which is between 0.8 and 1.0, hence acceptable.

Aerator Power Requirement:

- For field conditions; temperature = 32 degree C assuming C_L = 1 mg/l, C_s = 7.2 mg/l and α = 0.8 calculate, oxygen transfer capacity of available aeration equipment from Eq.(13.9) = 1.3 kg O_2/kW.h.
- Therefore aeration equipment power requirement
  = 7018 / 24 kg O_2 / h / 1.3 kg O_2 / kW.h = 225 kW.

Sludge Generated:

- Primary sludge solids
  = 50,000 m^3/ d x 400 g/m^3 x 0.75 x 1 kg / 1000g
  = 15000 kg/d

- Primary sludge volume
  = 15000 kg/d / 40 kg/m^3 = 375 m^3/d

- Secondary sludge solids (from earlier calculations)
  = 2630 kg/d.

- Secondary sludge volume = 263 m^3/d.

  Total sludge volume = 375 + 263 = 638 m^3/d.

APPENDIX 13.2

DESIGN OF FACULTATIVE AERATED LAGOON

Design a facultative aerated lagoon to serve 40,000 people. Sewage flow @ 180 lpcd = 7200 cu.m./day, Raw BOD_5 = 50 gcd or 277 mg/l and final BOD_5 is not to exceed 30 mg/l in winter. Average ambient air temperature in January is 18 deg. C and in summer 37 deg.C.

Lagoon Size

Assume detention time = 5 days

Lagoon volume = 7200 x 5 = 36,000 cu.m.

Let Lagoon dimensions be 70 m x 130 m x 4 m deep

Lagoon Winter Temperature

Use Eq.(13.3) to determine T_L. Assume T_i = 23° C

Hence,

$$\frac{5\ \text{days}}{4\ \text{m}} = \frac{(23 - T_L)}{0.49\,(T_L - 18)} \qquad \text{Hence, } T_L = 21^\circ \text{ C.}$$

Estimation of K

Assume K at 20° C = 0.7 per day

Hence, K at 21° C = 0.7x1.035 = 0.724/day.

D/UL Estimation

Keep lagoon geometry such that flow conditions are plug-flow type (i.e. D/UL = 0.2 approx.). This will be possible if a long and narrow lagoon (23m x 390 m) is provided (see Table 13.3) or baffles are provided within the rectangular lagoon of 70 m x 130 m to give a winding flow with the same effect. (See Fig.13.5).

BOD_5 Removal Efficiency (in Winter)

K x Θ = 0.724 x 5	=	3.62
See Fig.(13.4) at K x Θ = 3.62 and D/UL	=	0.2
Soluble BOD removal efficiency	=	92%
Namely, soluble BOD in effluent	=	22 mg/l
S.S.likely to flow out in effluent	=	35 mg/l (say)
BOD of VSS = 0.77 (0.6 x 35)	=	16 mg/l
Hence, BOD of effluent = 22 + 16	=	38 mg/l
Overall efficiency in winter	=	86%

In other months of the year, the efficiency will be higher and effluent BOD will be less than the above value.

Power Requirement

When efficiency = 86% and all BOD is removed aerobically,

O_2 required/day = 0.86 (1.4 x 2000 kg/d).

= 2,408 kg/d = 100 kg/hr.

Power needed = $\frac{100 \text{ kg/hr}}{(0.8)\ (2 \text{ kg } O_2/\text{KWh})}$

= 62.5 KW (i.e. about 80 HP)

Power level in Lagoon = $\frac{62.5 \text{ KW} \times 1000}{36{,}000}$

= 1.7 W/cu.m (acceptable)

Land Requirement

Net lagoon area = 9000 sq.m.

Area including embankments and slopes = 13,500 sq.m. (approx)

Area/person = 0.337 sqm/person

NOTE: If the lagoon was kept as a square shaped unit or a rectangular unit with say W:L = 1:2, the D/UL value would have been between 3.0 and 4.0 (namely, approaching completely - mixed conditions) and soluble effluent BOD would have increased to 49 mg/l, thus giving a total final effluent of about 65 mg/l instead of 38 mg/l seen above. Thus, lagoon geometry plays an important part in determining efficiency.

APPENDIX 14.1

DESIGN OF TRICKLING FILTER

Problem Statement:

Design a high rate trickling filter plant to treat settled domestic sewage with a BOD_5 of 200 mg/l for an average flow of 50 MLD. Assume a peak factor of 2.25. The desired BOD_5 of effluent is 10 mg/l.

Solution:

Several design approaches are available for the design of trickling filters. Two approaches will be used to design the trickling filter viz. (i) NRC equation and (ii) Rankine's approach.

Since the BOD_5 removal efficiency is high a two stage filtration system has to be used. The design of filters is done on the basis of average flow. However, the hydraulic design of the distribution arms, under drainage system, pipelines etc., is done for peak flow and checked for average flow.

i) Design Using NRC Equation

Assuming a BOD loading of 0.8 kg BOD_5 applied/m^3/d excluding recirculation, the volume of first stage filter,

$$\text{Volume} = \frac{BOD_5 \text{ load}}{BOD_5 \text{ loading}} = \frac{50 \times 200}{0.8}$$

$$= 12{,}500 \text{ m}^3$$

The efficiency of first stage filter using NRC equation,

$$E_1 = \frac{100}{1+0.44\sqrt{\dfrac{W_1}{V_1 F_1}}}$$

Adopting a recirculation ratio of 2.

$$F_1 = \frac{1+R_1}{(1+0.1R_1)^2} = \frac{1+2}{(1+0.1x2)^2} = 2.0833$$

W_1 = 50 x 200 = 10,000 Kg BOD_5/d

$$E_1 = \frac{100}{1+0.44\sqrt{\dfrac{10{,}000}{12{,}500x2.0083}}} = 78.6\%$$

The efficiency of second stage filter, E_2

$$E_2 = \frac{200-10}{200} x 100 - 78.6 = 16.4\%$$

The volume of second stage filter can be computed using the equation

$$E_2 = \frac{100}{1 + \frac{0.44}{(1-E_1)}\sqrt{\frac{W_1(1-E_1)}{V_2 F_2}}}$$

Adopting a recirculation ratio of one, the value of F_2 is

$$F_2 = \frac{1+R_2}{(1+0.1R_2)^2}$$

$$= \frac{1+1}{(1+0.1x1)^2} = 1.653$$

$$16.4 = \frac{100}{1 + \frac{0.44}{(1-0.786)}\sqrt{\frac{10,000(1-0.786)}{V_2 x 1.653}}}$$

V_2 = 274.8 m^3

ii) Rankine's Approach

Adopting an organic loading of 0.8 Kg $BOD_5/m^3/d$ as assumed in earlier case, the volume of first stage filter is 12,500 m^3

Adopting a filter depth of 1.5 m,

Filter area needed

$$= \frac{12{,}500}{1.5} = 8333 m^2$$

using a circular filter,

$$dia = \sqrt{\frac{8333 x 4}{\pi}} = 102.99 m$$

Since rotary distributors are available indigenously only upto 60 m, it is desirable to have a least three units.

$$= \sqrt{\frac{8333 x 4}{3 x \pi}} = 59.48 m$$

Say 60 m

Applying Rankine's formula for the first stage filter and varying value of R_1 = 0.5, 0.75, 1.0, 1.5, 2.0, 2.5 and 3.0 efficiency of first stage filter can be calculated by Rankine's equation.

$$E_1 = \frac{1 + R_1}{1.5 + R_1}$$

giving values of 75; 77.78; 80; 83.33; 85.77; 87.50 and 88.88 % respectively.

These values are entered in column 2 & 3 of Table 1 respectively.

Similarly the efficiency of second stage Filter

$$E_2 = \frac{1 + R_2}{2 + R_2}$$

Various values of R_2 and efficiencies are entered in columns 5 and 6 of Table 1. Column 4 gives the BOD_5 passing through the first stage filter.

Now, the combined efficiency of the filters required to give an effluent BOD_5 of 10 mg/l.

Efficiency of two stage Ec = $E_1 + E_2 (1-E_1)$

For a R_1 value of 0.5 this will be,

$0.95 = 0.75 + E_2 (1-0.75)$ or $E_2 = 0.8$

R_2 value from col.5 of Table = 3.0

Similarly R_2 values for various E_2 values for different R_1 values to obtain 95% efficiency are given in col.7 of Table 1.

TABLE 1
R_2 VALUES FOR DIFFERENT VALUES OF E_2 AND R_1 TO OBTAIN 95% EFFICIENCY

S.No.	R_1	E_1	S_4	R_2	E_2	R_2
	Recirculation Ratio of Ist stage filter	Efficienty of Ist stage filter.	BOD_5 passing through Ist stage filter.	Recirculation Ratio of 2nd stage filter.	Efficiency of 2nd stage filter	Values for various R_1 values to give 95% Efficiency.
1.	0.50	75.00	50.00	0.50	60.00	3.00
2.	0.75	77.78	44.44	0.75	63.64	2.50
3.	1.00	80.00	40.00	1.00	66.67	2.00
4.	1.50	83.33	33.33	1.50	71.43	1.50
5.	2.00	85.77	28.66	2.00	75.00	1.00
6.	2.50	87.50	25.00	2.50	77.78	0.50
7.	3.00	88.88	22.22	3.00	80.00	

The hydraulic loadings for different values of R_1 in terms of Kld/m^2 for the average flow.

$$= \frac{50 \times 10^6}{3 \times 10^3} \times \frac{4}{\pi \times (60)^2} \times (1+R_1) \text{ is worked out.}$$

$$= 5.89226 \times (1 + R_1)$$

R_1	Hydraulic Loading ($m^3/d/m^2$)
0.50	8.84
0.75	10.31
1.00	11.78
1.50	14.73
2.00	17.68
2.50	20.62
3.00	23.57

Choose R_1 = 2 for First Stage Filter and

R_2 = 1 for Second Stage Filter.

Organic loading (Recirculation included) for 3 filters of dia. 60m and depth 1.5 m

$$= \frac{50 x 10^6 x (200 + 2 x 28.66)}{10^3} x \frac{4}{3\pi x 60^2 x 1.5}$$

$= 1010.80\ g/d/m^3$

This is less than 1800 $g/d/m^3$ and therefore the equations are applicable.

Choosing an organic loading 0.5 $Kg/d/m^3$

$$= \frac{50 \times 28.66}{0.5} = 2866\ m^3$$

Adopting a depth of 1.0 m

Area of filter = 2866 m^2

Check for hydraulic loading

$$= \frac{50.00 x 10^3 x 1 x (1 x 1)}{2866} = 34.89 \frac{kld}{m^2}$$

Which is more than permissible.

Therefore area required for maximum permissible hydraulic loading of 30 Kld/m^2

$= 50 \times 10^3 \times [\ (1+1)/(30)\ ] = 3333.33\ m^2$

Adopting 3 circular Filters,

$$dia = \sqrt{\frac{333333 x 4}{3 x \pi}} = 37.6 m = 38 m$$

Adopting 3 units of 38 m dia and 1.0 m depth for 2nd Stage Filter.

iii) Hydraulic Design of First Stage Filter

This is designed for the peak flow + the recirculation of the average flow at the rates prescribed. In this case the recirculation is 2 times the average flow.

Total flow through the filters at the peak flow with 2.25 peak factor

$= 50 \times 2.25 + 2 \times 50 = 212.5$ Mld or 2.459 m^3/s

This flow is divided into 3 units

Therefore flow through each unit at peak flow = 0.82 m^3/s

Adopting a velocity of 2 m/s, dia of central column

$$= \sqrt{\frac{0.82 \times 4}{\pi \times 2}} = 0.722m$$

provide a central column = 0.75 m

check for velocity at average flow:

Ave. Flow = $50 \times 10^6 \times (1+2)$ = 150 Mld = 1.736 m^3/s

Therefore velocity at average flow =

$$\frac{1.736}{3} \times \frac{4}{\pi \times (0.75)^2} = 1.31 m/s (> 1 m/s)$$

Distributor:

Assuming rotary reaction spray type distributor with 4 arms:

Discharge per arm $= \frac{0.82}{4} =$ 0.205 m^3/s

Dia of filter = 60 m

Arm length = [(60 - 2) / 2] = 29m with 4 sections of 7.25m each

The flow in the arms has to be adjusted for every section of 7.25 m length in the proportion of the areas covered by these lengths of the arm. Therefore, the areas covered by the different lengths of the arms are calculated.

Let A_1, A_2, A_3 and A_4 be the areas covered by each length of arm starting from the centre. Allowing for 0.75 m dia in centre to be used up for central column etc.,the areas are

$A_1 = \pi (7.625^2 - 0.375^2) = 182.29\ m^2$

$A_2 = \pi (14.875^2 - 7.625^2) = 512.68\ m^2$

$A_3 = \pi (22.125^2 - 14.875^2) = 843.07\ m^2$

$A_4 = \pi (29.375^2 - 22.125^2) = 1173.46 m^2$

The proportionate area for each length of arm 1st i.e. from column to 7.625 m.

$$= \frac{A_1}{A_1 + A_2 + A_3 + A_4} = \frac{182.29}{2542.17} = 6.72\ \%$$

Similarly	2nd	18.91%
	3rd	31.09%
	4th	43.28%

Orifices:

Assuming a dia of 25mm for the orifices with Cd value of 0.6 and head causing flow equal to 1.5 m

$$discharge\ through\ each\ orifice = Cd\ x\ A\sqrt{2gh}$$

$$= 0.6 x \frac{\pi}{4} x 0.025^2 x \sqrt{2 x 9.81 x 1.5}$$

$= 0.001597\ m^3/s$

Therefore No.of Orifices required in each arm

$$= \frac{\text{Total discharge through arm}}{\text{Discharge through each orifice}}$$

$$= \frac{0.205}{0.001597} = 128.36 \text{ say } 129$$

No. of orifices in each section of the arm is

Ist section (6.72 / 100) x 129 = 9

2nd section (18.91 / 100) x 129 = 25

3rd section (31.09 / 100) x 129 = 40

4th section (43.28 / 100) x 129 = 56

Spacing of Orifices:

1st Section 9 Nos. in 725 cm i.e. 725/9 = 80cm c/c

2nd Section 25 Nos. in 725 cm i.e. 725/25 = 29cm c/c

3rd Section 40 Nos. in 725 cm i.e. 725/40 = 18cm c/c

4th Section 56 Nos. in 725 cm i.e. 725/56 = 13cm c/c

Diameters of different sections of the arm:

The flow through velocity in the arm should be less than 1.2 mps

a) ***Discharge through 1st section = 0.205 m^3/s***

Crosssectional area with 1.2 mps = (0.205 / 1.2) = 0.1708 m^2

Assuming circular section, dia of pipe

$$= \sqrt{\frac{0.1708 x 4}{\pi}} = 0.466 m \; say 470 mm$$

b) ***Discharge through 2nd section***

= (1-0.0672) x 0.205 = 0.1912 m^3/s

For V = 1.2m/s

$$dia = \sqrt{\frac{0.1912 x 4}{1.2 x \pi}} = 0.45 m \; say 450 mm$$

c) ***Discharge through 3rd section***

= [1 - (0.0672+0.1891)] x 0.205 = 0.1525 m^3/s

For V = 1.2m/s

$$dia = \sqrt{\frac{0.1525 x 4}{1.2 x \pi}} = 0.402 m \; say 400 mm$$

d) ***Discharge through 4th section***

= 0.4328 x 0.205 = 0.0887 m^3/s

For V = 1.2 m/s

$$dia = \sqrt{\frac{0.0887x4}{1.2x\pi}} = 0.3067m\ say\ 310mm$$

Under Drainage System:

Total discharge through each filter at peak flow = $0.82 m^3/s$.

The underdrainage system is designed with a peripheral collecting channel fed by semi circular laterals placed at 0.6 m c/c with a slope of 2.5% in each half circle. The invert level of all laterals at their junction with the peripheral main collecting channel is kept the same R.L.

Average discharge per lateral:

$$= \frac{0.82}{100 \times 2} = 0.0041\ m^3/s$$

$$ar^{2/3} = \frac{nq}{S^{1/2}} = \frac{0.015 \times 0.0041}{0.025^{1/2}} = 0.000389$$

The laterals are designed to flow half full to provide for proper ventilation.

i.e.

$$\frac{a}{A} = 0.25\ ; \quad q = \frac{q}{d_o^2} = \frac{\pi}{4} \times 0.25 = 0.1963$$

From Appendix 26

for (a / d_o^2) of 0.1963 ; $(ar^{2/3} / d_o^{8/3}) = 0.05915$

$d_o = 0.152$ m

adopting 16 cm dia

$(ar^{2/3} / d_o^{8/3}) = (0.000389 / 0.16^{8/3}) = 0.051$

From Appendix 26

corresponding $(a / d_o^2) = 0.1753$

$$Velocity = \frac{0.0041}{0.1753 \times 0.16^2} = 0.9136\ m/s\ (> 0.75\ m/s\ required)$$

Check for Velocity at Average Flow:

Total discharge = 50+2x50 = 150 Mld = 1.736 m^3

1.736

Flow through each filter $= \frac{\quad}{3} = 0.579\ m^3/s$

Average flow per lateral $= [\,0.579 / (100 \times 2)\,] = 0.00290\ m^3/s$

$$ar^{2/3} = \frac{0.015 \times 0.0029}{0.025^{1/2}} = 0.000275$$

$$\text{and}\quad \frac{ar^{2/3}}{d_o^{8/3}} = \frac{0.000275}{(0.16)^{8/3}} = 0.0365$$

corresponding $(a / d_o^2) = 0.1379$

$\therefore$ Velocity $= [\,0.0029 / (0.1379 \times 0.16^2)\,] = 0.8215$ m/s (>0.6 m/s required)

The laterals are covered with perforated blocks capable of withstanding the load of the filter media. It should be ensured that there is at least 15% of the total filter area available in the form of inlet openings for the flow into the laterals to ensure proper ventilation.

In the present design the total surface area of the laterals at the floor level of the filter is about 20% of the filter area. Therefore it is to be provided with cover blocks having about 75% openings so that inlet area available is about 15% of the filter area.

DESIGN OF MAIN COLLECTION CHANNEL

It is desirable to provide the main collection channel along the periphery of the filter. The flow is divided into two and the flow from each semi circle is collected in the peripheral main channel which is laid to a constant slope of 0.5%. The filter can be divided into four segments and the main channel checked to see if free fall conditions exist while flow from the laterals of each segment falls into it.

To provide a free fall from the invert of the laterals assume the depth of flow to be 5% less than depth of semicircular section.

$$\text{i.e.}\quad \frac{y}{d_o} = \frac{0.95}{2} = 0.475$$

1st Segment:

$q = 0.1 \times 0.82 = 0.082\ m^3/s$

from Appendix 26

for $(y / d_o) = 0.475$;

& $(ar^{2/3} / d_o^{8/3}) = 0.1426$

$(a / d_o^2) = 0.3677$

for a slope of 0.5% and $n = 0.015$

$$ar^{2/3} = \frac{nq}{S^{1/2}} = \frac{0.015 \times 0.082}{0.005^{1/2}} = 0.01739$$

$$d_o = [(0.01739) / (0.1426)]^{3/8} = 0.4543 \text{ m}$$

Adopting 46 cm or 0.46 m dia & 0.5% slope

$$\frac{ar^{2/3}}{do^{8/3}} = \frac{0.01739}{(0.46)^{8/3}} = 0.1379$$

And for this $(y / d_o) = 0.4658$ and $(a / d_o^2) = 0.3585$

$$\text{Velocity} = \frac{0.082}{0.3585 x 0.46^2} = 1.08 \text{ m/s} \ (> 0.75 \text{ m/s required})$$

2nd Segment:

$q = 0.25 \times 0.82 = 0.205\ m^3/s$

vertical depression at the end of the 2nd section

$= (\pi D / 4) \times (0.5 / 100) = [\pi \times (60 / 4) \times (0.5 / 100)] = 0.24$ m

Total additional flow in this section

$0.15 \times 0.82 = 0.123\ m^3/s$

Flow that can be accomodated

$= 0.24 \times 0.46 \times 1 = 0.1104\ m^3/s$ (Assuming 1 m/s velocity)

Hence choose a bigger section say 53 cm.

Redesign of Ist Segment:

$ar^{2/3} = 0.01739$

$$\frac{ar^{2/3}}{d_o^{8/3}} = \frac{0.01739}{(0.53)^{8/3}} = 0.09453$$

For this value, $(a / d_o^2) = 0.27$ and $(y / d_o) = 0.3778$

$$\text{Velocity} = \frac{0.082}{0.27 \times 0.53^2} = 1.081 \text{ mps} \ (> 0.75 \text{ mps required})$$

Check for Average Flow (Recirculation included)

Flow in Segment 1,

$= (1.736 / 3) \times 0.1 = 0.0579\ m^3/s$

$$\frac{ar^{2/3}}{do^{8/3}} = \frac{0.0579 \times 0.015}{0.005^{1/2}} \times \frac{1}{(0.53)^{8/3}} = 0.06676$$

for this $(a / d_o^2) = 0.2113$ and $(y / d_o) = 0.314$

$$\text{Velocity} = \frac{0.0579}{0.2113 \times 0.53^2} = 0.9755 \text{ mps } (> 0.75 \text{ mps required })$$

2nd Segment:

$q = 0.205 \ m^3/s$

$$ar^{2/3} = \frac{0.205 \times 0.015}{0.005^{1/2}} = 0.04349$$

$$\frac{ar^{2/3}}{do^{8/3}} = \frac{0.04349}{(0.53)^{8/3}} = 0.2364$$

For this value, $(a / d_o^2) = 0.541$ and $(y / d_o) = 0.65$

$$\text{Velocity} = \frac{0.205}{0.541 \times (0.53)^2} = 1.349 \text{ mps}$$

Depth of Flow = 0.65 x 0.53 = 0.3445 say 0.35m

Depth from invert of channel to invert of lateral =

$$\frac{0.53}{2} + 0.24 = 0.5\text{m}$$

Clearance = 0.5 - 0.35 = 0.15 m ensuring free flow conditions

3rd Segment:

$q = 0.4 \times 0.82 = 0.328 \ m^3/s$

Assuming depth of flow above semi circular section to be x

$$r = \frac{a}{p} = \frac{\frac{\pi R^2}{2} + 0.5x}{\pi R + 2x}$$