$$= \frac{0.1104+0.5x}{0.8324+2x}$$

$$0.328 = \frac{1}{0.015}(0.1104+0.5x)\left(\frac{0.1104+0.5x}{0.8324+2x}\right)^{2/3} x(0.005)^{1/2}$$

or

$$\frac{(0.1104+0.5x)^{\frac{5}{3}}}{(0.8324+2x)^{\frac{2}{3}}} = 0.06958$$

solving x = 0.225

and depth of Flow = 0.265 + 0.225 = 0.49 m

against available depth of 0.265 + 0.36 = 0.625 m which ensures free flow conditions.

4th Segment:

q = 0.5 x 0.82 = 0.41 m^3/s

Let y be depth of flow above semi circular section,
then as in 3rd segment.

$$r = \frac{a}{p} = \frac{(0.1104+0.5y)^{\frac{5}{3}}}{(0.8324+2y)^{\frac{2}{3}}}$$

$$\frac{0.41 x 0.015}{(0.005)^{\frac{1}{2}}} = 0.08697$$

solving by trial and error, y = 0.315m

Depth of Flow = 0.265 + 0.315 = 0.58m

against available depth of = 0.265 + 0.48 = 0.745m ensuring free flow condition

$$\text{Velocity} = \frac{0.41}{0.1104 + 0.1575} = 1.53 \text{ mps}$$

Design of Exit Channel

q = 0.82 m^3/s for each filter.

Assuming a channel of rectangular section with a slope of 0.5%

p = 2d+w and A = wxd, r = (A / p) or [r / (w + 2d)] = [(w x d) / (w + 2d)]

q = [(1 / n) $ar^{2/3}$ $S^{1/2}$; 0.82 = (1 / 0.015) x w x d x $[(w \times d) / (2d + w)]^{2/3}$ x $(0.005)^{1/2}$

or

$$\frac{(wxd)^{\frac{5}{3}}}{(2d+w)^{\frac{2}{3}}} = 0.174$$

or

$$\frac{(wxd)^5}{(2d+w)^2} = 0.005268$$

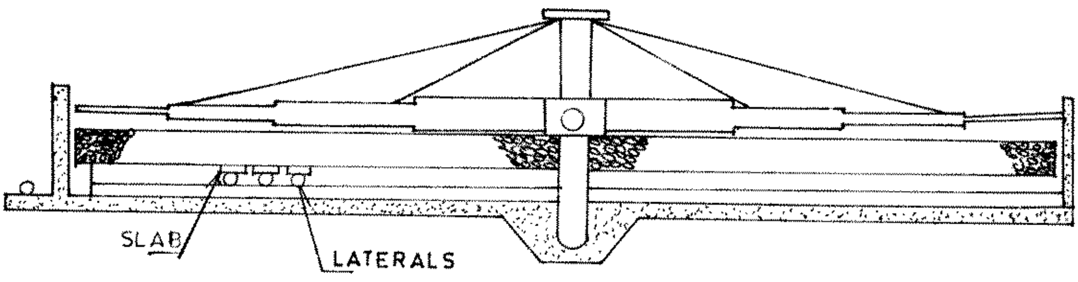
Assuming a depth of 0.45m of exit channel, by trial & error

w = 1.05 m

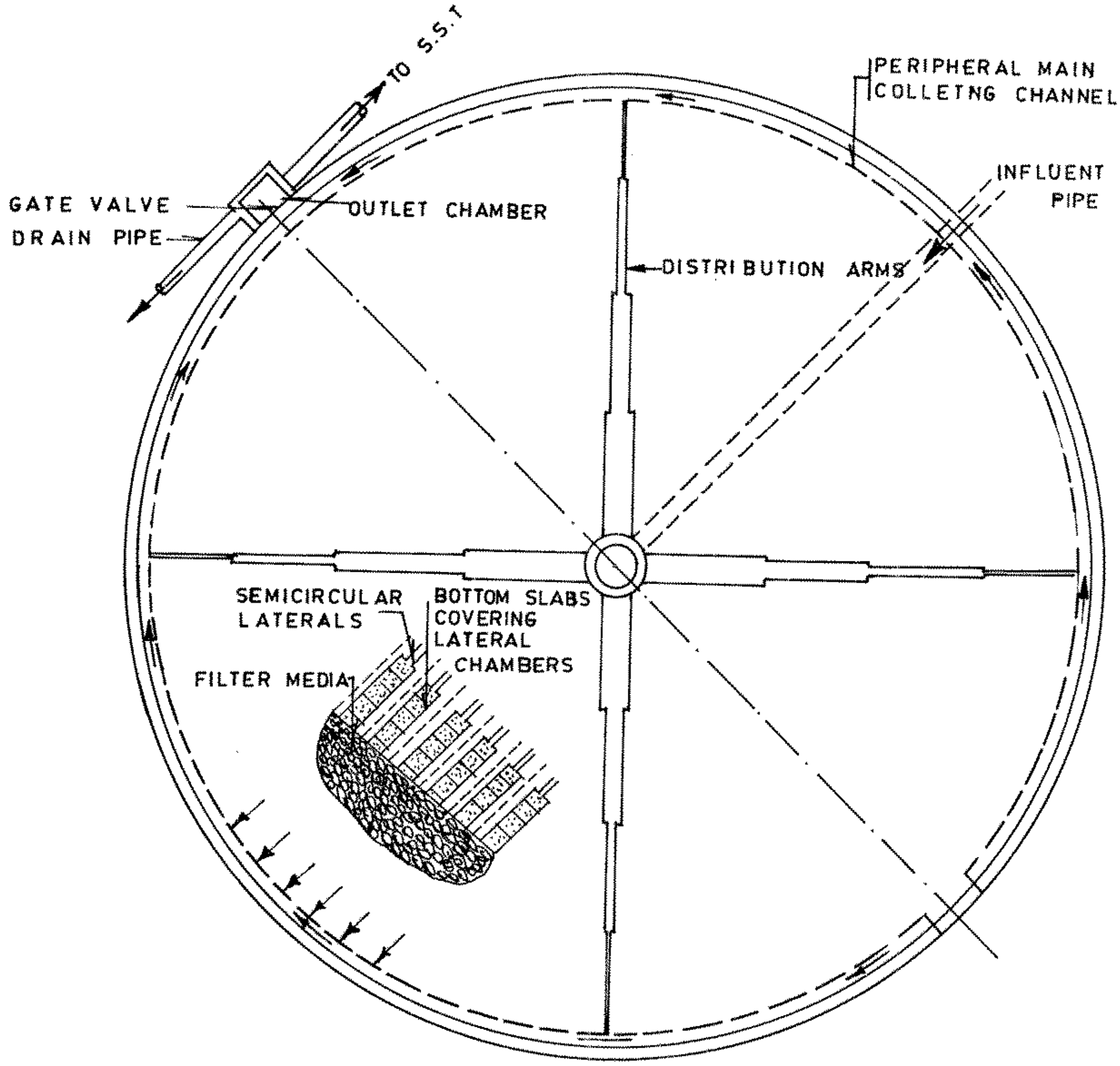
If width w of channel is 1.1 m. then depth of flow of channel d by trial & error.

$$\frac{(1.1 \times d)^5}{(2d + 1.1)^2} = 0.005268$$ we get d = 0.415 m

APPENDIX - 14.1



SECTION



PLAN

TYPICAL DETAILS OF TRICKLING FILTER

therefore effluent channel from each filter will be of size 1.1m x 0.45m with 0.5% slope.

Ventilation:

Since the filter is large having a dia of 60m, provision for open grating area to be mode at 1/250 of the filter area.

Area of grating needed = $[(\pi \times 60^2) / (4 \times 250)] = 11.32\ m^2$ say $12\ m^2$

Therefore provide 12 Nos. of gratings of size 4m x 0.25m providing a total of $12\ m^2$ ventilation area.

2nd Stage Filter:

The details of Second Stage Filter are also worked out on similar lines.

APPENDIX 14.2

DESIGN EXAMPLE OF ROTATING BIOLOGICAL CONTACTOR

Problem Statement:

Design rotating biological contactor modules to treat 50,000m^3/d of primary settled sewage with the following assumptions:

Hydraulic loading rate	=	110 $l/m^2.d$
Diameter of the discs	=	3.5 m
Centre to centre spacing between discs.	=	20 mm

Solution:

i) Total surface area of discs $= \dfrac{\text{Flow}}{\text{Hydraulic loading rate}}$

$$= \frac{50,000 \times 10^3}{110} = 454545.5\ m^2$$

ii) Surface area of One discs (neglecting the thickness) $= \dfrac{\pi \times (3.5)^2 \times 2}{4}$

$$= 19.24\ m^2$$

iii) Number of discs $= \dfrac{454545.5}{19.24} = 23625$

iv) Minimum length of shaft on which discs are mounted $= 23625 \times 0.02 = 472.5$ m

v) Provide 40 modules of 12 m length with 23625 discs.

vi) Hydraulic residence time assuming 50% submergence of discs:

$$= \frac{(\pi / 4) \times (3.5 + 0.1)^2 \times 40 \times 12 \times 24 \text{ hrs} \times 0.5}{50,000} = 1.17 \text{ hours}$$

APPENDIX 15.1

DESIGN EXAMPLE OF FACULTATIVE STABILIZATION POND

Design a facultative stabilization pond to treat 5000 m^3/d municipal wastewater, BOD_5 230 mg/l, from a town (population 25,000 persons) located in Central India, latitude 22 deg.N, elevation 100 m above sea level. The average temperature in January is 18° .C. The effluent from the pond is to be used for irrigation.

Solution :

Pond Size:

Permissible organic load according to temperature correlation = 20 x 18 - 120 = 240 kg BOD/ha.d

Permissible organic load according to latitude and elevation = 235/(1 + 0.003 x 100) = 180 kg BOD/ha.d

Adopt a conservative loading rate of 200 kg BOD/ha.d

BOD load from the town = 5000 x 0.23 = 1150 kg/d

Therefore pond area = 1150/200 = 5.75 ha

Adopt an average depth of 1.5 m

Therefore pond detention time = $5.75 \times 10^4 \times 1.5/5000 = 17.25$ d.

Provide three ponds of equal volume and surface area; two primary ponds in parallel and one secondary pond in series receiving the effluent of the two primary ponds. Use of multiple ponds improves performance from view points of stability, efficiency of treatment and maintenance. However, it requires greater land area for the same pond surface area.

Check for Detention Time:

For 90% BOD reduction, the BOD reaction rate constant = 0.2/d for plug flow condition. The total overall detention time, Θ, is given by:

$0.1 = \exp - 0.2\ (2 \times \Theta/3 + \Theta/3)$, or $\Theta = 11.5$ d

For a conservative estimate, for completely mixed condition in all three ponds, the total overall detention time is given by:

$0.1 = 1/(1+0.2 \times 2\ \Theta/3)\ (1 + 0.2 \times \Theta/3)$, or $\Theta = 22.5$ d

In actual conditions the hydraulic regime in the ponds is going to be between the two ideal conditions of plug flow and completely mixed flow. The detention time of 17.25 d is therefore acceptable.

Check for Microbial Quality for Irrigation

WHO guidelines recommend sewage retention in stabilization ponds for 8 - 10 days for irrigation of cereal, fodder and industrial crops and trees. This assures removal of intestinal nematodes from sewage. The design meets this requirement.

For irrigation of crops likely to be eaten uncooked, the guide lines recommend a faecal coliform limit of 1000 organisms/100 ml. For microbial reduction rate constant of 2.0/d at 20° C or 1.4 at 18° C, and influent faecal coliform concentration = 10^7/100 ml, the effluent concentration N is given by

$$N = 10^7/(1+1.4 \times 2 \times 17.25/3)\ (1+1.4 \times 17.25/3)$$

or

$$N = 64,600/100\ ml$$

Therefore the design does not meet the criteria of irrigation water quality for crops likely to be eaten uncooked. If two maturation ponds, each of 17.25/3 d detention time are provided in series after the secondary pond, the effluent concentration is expected to be:

$$N = 10^7/\ [1+1.4 \times 2 \times (17.25\ /\ 3)]\ \ (1+1.4 \times 17.25/3)^3$$

$$= 788/100\ ml.$$

The above calculations are based on assumption of complete mixing. In actual condition the performance is likely to be better.

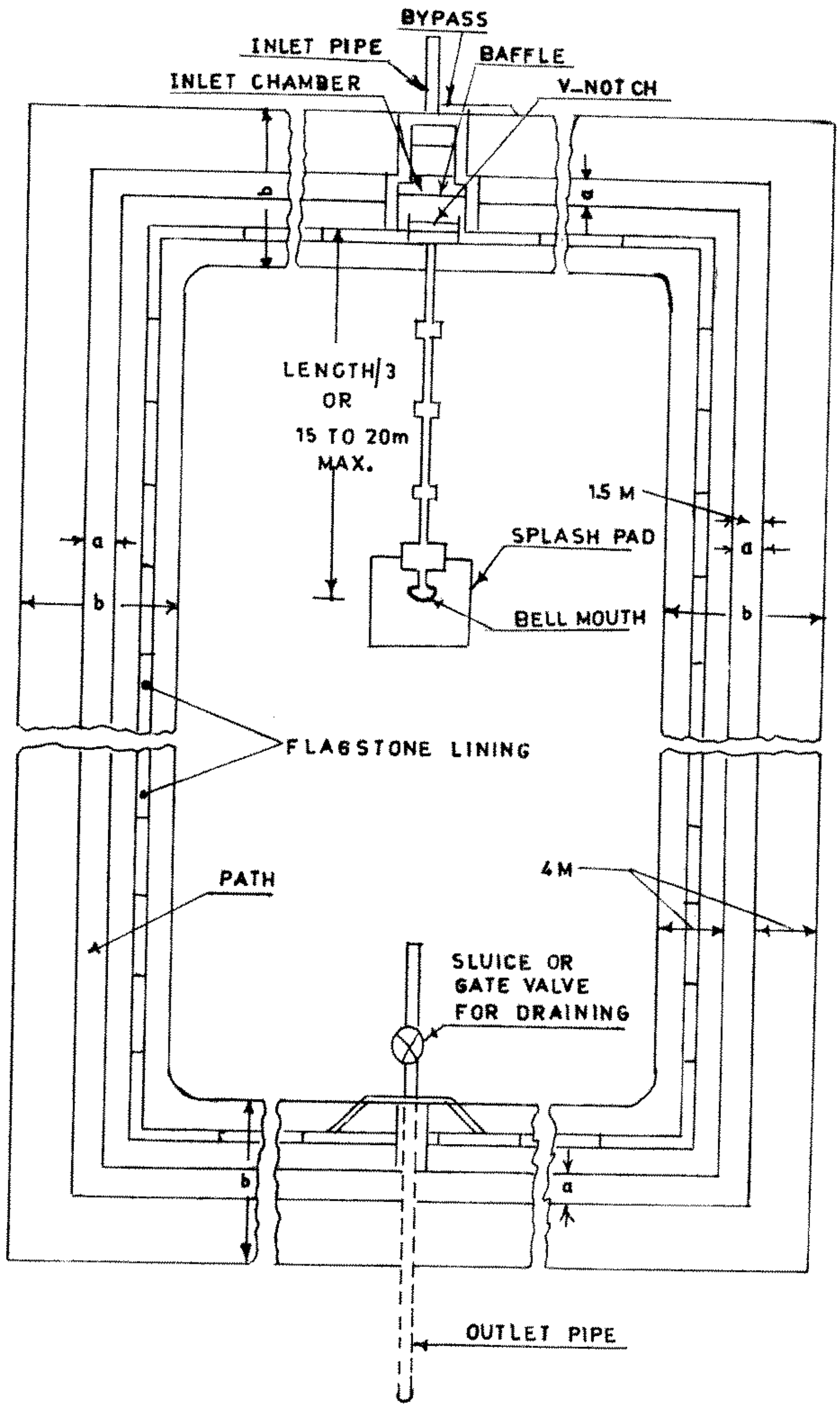
Sludge Accumulation:

Most of the sludge will accumulate in primary ponds. Assuming 0.75 m deep allowable sludge deposition, capacity available = 0.75 x (2/3) x 5.75 x 10^4 = 28750 m^3. For 0.07 m^3/person/year sludge accumulation rate,

desludging frequency = 28750/(0.07 x 25000) = 16 years.

Because of non-uniform deposition of sludge, a desludging frequency of once in 10 years is recommended.

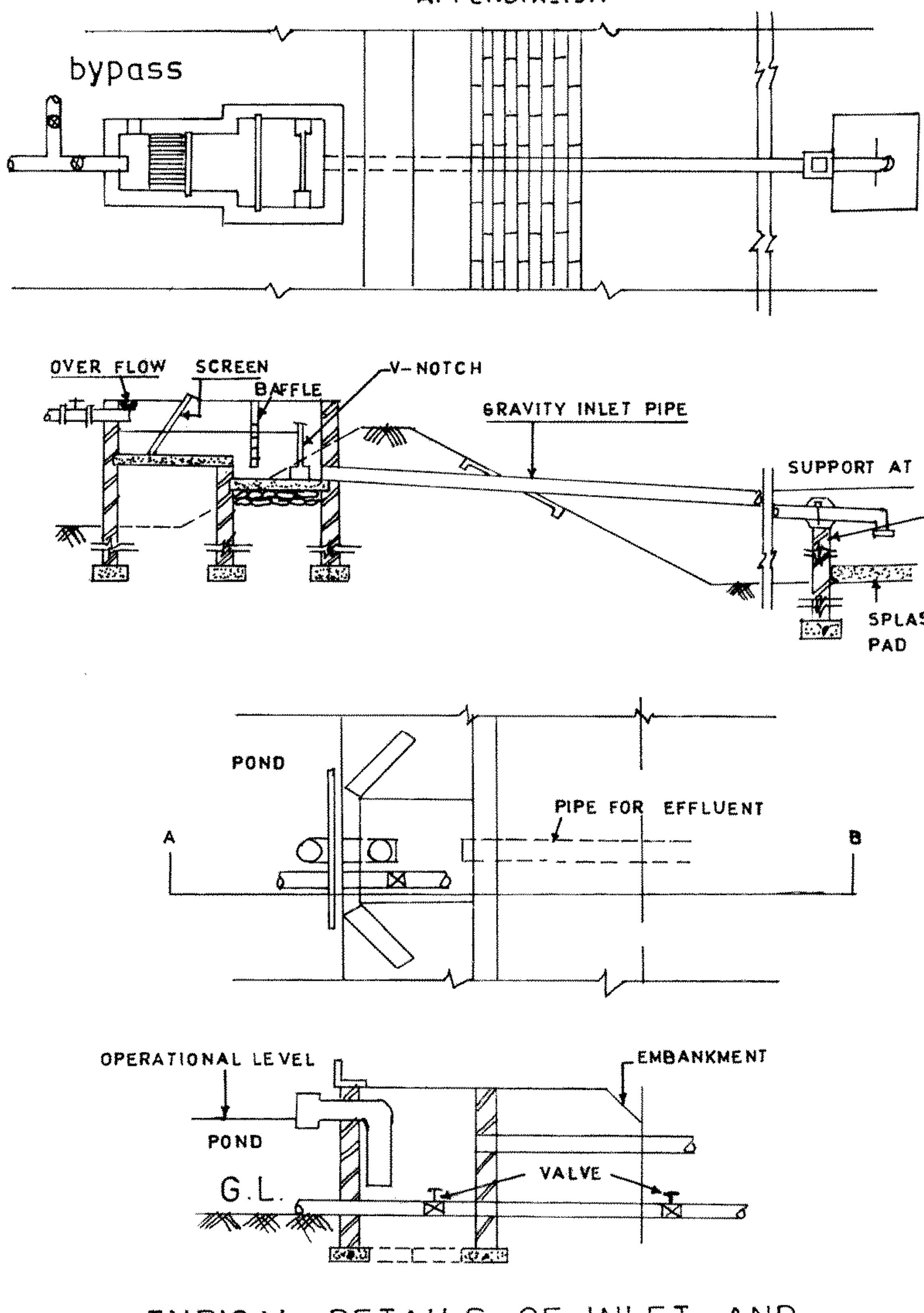
APPENDIX_15.1



a = Top width of the bund
b= Bottom width of the bund

TYPICAL PLAN OF A WASTE STABILIZATION POND

APPENDIX_15.1



TYPICAL DETAILS OF INLET AND OUTLET CHAMBER FOR FACULTATIVE WASTE STABILISATION POND

APPENDIX 16.1

DESIGN EXAMPLE FOR UPFLOW ANAEROBIC SLUDGE BLANKET REACTOR

Problem Statement

Design an upflow sludge blanket reactor for an average flow of 5 MLD of wastewater with the following data:

1.	COD of wastewater	=	400 mg/l
2.	Design hydraulic residence time	=	6 hrs
3.	Design COD loading	=	1 - 2 kg $COD/m^3.d$
4.	Velocity of rise of wastewater in the reactor through sludge bed	=	0.75 m/hr
5.	Velocity of wastewater in settling chamber	=	< 1.5 m/hr
6.	Flow area covered by each inlet	=	1 - 2 m^2

Solution

1 Determine the dimensions of UASBR

Volume of UASBR = 5000 x (6 / 24) = 1,250 m^3

Actual volumetric organic loading = [(5 x 400) / 1250] kg $COD/m^3.d$

= 1.6 [O.K. as it is between 1-2 kg $COD/m^3.d$]

Height of waste water in reactor = Rise velocity x HRT

= 0.75 x 6 = 4.5 m

Area of Reactor = [1250 / 4.5] = 277.8 m^2

Provide two reactors of 11.8 m x 11.8m x 5.25 m (height)

2. No. of Inlets

Assume that each inlet can serve 2.0 m^2 of flow area

Number of inlets in each reactor = [138.9 / 2] = 70

3. Area of Settling Chamber

Assuming a velocity of 1.2 m/hr in the settling zone

Area of settling chamber in each reactor = [5000 / (2 x 24 x 1.2)] = 86.8 m^2

APPENDIX 16.2

DESIGN EXAMPLE FOR ANAEROBIC FILTER

Problem Statement

Design anaerobic filters to treat an average flow of 5 MLD of wastewater with the following assumptions.

1. COD of the wastewater = 400 mg/l
2. Design COD Loading = 1.0 kg COD/m^3.d
3. Depth of media = 1.2 m

Solution

1. Dimensions of anaerobic filter

Total COD load	= 5 x 400	=	2000 kg COD/d
Volume of anaerobic filters for media	= [2000/1.0]	=	2000 m^3
Plan Area of filters	= [2000/1.2]	=	1666.7 m^2

Provide two filters of diameter 32.6 m and height 1.5 m including free board and bottom zone for dispersion of wastewater and supporting media.

2. HRT for filters = [2000 / 5000] d

= 9.6 hrs.

APPENDIX 17.1

DESIGN EXAMPLE OF SLUDGE DIGESTERS

Design low rate and high rate digesters for digesting mixed primary and activated sludge from a 50,000 m^3/day capacity activated sludge Wastewater Treatment Plant.

Given:

From the Appendix 13.1 on the design of activated sludge process:

a)	Raw effluent suspended solids (SS) concentration	=	400 mg/l
b)	SS removal efficiency in the primary sedimentation tank	=	75%
c)	Therefore, quantity of primary sludge generated (0.4 Kg/m^3 x 50,000 m^3/dayx0.75)	=	15,000 Kg/day
d)	At 4%* consistency or 40 Kg/m^3 SS concentration,primary sludge volume (15000 Kgs.day ÷ 40Kg/m^3)	=	375 m^3/day
e)	The excess activated sludge generated	=	2,630 Kg/day
f)	At 1% consistency or SS concentration of 10 Kg/m^3 the excess activated sludge volume (2630 Kgs ÷ 10Kgs/m^3)	=	263 m^3/day
g)	Total volume of the raw mixed sludge (375 + 263)	=	638 m^3/day
h)	Total quantity of the raw mixed sludge (15,000 + 2630)	=	17,630 Kg/day
i)	SS concentration of the raw mixed sludge (17630 Kgs/day ÷ 638m^3/day)	=	27.6 Kg/m^3
j)	The approximate percentage of volatile matters (VM) in the mixed sludge	=	70 %
k)	Quantity of VM in the raw mixed sludge (0.7 x 17630)	=	12,341 Kg/day
l)	Quantity of Non-VM or inorganic (0.3 x 17630)	=	5,289 Kg/day

* 1% consistency = 10,000 mg/l = 10 m^3/kg

Low Rate Digester

a)	Approximate Percentage destruction of VM (design value)	=	50%
b)	For achieving 50 % VM destruction, under mesophilic conditions, the HRT required (from Fig.17.3)	=	40 Days
c)	Quantity of VM in the digested sludge (0.5 x 12,341)	=	6,170 Kg/day
d)	Quantity of nonvolatile matters or inorganic matters in the digested sludge	=	5,289 Kg/day
e)	Total quantity of solids in the digested sludge (6,170 + 5,289)	=	11,459 Kg/day
f)	percentage of VM in the digested sludge (6,170 ÷ 11,459)	=	53.80%
g)	percentage of inorganic matter in the digested sludge (5,289 ÷ 11,459)	=	46.20%
h)	Depending on the frequency of sludge withdrawal the consistency of the digested sludge withdrawn from the low rate digester is expected to be in the range of 4 - 6 %.		
i)	For an average consistency of 5 % (or 50 Kg/m^3), the volume of digested sludge (11,459 ÷ 50)	=	229 m^3/day

j) Therefore the volume of digester

$V = [V_f - 2/3 (V_f - V_d)] T_t$

$= [638 - 2/3 (638 - 229] 40$

$= 14{,}624\ m^3$

Check for volatile solids loading rate Kg VSS/day m^3

$=$ ·12,341 ÷ 14,624 = 0.84 Kg VSS/Day m^3.

(The VSS loading is within the permissible range - 0.6 to 1.6 Kg VSS/Day/m^3)

Gas generation

Gas production per Kg of VM destroved	=	0.9 m^3
Total gas generation (0.9m^3/Kg VM * 6,170 KgVM/day)	=	6,039 m^3
To avoid foaming, the minimum surface area required to meet the condition - 9m^3 of gas generated per Day per m^2 surface area, will be [6039÷9]	=	617 m^2

For operational flexibility and constructional reasons, it is suggested to install two digesters of the following dimensions.

Volume of each digester [14,624 m^3 ÷ 2]	=	7,312 m^3
Minimum surface area of each digester [617 m^2 ÷ 2]	=	309 m^2
Choosing the digester shape as a low, vertical cylinder and for a diameter of 34 m, the surface area of each digester will be	=	908 m^2
Therefore the effective digester depth will be { 7,312 m^3 ÷ 908 m^2 }	=	8.0 M

Additional Volume

Volume for sludge storage during the monsoon period - when the sludge drying bed option is used for sludge dewatering	=	Vd * T2
For a storage period of 12 days {229 m^3/day * 12 days}	=	2,748 m^3
equivalent to 2748 m^3 ÷ 908 m^2	=	3.0 m
Additional allowance for grit and scum accumulation	=	0.6 m
Free board	=	0.6 m
Therefore total additional depth	=	4.2 m

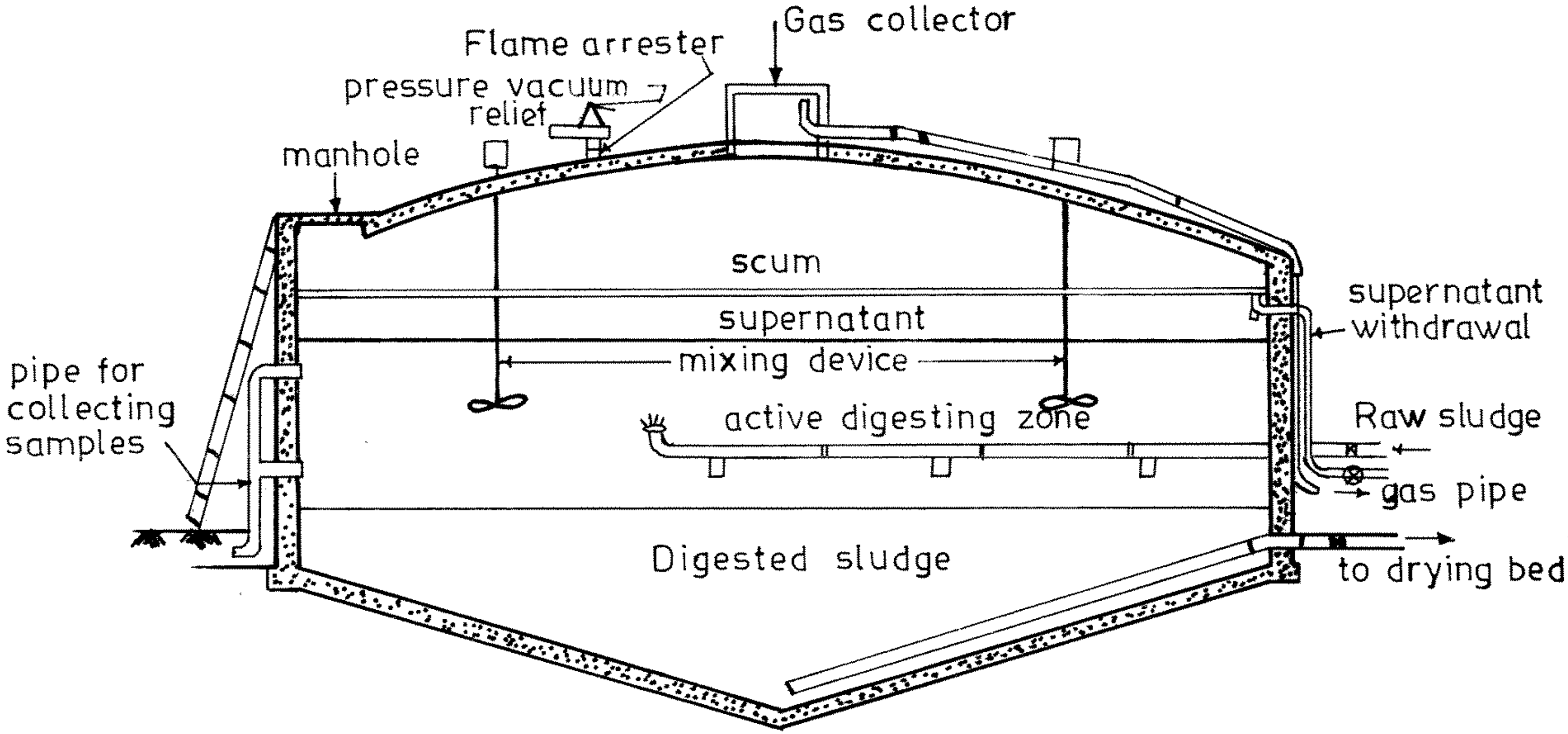
TWO DIGESTERS - EACH OF 34 M DIAMETER & 12.2 M DEPTH

High Rate Digesters

For a sludge temperature of 20° C, the Solids Retention Time (SRT) required for 50% VSS destruction (refer Fig.17.3)	=	20 days
Therefore the digester volume will be	=	638 * 20
(Volume of fresh sludge * Retention time)	=	12760 m^3
Choosing two digesters, the capacity of each digester will be:		
Volume (12760 $m^3 \div 2$)	=	6380 m^3
Choosing a diameter of 27 M, the effective depth will be	=	11.2 M
Additional allowance for grit accumulation	=	0.5 M
Free board	=	0.6 M
Total additional depth	=	1.1 M

Two digesters of 27 M diameter and 12.3 M depth

Additional, separate sludge holding facility for storage during monsoon period (when sludge drying bed option is used for dewatering) is to be computed as before.



SECTION

TYPICAL DETAILS OF LOWRATE SLUDGE DIGESTER

APPENDIX 17.2

DESIGN EXAMPLE OF SLUDGE DRYING BEDS

Problem Statement

Design sludge drying beds for digested sludge obtained from low rate anaerobic digesters for digesting a mixture of primary and excess activated sludge. The capacity of activated sludge plant is 50,000 m^3/d and following data is assumed:

i) Volume of digested sludge (Refer to design example on low rate anaerobic digester in Appendix 17.1) = $229 m^3/d$

ii) Dewatering, drying and sludge removal cycle = 10 d

iii) Depth of application of sludge = 0.3 m

Solution

a) Total plan area of sludge drying = $\frac{229 \times 10}{0.3} m^2$ = $7633 m^2$

b) Number of beds is assumed to be = 30

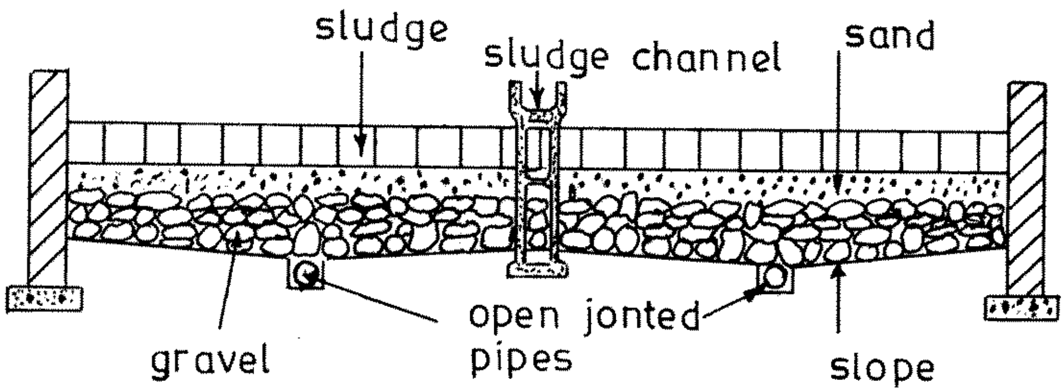
Plan area of each bed $\frac{7633}{30}$ = $254.43 m^2$

c) If per capita wastewater flow is assumed as 150 lpcd

contributory design population = $\frac{50000 \times 10^3}{150}$ = 3,33,333

Plan area of sludge drying bed = $7633 \div 3{,}33{,}333 = 0.023\ m^2/capita$

APPENDIX -17.2



SECTION

TYPICAL DETAILS OF SLUDGE DRYING BED