PROJECT REPORT

WATER TREATMENT AND ITS SUPPLY

NAME – Ayush Prajapati

ROLL NUMBER – 200250

INSTRUCTOR NAME – Dr. Mukesh Sharma

CONTENT

ESTIMATION OF POPULATION

Land available = 100 acres
Built-up area = 30 acres
FAR= 2.5
Allowable floor area = 2.5.(100) = 250 acres = 250.(4046) = 1,011,500 m^2
Average building height = 1011500/{(30).(4046)}= 8 stories
Residential area = (0.75).(1011500) = 758,625 m^2
Population density = 25 m^2 / person
Expected population in 2041 = 758625/25 = 30345

Thus, the expected population in 2041 is 30345.

ESTIMATION OF WATER DEMAND

Average per capita domestic water demand = 235 lpcd
Average domestic water demand = 235.(30345)/10^6= 7.13 MLD

Temporary population = 10000
Average per capita temporary water demand = 60 lpcd
Average temporary water demand = 60.(10000)/106= 0.60 MLD

Commercial water demand = 0.5.(7.13) = 3.57 MLD

Horticultural demand = (0.5/100).(60).(4046)/1000 = 1.21 MLD

Average daily demand = 7.13 + 0.60 + 3.57 + 1.21 = 12.51 MLD

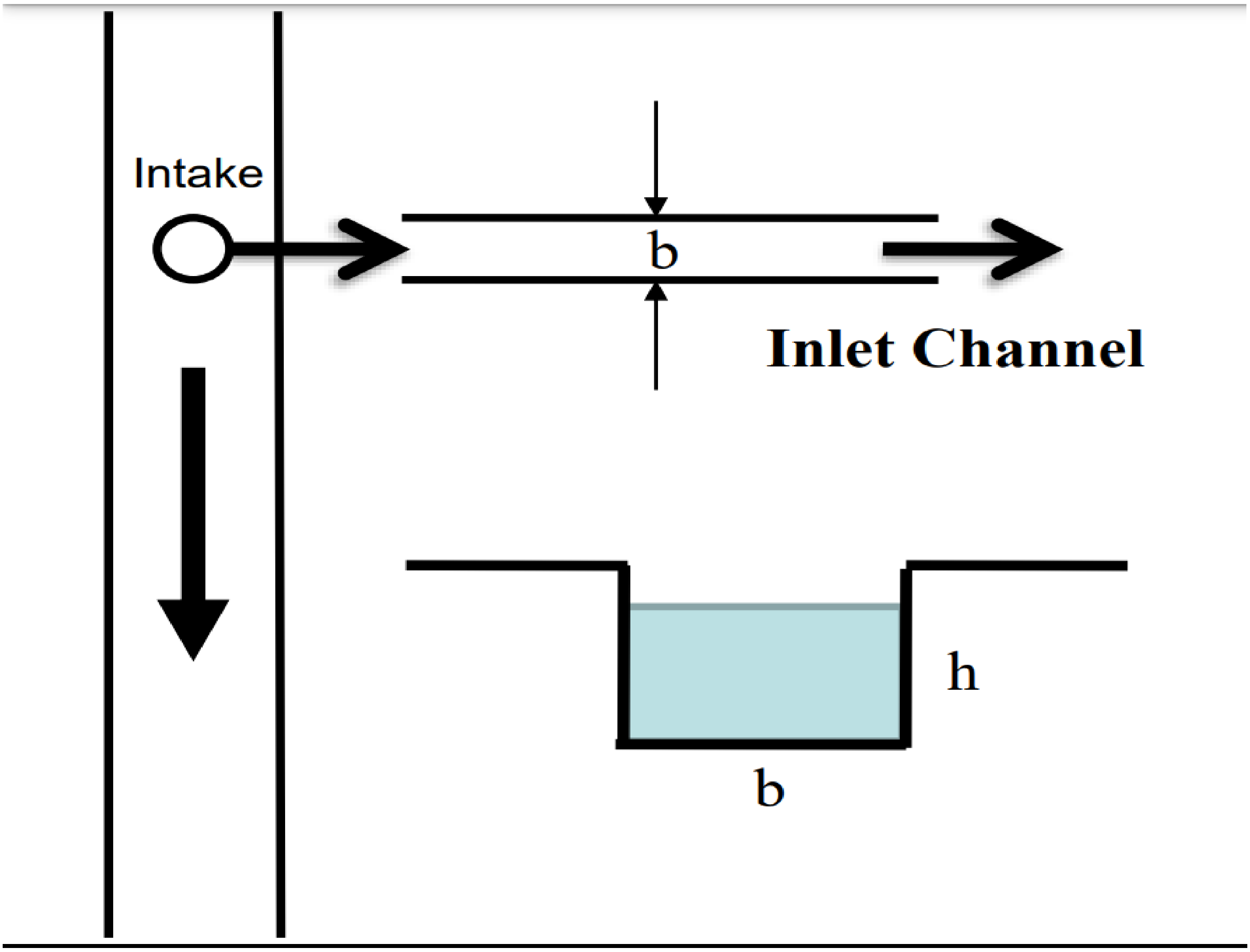
Maximum Daily demand = (1.8).(12.51) = 22.52 MLD
(The water intake pumps and the water treatment plant must be designed for this value)

Peak hourly demand = 3.(12.51) = 37.53 MLD
(the distribution system must be designed for this value)

Fire Demand = (100/1000).(30345/1000)0.5= 0.55 ML
Duration of storage = 6 hours
Total Storage = (37.53 –22.52).(6/24) + 0.55 = 4.30 ML
(provided in underground or overhead tanks)

Thus, the water demand in 2041 is estimated to be 22.52 MLD.

INLET CHANNEL



Q = Design flow
b = Channel width
h = depth of flow
S = channel slope
A = flow cross section
V_H = flow velocity

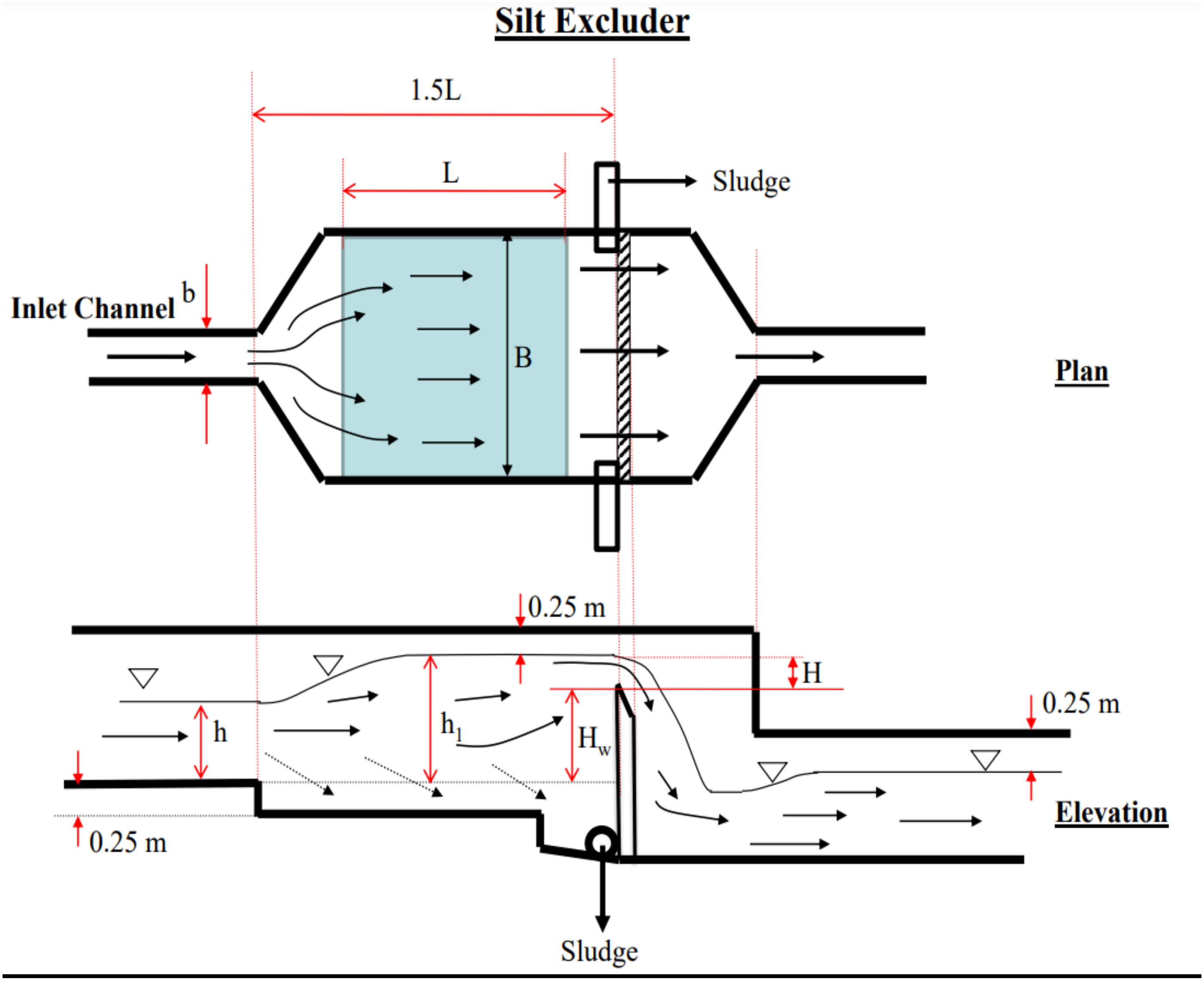
No particle <2.5 mm in diameter must settle in the inlet channel

V_{sc} of 2.5 mm particle = $4.(g.((\rho_s - \rho)/\rho).d)^{0.5}$
$=4.(9.81.((2650-1000)/1000).(0.0025))^{0.5}=0.805 m/s$

DESIGN OF INLET CHANNEL

Q = 22.519 MLD = 0.261 m3/s
Choose: $n = 0.012$; $S = 0.005$; $b = 0.20$ m
Assuming rectangular channel, $R = (b.h)/(b + 2.h)$
$A = b.h$;
$Q = A.V_H = (1/n).A.(R)^{2/3}.(S)^{1/2}$;
Choose h such that Q = 0.261 m3/s; h = 1.09 m
$V_H = Q/(A) = 1.197$ m/s; $V_H > V_{SC}$, hence okay

SILT EXCLUDER



All particles greater than 0.25 mm diameter must settle in the silt excluder

V_{SC} (0.25 mm particles) = 0.254 ms^{-1} ; Thus, $V_H < V_{SC}$

Design of Silt Excluder

Height of Weir (H_W) = 1.1 m

Flow over rectangular weir, $Q = C_w.(2g)^{1/2}.B.H^{3/2}$; $C_w = 0.95$; Choose B = 0.90 m

Height of flow over the weir (H) = 0.168 m

Total Height of flow (h_1) = $H_w + H$ = 1.268 m (should be 1 – 2 m)

$V_H = Q/(h_1 .B)$ = 0.228 m s-1 (< V_{SC}, okay)

V_S (0.25 mm particle) = 0.032 ms^{-1} ; $T = h/V_S$ = 40.03s ; $L = T.V_H$ = 9.14 m

Provide 50% extra length (i.e., TE ~ 60 sec); L_{prov} = 13.71 m

PRIMARY SEDIMENTATION TANK

The objective is to remove particles up to 20 mm size completely

$V_{SC} = 0.072\ ms^{-1}$; $V_S = 3.56 \times 10^{-4}\ ms^{-1}$

For a 20 mm particle, SOR = 30.7584 m3 /m2 /day

A PST is generally designed with SOR of 30-50 m3 /m2 /day

DESIGN CONDITION

L: 20 – 100 m; B: 3 – 25 m; D: 3 – 5 m

L/B: 1.0 – 7.5; L/D: 4.2 – 25.0; T: 2 – 5 hours

DESIGN OF PRIMARY SEDIMENTATION TANK

$Q = 0.261\ m^3s^{-1}$; Design SOR = 30 m3/m2/d ; $A_S = 751.68\ m^2$;

No. of tanks provided = 2

L =25 m; B = 15 m; D =3 m

L/B = 1.667 (okay); L/D = 8.3333 (okay)

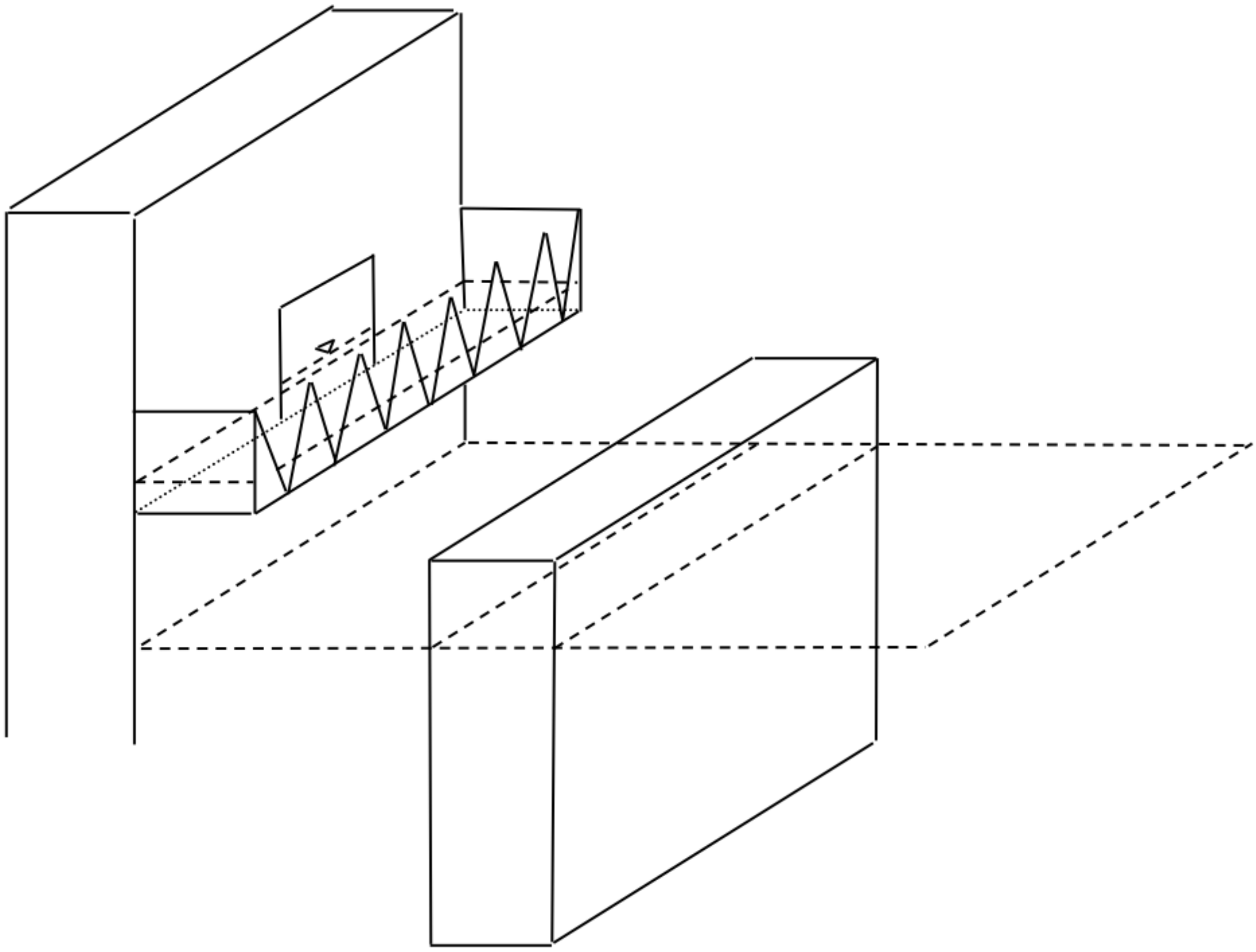
$V = 1125\ m^3$ per tank, total= $2250\ m^3$

T = 2.4 hours (okay); $V_H = 0.0058\ ms^{-1}$ ($< V_{SC}$, okay)

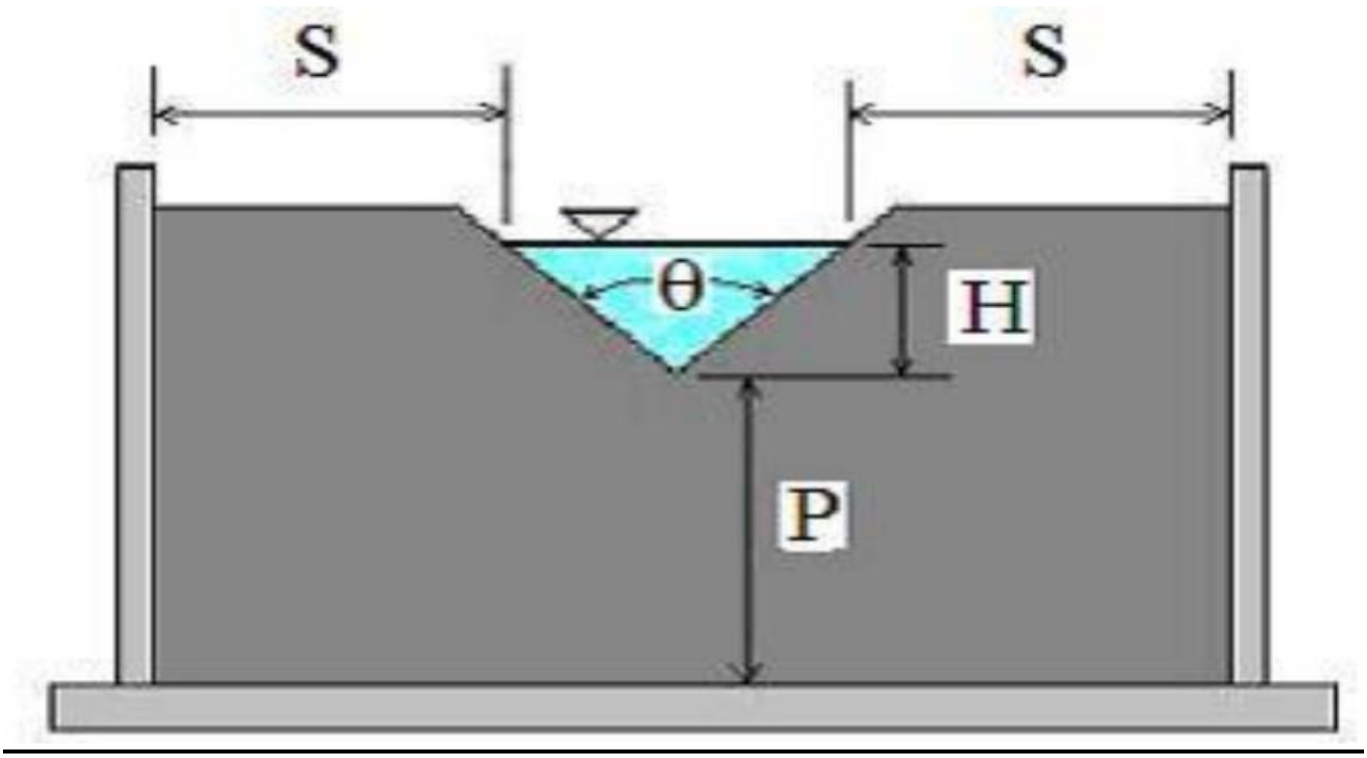
(T is Detention time & V_H is Horizontal velocity)

INLET AND OUTLET (LAUNDER) WEIR

INLET WEIR



V-NOTCH WEIR



$Q = C_e.\tan(\theta/2).(H + k)^{5/2}$, with H in m, θ in degrees and Q in m^3/s

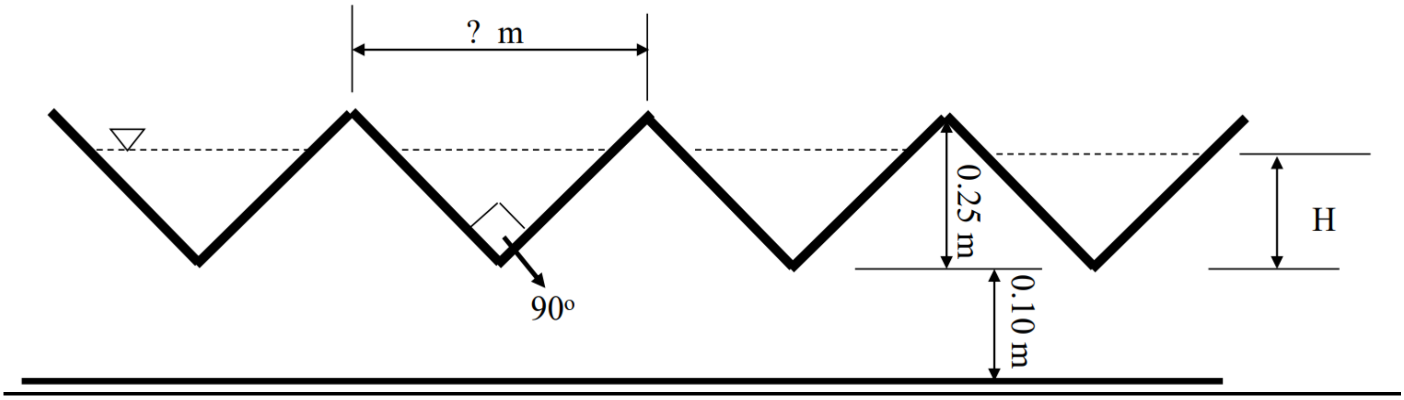
$k = 0.3048[0.0144902648 - (0.00033955535)\theta + (3.29819003 \times 10\text{-}6)\theta^2 - (1.06215442 \times 10\text{-}8)\theta^3]$ m

$C_e = 2.36.[0.607165052 - (0.000874466963)\theta + (6.10393334 \times 10\text{-}6)\theta^2]$

For θ = 90°; k = 4.26 x 10^{-3} m , C_e = 1.43

INFLUENT WEIR DESIGN

Influent Weir Loading Rate = Q/B = 31.32 m3/m/hr



$Q = C_e.\tan(\theta/2).(H + k)^{5/2}$, with H in m, θ in degrees and Q in m3 /s

Weir Length (l) = 30 m; No. of weirs = 60

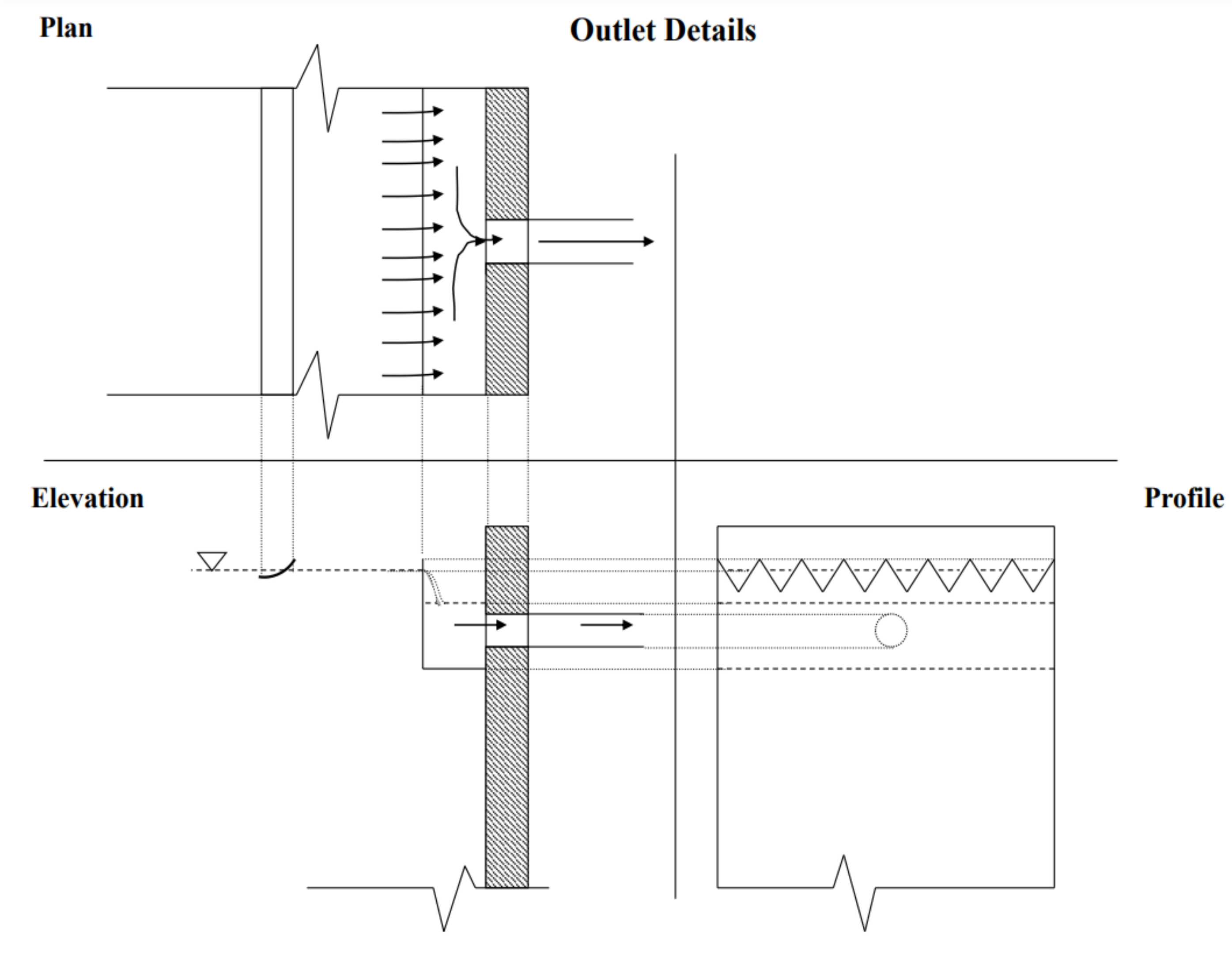
Flow/weir = 0.00435 m^3/s

H = 9.42 cm

Width of Weir (b) = 1 m

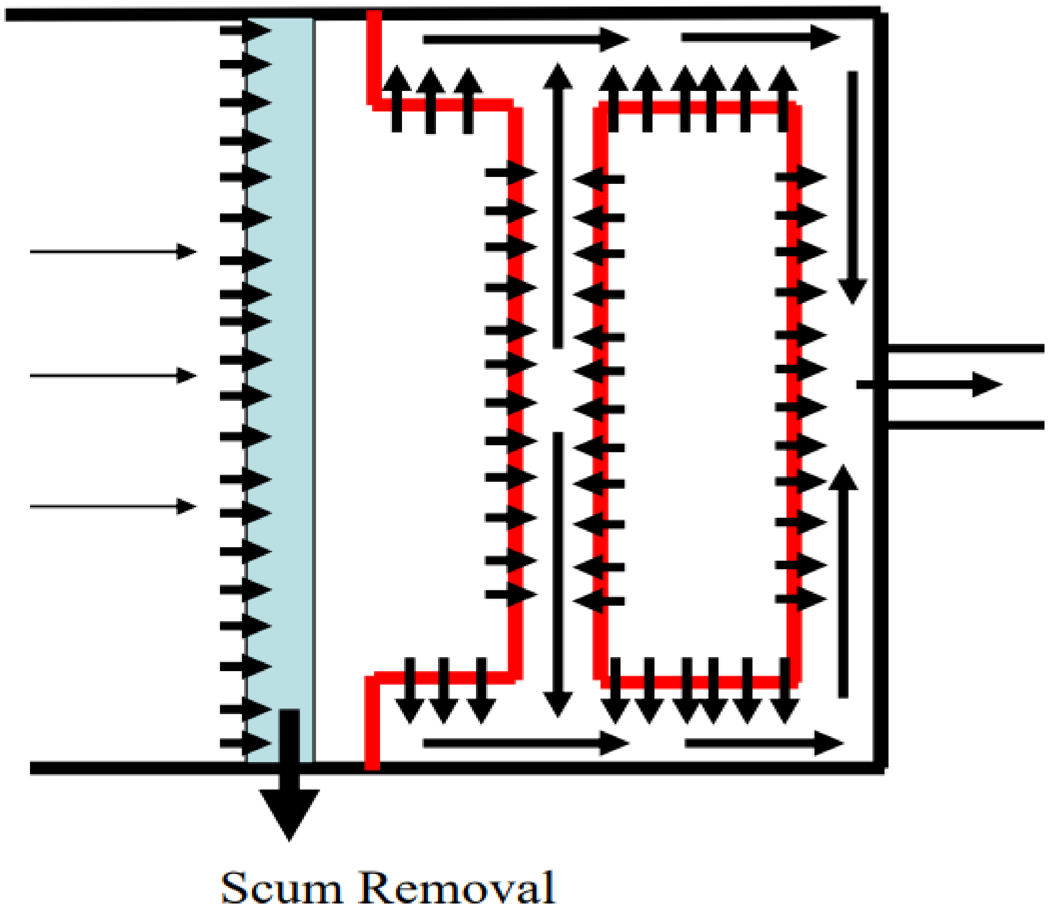
Detention Time = 22.32 s

OUTLET WEIR ORTHOGRAPHIC VIEW



ALTERNATE ARRANGEMENT OF EFFLUENT LAUNDERS

(FOR REDUCING WEIR LOADING RATES)



EFFLUENT WEIR DESIGN

Acceptable effluent weir loading rate in PST: 10 – 15 $m^3/m/hr$

Effluent weir loading rate = 12.50 $m^3/m/h$

Weir Length required (l) = 75.168 m

No. of weirs = 150

Flow / weir = 0.00174 m^3/s

H = 6.4 cm

Distance between bottom of V-notch to floor of the Effluent Launder = 10 cm

Width of weir (b) = 1 m

Detention Time = 47.23 s

PRIMARY SLUDGE PRODUCTION

Q (Flowrate):		22.52	MLD
Q:		0.261	m^3/s
Influent Suspended Solids Concentration:		**750.00**	**mg/L**
Effluent Suspended Solids Comcentration:		**150.00**	**mg/L**
Mass of Solids in Sludge (daily basis):		13511	kg/d
Density of Solids in Sludge:		**2650**	**kg/m^3**
Volume of Solids in Sludge (daily basis):		5.10	m^3/d
Percent Solids in Sludge (w/w):		**4.00**	**percent**
Mass of Sludge (daily basis):		337781	kg/d
Mass of Water in Sludge (daily basis):		324270	kg/d
Density of Water:		**1000**	**kg/m^3**
Volume of Water in Sludge (daily basis):		324.27	m^3
Total Mass of Sludge (daily basis):		337781	kg/d
Total Volume of Sludge (daily basis) [QP]:		0.33	MLD
Density of Sludge:		1025.54	kg/m^3

Mass of sludge solids (dry basis): (750 – 150).22.52 = 13511 kg/day

Mass of sludge = (13511/4).100 = 337781 kg/day

Weight of water in sludge = (337781 – 13511) = 324270 kg/day

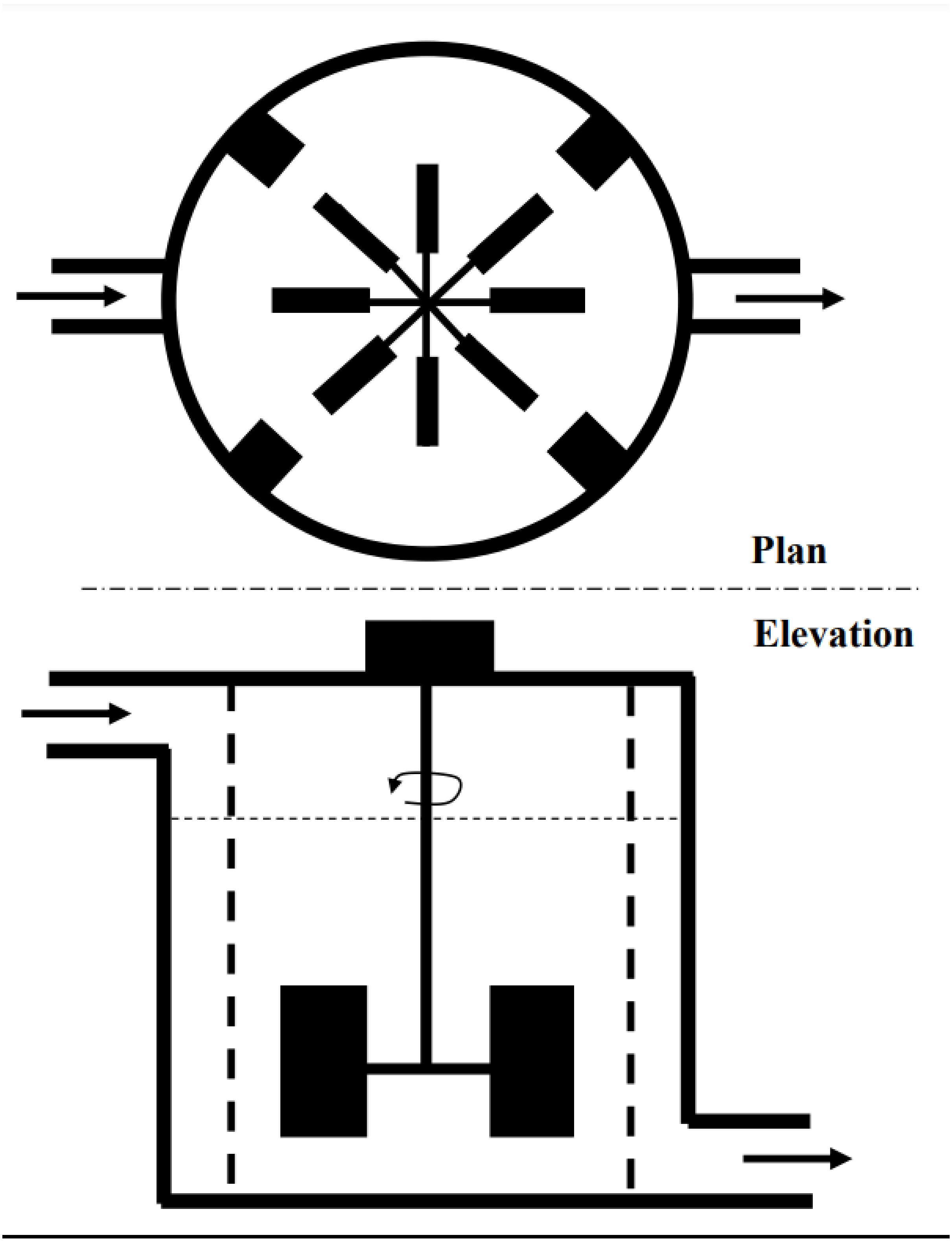
Volume of water in sludge = (324270/1000) = 324.27 m^3

Volume of sludge (dry basis) = 13511/2650 = 5.10 m^3

Total volume of sludge = 324.27 + 5.10 = 329.37 m^3 = 0.33 MLD

Density of sludge = 337781/329.37 = 1025.53 kg/m^3

RAPID MIX TANK



DESIGN OF RAPID MIX TANK

Q = 11.75 MLD = 0.14 m^3/s

Let t = 40 s (between 20 – 60 s)

V = t.Q = 5.44 m3 ; Let D = 2 m; A_{CS} = 3.14 m2

H = V/A_{CS} = 1.73 m; Freeboard = 0.5 m; H_T = 2.25 m

H/D = 0.865 (in the range 0.33 – 1.0, hence okay)

Let, G = 400 s^{-1} (> 300 s^{-1} , hence okay);

Gt = 16000 (in the range 10000 – 20000, hence okay)

Let the blade tip speed (v_p) = 1.8 m/s (in the range, 1.75 – 2.00 m/s, hence okay)

Velocity of the blade relative to water (v) = 0.75.(1.8) = 1.35 m/s

$G=\{(C_D.A_P.v^3)/(2.\nu.V)\}^{1/2}$; A_p = 0.393 m2

Tank sectional area (A_T)= D.H = 3.46 m2

A_p / A_T = 0.393 / 3.46 = 0.113 (between 0.10 – 0.20, hence okay)

Let the impeller diameter be 0.8 m, i.e.,

(D_I/D = 0.4) ,(is it between 0.2 – 0.4 ?)

Choose length of each impeller blade (L) as 0.20m i.e.,

(L/D_I = 0.25) ,(up to 0.25 ?)

Choose breadth of each impeller blade (B) as 0.15m, i.e.,

(B/D_I = 0.1875) ,(up to 0.20 ?)

Area of each blade = 0.03 m^2

Blade thickness = 5 cm

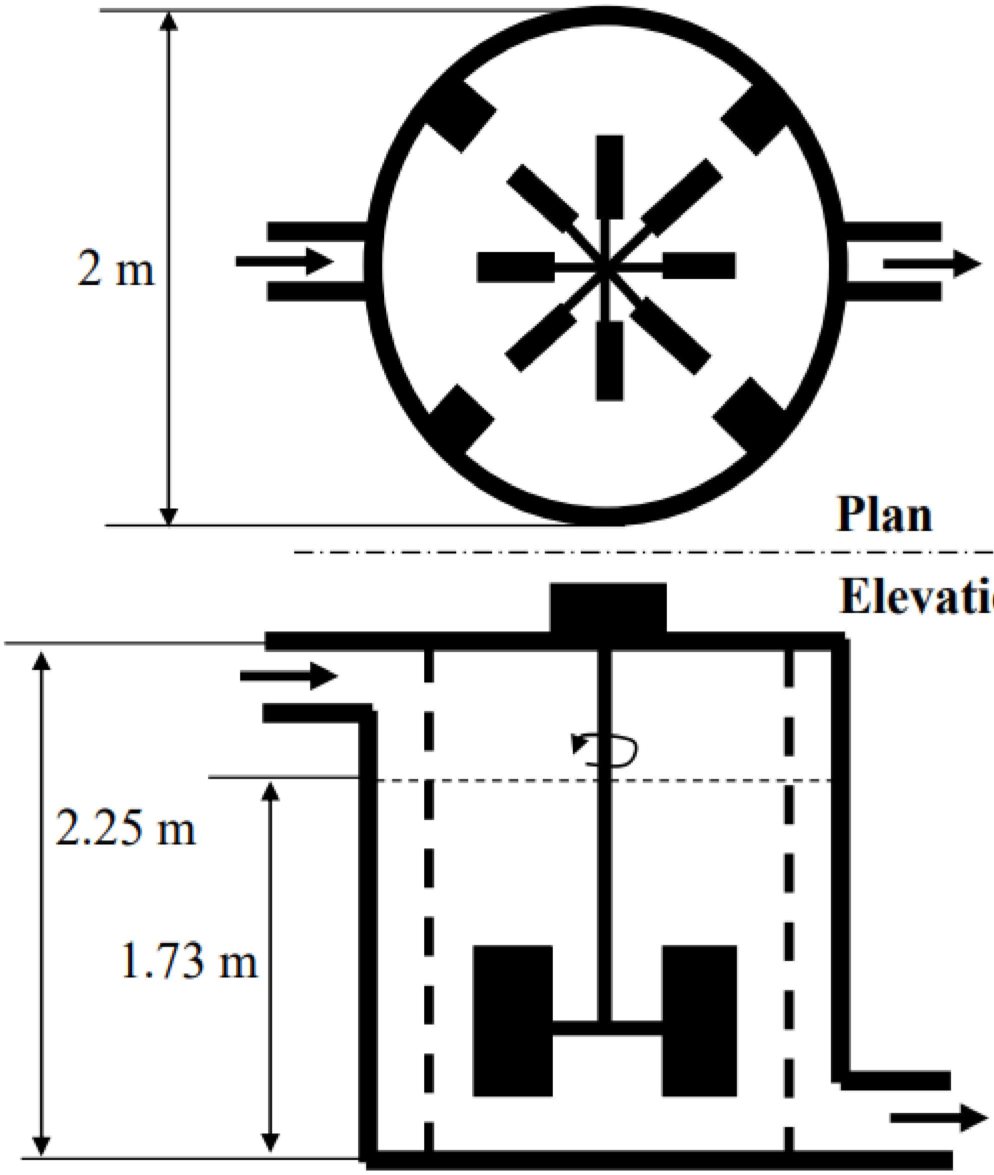
Therefore, number of blades to be provided = 13.1 (say 14)

Clearance of the paddles from the tank bottom = 0.8 m

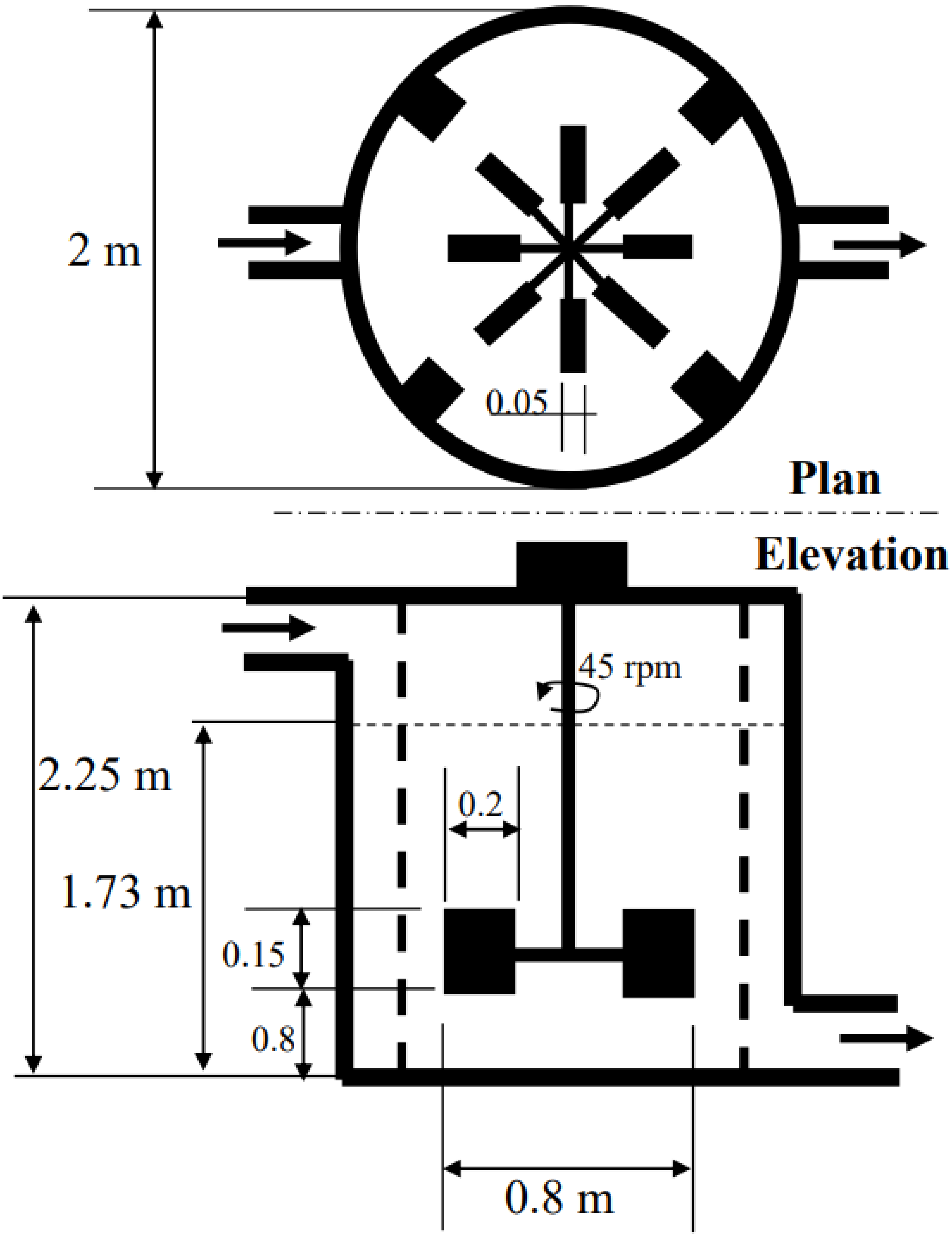
Paddle rotation speed (w) = $2.v_p / D_l$ = 4.5 radians/s, (~ 45 revs/min)

Power requirement (P): $G = \{P/(V.\mu)\}^{1/2}$ P = 0.872 KW

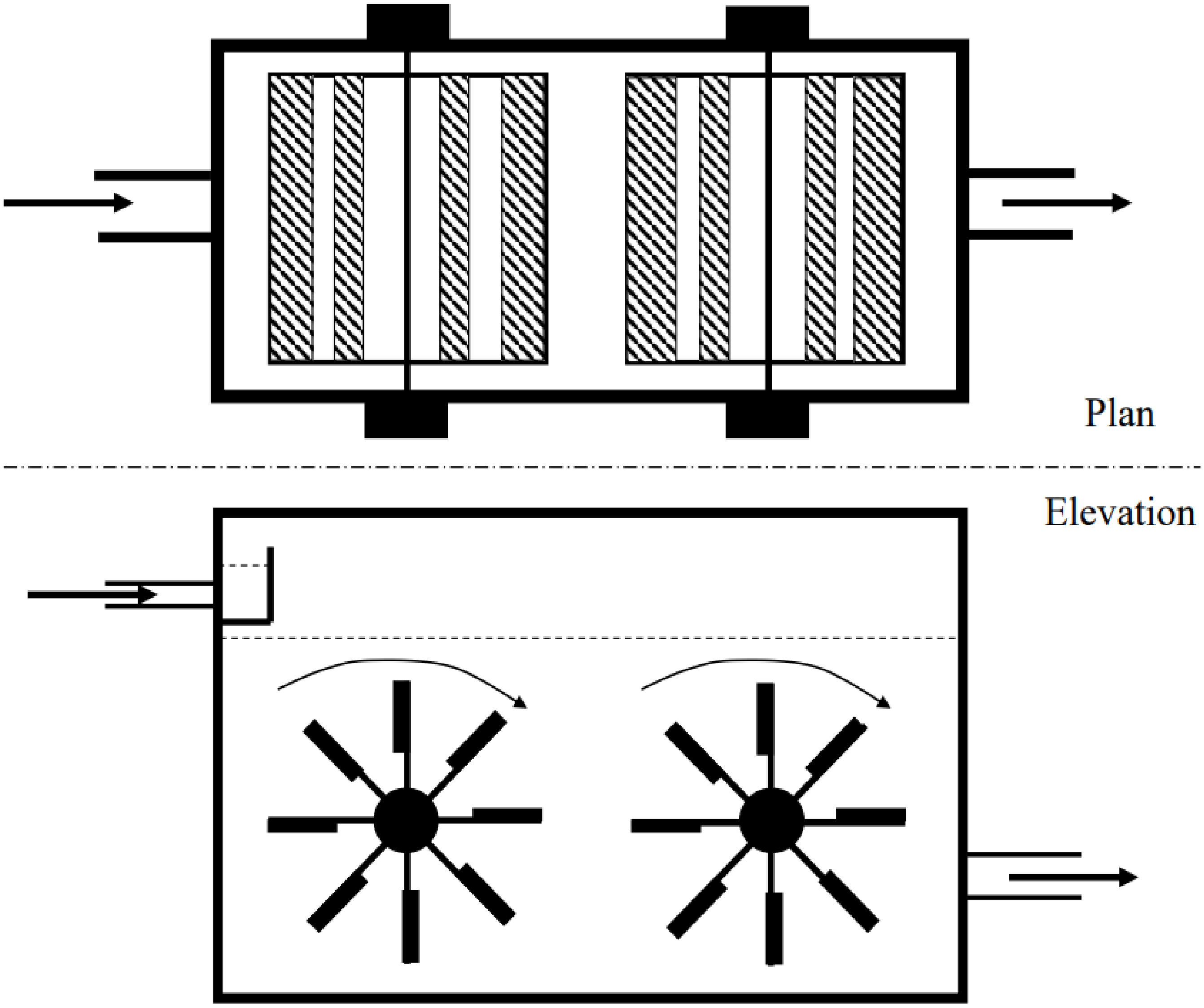
Provide 1 kW motor



RAPID MIX TANK DIMENSION



FLOCCULATION TANK



DESIGN OF FLOCCULATION TANK

Let the detention time (t) be 20 minutes (within 10-30 minutes, okay)

Let the velocity gradient (G) be 30 /s (within 20-75 /s, okay)

Therefore G.t = 36000 (within 20000 – 60000, Yes)

Volume of the tank (V) = 313.2 m^3

Let the two paddle shafts be placed width-wise

Let the paddle diameter (D_P) be 5 m (within 2-5 m, okay)

Depth of the tank (D) = Paddle diameter + submergence + bottom clearance = 6 m (Submergence and bottom clearance are taken as 50 cm)

Surface Area of the tank (A) = 52.2 m^2

Length of the tank (L) = 2.(end clearance) + 2.(paddle diameter) + 0.25 = 10.75 m (End clearance is taken as 25 cm)

Tank width (W) = 4.8558 m

Length to width ratio = 2.214 (Range: 2:1 to 6:1, okay)

Paddle length (Lp) = tank width -2.(End clearance) = 4.3558 m (within 2-5 m ?)

Let the paddle tip speed (v_p) be 0.40 m/s (within 0.25 – 0.75 m/s, okay)

Then the velocity of paddle relative to water (v) = 0.30 m/s

$G=\{(C_D.A_P.v^3)/(2.\nu.V)\}^{1/2}$; Total paddle area (A_p) = 11.6 m^2

Paddle area per shaft = 5.8 m^2

Tank sectional area = (W.D) = 29.135 m^2

Paddle area per shaft /Tank Sectional Area (D.L) = 0.09

Let four paddles be provided per shaft.

Area of each paddle = 1.45 m^2

Breadth of each paddle (B_P) = 0.333 m (Range: 22 - 50 cm, okay)

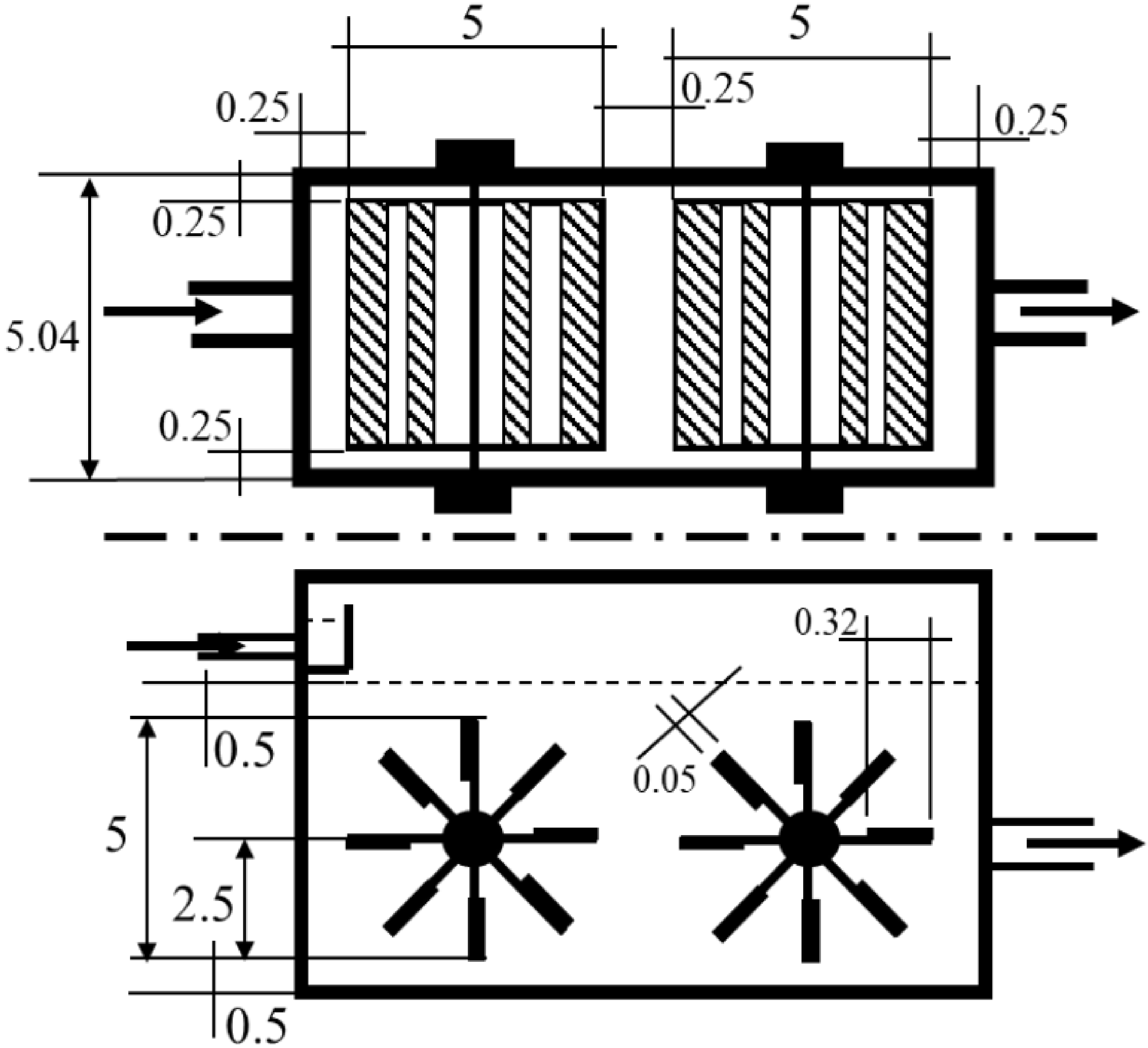
Paddle thickness = 5 cm

Paddle rotation speed (w) = $2.v_P/D_P$ = 0.16 radians/s (or, 1.53 revolutions per minute)

Power requirement is given by, $G = \{P/(V.\mu)\}^{1/2}$

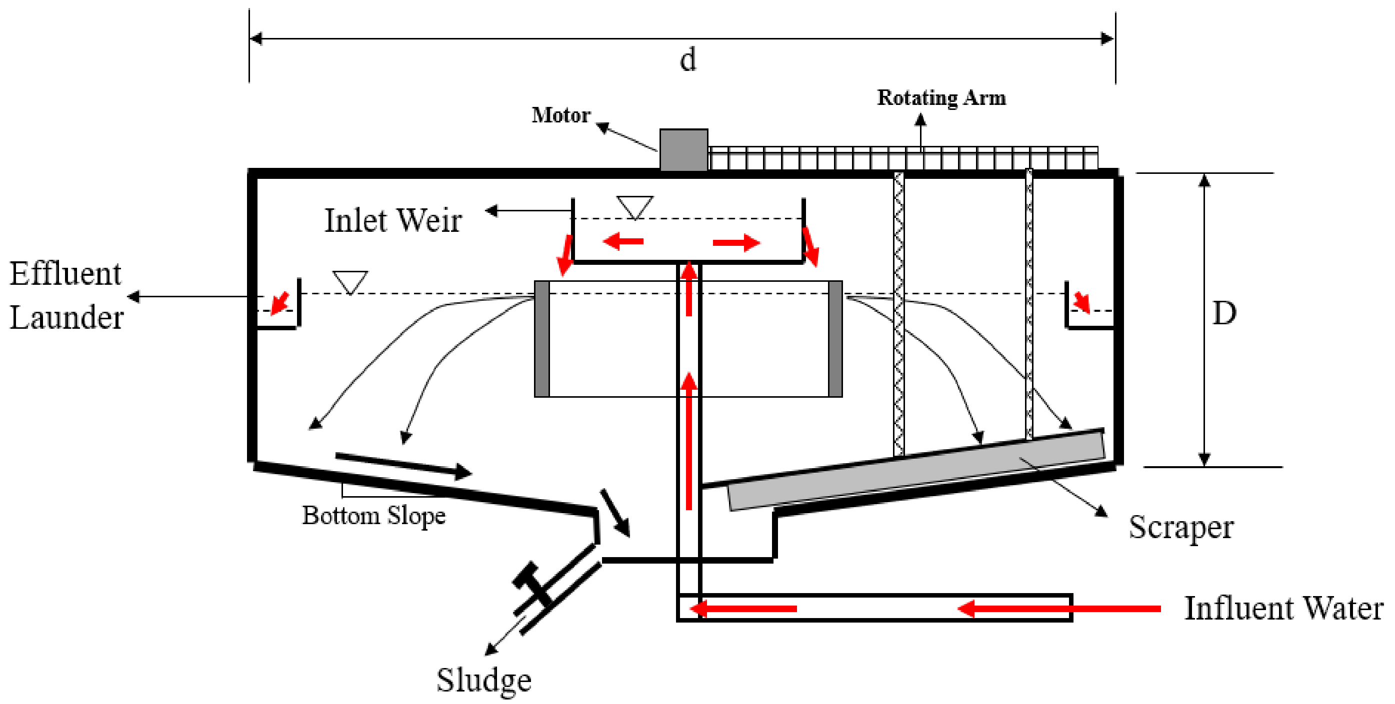
P = 282.44 Watts

FLOCCULATION TANK DIMENSIONS



SECONDARY SEDIMENTATION TANK

Secondary sedimentation tanks are generally circular



Design Parameters: -

SOR: 30-40 $m^3/m^2/day$

Depth: 3- 4.5 m

Dia.: 3-60 m

Bottom slope(cm/m): 6.3 – 17

Detention Time: 2-5 hours

DESIGN OF SECONDARY SEDIMENTATION TANK

Q = 0.261 m3/s

Assuming SOR to be 30 $m^3/m^2/d$

Then As = 751.68 m^2 ~ 752 m2

No. of tanks provided = 2

Area of each tank = 752/2 = 376 m^2

Diameter d = $(376*4/3.14)^{0.5}$ = 22 m; D = 4 m

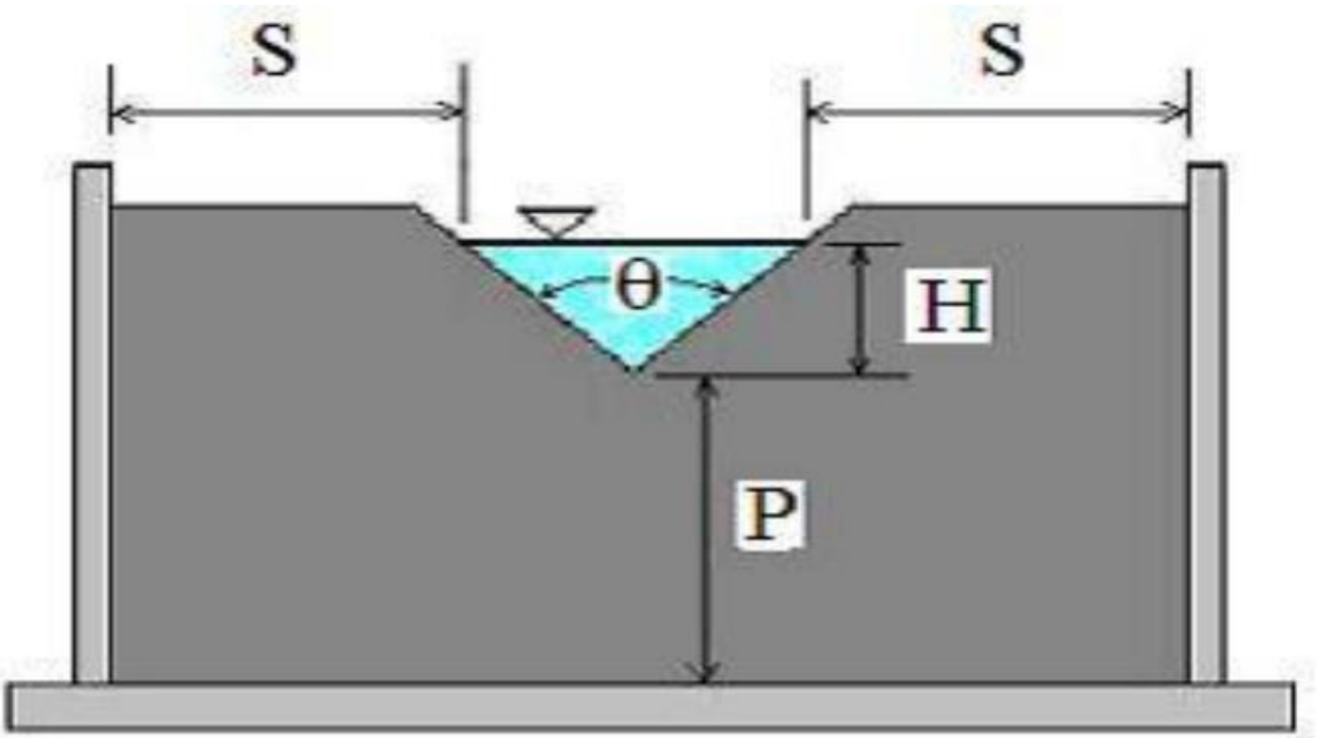
So, V = 1520 m^3 per tank, total volume = 3040 m^3

T = V/Q = 3040/(0.261*3600) = 3.24 hrs, (T in 2-5 hrs. hence okay) .

So we have design the secondary sedimentation tank with diameter 22 m and depth 4m

INLET AND OUTLET (LAUNDER) WEIR

V-NOTCH WEIR



$Q = C_e.\tan(\theta/2).(H + k)^{5/2}$, with H in m, θ in degrees and Q in m^3/s

$k = 0.3048[0.0144902648 - (0.00033955535)\theta + (3.29819003 \times 10\text{-}6)\theta^2 - (1.06215442 \times 10\text{-}8)\theta^3]$ m

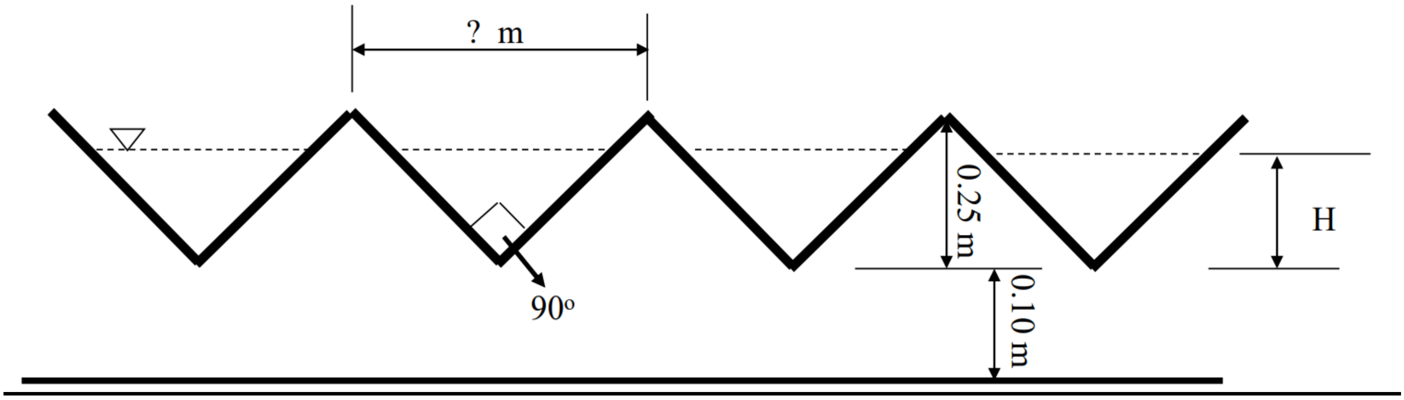
$C_e = 2.36.[0.607165052 – (0.000874466963)\theta + (6.10393334 \times 10\text{-}6)\theta^2]$

For θ = 90°; $k = 4.26 \times 10^{-3}$ m , $C_e = 1.43$

INFLUENT WEIR DESIGN

Assume inlet weir length = 30m

Influent Weir Loading Rate = Q/B = 31.32 m^3/m/hr



$Q = C_e.\tan(\theta/2).(H + k)^{5/2}$, with H in m, θ in degrees and Q in m3 /s

Weir Length (l) = 30 m; No. of weirs = 60

Flow/weir = 0.00435 m^3/s

H = 9.42 cm

Width of Weir (b) = 1 m

Detention Time = 22.32 s

EFFLUENT WEIR DESIGN

Acceptable effluent weir loading rate in SST: 10 – 15 m^3/m/hr

Effluent weir loading rate = 12.50 m^3/m/h

Weir Length required (l) = 75.168 m ~ 75 m

No. of weirs = 150

Flow / weir = 0.00174 m^3/s

H = 6.4 cm

Distance between bottom of V-notch to floor of the Effluent Launder = 10 cm

Width of weir (b) = 1 m

Detention Time = 47.23 s

SECONDARY SLUDGE PRODUCTION

For 22.52 MLD capacity water treatment plant, the effluent turbidity from primary sedimentation tank is 80 NTU. This corresponds to a suspended solids concentration of 150 mg/L. 30 mg/L of alum [(Al)2(SO4)3.18H2O] and 10 mg/L soda is added to this water for coagulation and pH adjustment respectively. After flocculation and secondary sedimentation, turbidity of the water is 10 NTU, which corresponds to a suspended solids concentration of 20 mg/L. Assume that the alum added is fully converted to equivalent amount of aluminum hydroxide flocs, and removed during secondary sedimentation. Calculation for the volume of sludge produced per day in the SST, assuming that the sludge has 3 percent solids concentration is presented in the table below. Specific gravity of aluminum hydroxide flocs is 1.2 and that of other inorganic particles is 2.65. (Atomic Weights; Al: 27, S: 32, O: 16, H: 1)

Q:		22.52	MLD		Calculated
Q:		0.261	cum / s		Calculated
Influent Suspended Solids Concentration:		**150.00**	**mg/L**		**Given**
Effluent Suspended Solids Comcentration:		**20.00**	**mg/L**		**Given**
Mass of Suspended Solids Solids Removed:		2927.44	kg/d		Calculated
Alum Dose		**30**	**mg/L**		**Given**
Alum Added:		676	kg/d		Calculated
Molecular Wt. of Alum		**666**			**Given**
Alum Added:		1014	moles/d		Calculated
Aluminum Hydroxide produced:		2029	moles/d		Calculated
Molecular Wt. of Aluminum Hydroxide		**78**			**Given**
Aluminum Hydroxide produced:		158	kg/d		Calculated
Total Mass of Solids Removed		3086	kg/d		Calculated
Solids Content of Secondary Sludge		**3**	**percent**		**Given**
Total Mass of Sludge		102856	kg/d		Calculated
Weight of Water in Sludge		99770	kg/d		Calculated
Density of Water:		**1000**	**kg/m3**		**Given**
Volume of Water in Sludge		99.77	m3/d		Calculated
Density of Aluminum Hydroxide		**1200**	**kg/m3**		**Given**
Volume of Aluminum Hydroxide Sludge		0.13	m3/d		Calculated
Density of Suspended Solids		**2650**	**kg/m3**		**Given**
Volume of Suspended Solids		1.10	m3/d		Calculated
Total Sludge Volume		101.01	m3/d		Calculated
Density of Sludge:		1018.31	kg/m3		Calculated

RAPID SAND FILTER DESIGN

22.52 MLD of water after secondary sedimentation (Turbidity: < 10 NTU) is to be filtered through a battery of rapid sand filters to reduce water turbidity to < 1.0 NTU. Based on pilot plant studies, it was determined that 60 cm deep filter beds of sand (0.5 mm average sand diameter) were suitable for this purpose. It was further determined that such beds could be safely operated up to 7.5 hours at a filtration rate of 10 m3 /m2 /hr, without the terminal head-loss reaching 3 m. Filter backwashing rate was 1 m3 /m2 /min and the backwash time was 5 minutes. A filter unit will be off-line for 30 minutes during each backwash operation. Based on this information, calculation for the numbers of filter units to be provided and dimensions of each unit is shown below. Amount of filtered water required for backwashing each day is also calculated.

Note that the total water to be filtered is more than 22.52 MLD, since filter backwash (say, B m^3/d) is cycled back to the head for the flocculation tank and filtered again.

So, total water to be filtered = (22520 + B) m^3/d

Three filter backwash cycles/day. Hence off-time due to backwashing = 1.5 hrs/day

(22520 + B) m^3 of water must be filtered in 22.5 hours at a filtration rate of 10 m3 /m2 /hr

Therefore, (22520 + B) = (22.5).(10).A; B = 1.(A).5.(3)

Therefore, 22520 + 15.A = 22.5.(10).A; A = 107.23 m^2

B = 15.A = 15.(107.23) = 1608.45 m^3 /d

Hence total water produced = 22520 + 1608.45 = 24128.45 m^3 in 22.5 hrs using 107.23 m^2 filter area

Provide 8 filters (in parallel), each with cross-sectional area of 107.23/8 = 13.40 m^2

L = 4.25 m; B = 3.15 m L/B = 1.35 (okay)

24128.45 m^3 filtered in 22.5 hours 1608.45 m^3 filtered in 1.50 hours = 90 minutes

Water (for consumption) actually produced by a filter for 22.5 – 1.5 = 21 hours

In 21 hours of useful operation per day, the filter produced 22520 m^3 of water

The filter produces further 1608.45 m^3 of water in 1.5 hours of operation to be used for backwashing

Further, the filter is off-line (producing no water) for 1.5 hours a day for backwashing

FILTER RUN SCHEDULE

	1st Cycle		2nd Cycle		3rd Cycle	
	Start	Stop	Start	Stop	Start	Stop
Filter 1	00:00	07:30	08:00	15:30	16:00	23:30
Filter 2	01:00	08:30	09:00	16:30	17:00	00:30
Filter 3	02:00	09:30	10:00	17:30	18:00	01:30
Filter 4	03:00	10:30	11:00	18:30	19:00	02:30
Filter 5	04:00	11:30	12:00	19:30	20:00	03:30
Filter 6	05:00	12:30	13:00	20:30	21:00	04:30
Filter 7	06:00	13:30	14:00	21:30	22:00	05:30
Filter 8	07:00	14:30	15:00	22:30	23:00	06:30

DAILY OPERATING SCHEDULE OF FILTER 1

	Backwash Water Production Period	Volume of Backwash Water Produced (m^3)	Filtered Water Production Period	Volume of Filtered Water Produced (m^3)	Backwash Period
Cycle 1	00:00 – 00:30	67	00:30 – 07:30	938	07:30 – 08:00
Cycle 2	08:00 – 08:30	67	08:30 – 15:30	938	15:30 – 16:00
Cycle 3	16:00 – 16:30	67	16:30 – 23:30	938	23:30 – 00:00
	Total:	**201**	**Total:**	**2814**	

BACKWASH TANK INPUT/OUTPUTS

Time Period	Back wash water production		Backwashing	
	Filter	Time	Filter	Time
00:00 – 01:00	F1 – C1	00:00 – 00:30	F2 – C3	00:30 – 01:00
01:00 – 02:00	F2 – C1	01:00 – 01:30	F3 – C3	01:30 – 02:00
02:00 – 03:00	F3 – C1	02:00 – 02:30	F4 – C3	02:30 – 03:00
03:00 – 04:00	F4 – C1	03:00 – 03:30	F5 – C3	03:30 – 04:00
04:00 – 05:00	F5 – C1	04:00 – 04:30	F6 – C3	04:40 – 05:00
05:00 – 06:00	F6 – C1	05:00 – 05:30	F7 – C3	05:30 – 06:00
06:00 – 07:00	F7 – C1	06:00 – 06:30	F8 – C3	06:30 – 07:00
07:00 – 08:00	F8 – C1	07:00 – 07:30	F1 – C1	07:30 – 08:00
08:00 – 09:00	F1 – C2	08:00 – 08:30	F2 – C1	08:30 – 09:00
09:00 – 10:00	F2 – C2	09:00 – 09:30	F3 – C1	09:30 – 10:00
10:00 – 11:00	F3 – C2	10:00 – 10:30	F4 – C1	10:30 – 11:00
11:00 – 12:00	F4 – C2	11:00 – 11:30	F5 – C1	11:30 – 12:00
12:00 – 13:00	F5 – C2	12:00 – 12:30	F6 – C1	12:30 – 13:00
13:00 – 14:00	F6 – C2	13:00 – 13:30	F7 – C1	13:30 – 14:00
14:00 – 15:00	F7 – C2	14:00 – 14:30	F8 – C1	14:30 – 15:00
15:00 – 16:00	F8 – C2	15:00 – 15:30	F1 – C2	15:30 – 16:00
16:00 – 17:00	F1 – C3	16:00 – 16:30	F2 – C2	16:30 – 17:00
17:00 – 18:00	F2 – C3	17:00 – 17:30	F3 – C2	17:30 – 18:00
18:00 – 19:00	F3 – C3	18:00 – 18:30	F4 – C2	18:30 – 19:00
19:00 – 20:00	F4 – C3	19:00 – 19:30	F5 – C2	19:30 – 20:00
20:00 – 21:00	F5 – C3	20:00 – 20:30	F6 – C2	20:30 – 21:00
21:00 – 22:00	F6 – C3	21:00 – 21:30	F7 – C2	21:30 – 22:00
22:00 – 23:00	F7 – C3	22:00 – 22:30	F8 – C2	22:30 – 23:00
23:00 – 00:00	F8 – C3	23:00 – 23:30	F1 – C3	23:30 – 00:00
	Total backwash water production: 1608 m^3 / d (67.0 m^3 in each hour)		Total backwash water utilization: 1608 m^3 (67 m^3 in each hour)	

SUMMARY OF OPERATIONAL DETAILS

Eight filters in operation in parallel

Of these, one filter is employed for producing backwash water in the first half-hour of every hour

Amount of backwash water produced as above = 67 m^3

This water is stored in a backwash tank of 70 m^3 capacity

The backwash water produced as above is used to backwash a filter in the second half hour of every hour

Thus, water is effectively being produced by seven filters every hour

Water production per hour = 134 m^3 / filter x 7 filters = 938 m^3

Water production per day = 22.52 MLD

DISINFECTION THROUGH CHLORINATION

Following results are used:

1. 1 mg/L chlorine is required to completely oxidize 1 mg/L BOD5.
2. 2.0-Log removal of Giardia cysts occur during conventional treatment.
3. “Ct” for 2-Log kill of Giardia cysts at pH 7 using free chlorine is 39 mg/L-min.
4. OCl^- is totally ineffective in killing microorganisms.

Instantaneous chlorine demand

1 mg/L chlorine (as Cl_2) is required per 1 mg/L

BOD_5 = 3 mg/L

Hence instantaneous chlorine demand = 3 mg/L chlorine as Cl_2

Breakpoint chlorination demand

NH_4^+ concentration in water to be oxidized is 2 mg/L (as N)

Thus, NH_4^+ concentration in water = (2/14000) = 1.428 x 10-4 moles/L

The relevant equation: $3HOCl + 2NH_4^+ \rightarrow 3Cl^- + N2 + 3H_2O+5H^+$

Therefore, amount of free chlorine required for ammonia oxidation

= $(3/2).(1.428 \times 10^{-4}) = 2.143 \times 10^{-4}$ moles/L = 15.21 mg/L (as Cl_2)

Chlorine Demand (up to breakpoint) = 15.21 + 3 = 18.21 mg/L (as Cl_2)

Required free chlorine residual = 2 mg/L (as Cl_2)

Therefore, total chlorine requirement = 3 + 15.21 + 2 = 20.21 mg/L (as Cl_2)

Disinfection objective: 5-Log kill of Giardia cysts required

(2-Log kill achieved through conventional treatment (‘CT’ credit)

Therefore, additional: 3-Log kill is required through disinfection

Fraction of HOCl in free chlorine =([H^+]/([K]+[H^+]))

Therefore, fraction of HOCl at pH 7 = $F_{7.0}$ = (10^{-7}/(10^{-7}+$10^{-7.5}$)) = 0.759

Fraction of HOCl at pH 7.5 = $F_{7.5}$ = ($10^{-7.5}$/($10^{-7.5}$+$10^{-7.5}$)) = 0.5

Using simplified Chick-Watson Law,

At pH 7.0, 2 = (λ.$F_{7.0}$).(C.t); Ct = 39; i.e., λ = 6.76×10^{-2}

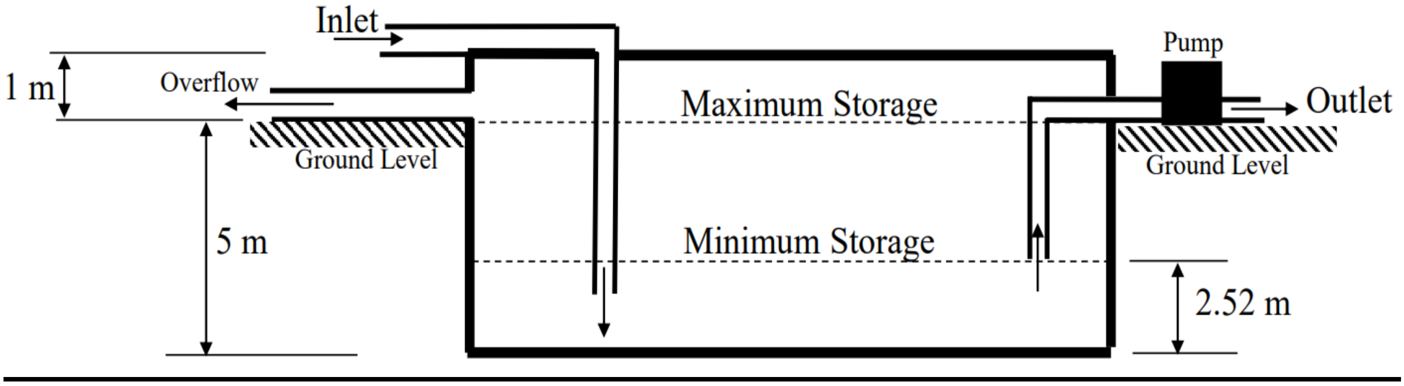
At pH 7.5, 3 = (6.76×10^{-2}).$F_{7.5}$.(C.t); i.e., Ct = 39;

Therefore, required contact time = 89/2 = 44.5 minutes for obtaining a total 5-Log removal.

(2-Log removal by conventional treatment + 3-Log kill by disinfection) of Giardia Cysts.

Chlorine Requirement: (20.21 mg/L).(22520 m^3/day).(1000 L/m^3).(10^{-6} kg/mg) = 455 kg/day

UNDERGROUND STORAGE TANK



Water demand = 22.52 MLD

Peak water demand = 37.53 MLD

Total storage capacity (in UG and OH tanks combined) = 4.30 ML

Minimum detention time in UG tank = t_{min} = 44.4 minutes = V_{min}/Q_{max}

$V_{min} = t_{min} \cdot Q_{max}$ = (44.4/60)(37.53/24) = 1.16 ML

Let maximum storage (V_{max}) = 2.30 ML = 2300 m3

Let maximum depth of storage (d_{max}) = 5 m; Freeboard = 1 m

Surface area = 2300/5 = 460 m^2 ; Length = 23 m; Breadth = 20 m

Minimum depth of storage (d_{min}) = 1160/460 = 2.52 m