TABLE III

TABLE SHOWING COMPARATIVE STATEMENT OF OVERALL COST STRUCTURE OF PUMPING MAIN `L` FOR DIFFERENT PIPE SIZE

Sl. No.	Pipe size in mm	Total head in m		Class of CI pipes required	Rate* per m length	Cost of 7000 m pipe line	1st stage flow of 7.5 MLD				2nd stage flow of 10 MLD				Grand total of capitalized cost for 30 years (16) = (11) + (15)
		1st stage	2nd stage												
1	2	3	4	5	6	7	8*	9*	10*	11*	12*	13*	14*	15*	16*
1	300	115	165	A	640.77	4486	520	1192	9067	14073	660	2391	18186	4512	18585
2	350	80	105	LA	804.92	5635	360	829	6306	12301	420	1522	11577	2872	15173
3	400	75	80	LA	918.72	6431	340	777	5910	12681	320	1160	8823	2189	14870
4	450	60	65	LA	1106.51	7746	280	622	4731	12757	260	942	7165	1778	14535
5	500	55	60	LA	1302.66	9119	260	570	4336	13715	240	870	6618	1642	15357

8* Cost of pumpsets

9* Annual cost of energy charges = 7001.65 KW_1 = CR_1

10* Energy charges capitalised = C_{C1} = 7.606 CR_1

11* Total capitalised cost = (7) + (8) + (10)

12* Cost of pump sets

13* Annual cost of energy charges = 7348.63 KW_2 = CR_2

14* Energy charges capitalised = C_{C2} = 7.606 CR_2

15* Initial capital investment for pumpsets and annual electrical charges = (Col. 12 + Col. 14) / 4.177

* Cost of pipe includes cost of specials (10% of actual pipe cost), of excavation and cost of laying and jointing

REMARKS: From this table it is seen that most economical sizes of main is 450 mm `LA` Class C.I. pipe

APPENDIX 6.6

DESIGN OF THRUST BLOCKS

To design a thrust block for 900mm diameter main conveying water at $11 kgs/cm^2$ pressure(P).

The deviation angle α is 45° and density of concrete is $2300\ kgs/m^3$. Soil density is assumed to be $1800\ kgs/m^3$ and angle of internal friction $\phi = 30°$.

Assume minimum cover of earth is 600 mm. Cohesion is 0 for sandy soils.

HORIZONTAL THRUST : $F = 2\ pA \sin \alpha/2$

Cross sectional area $A = (\pi/4)(90)^2 = 6364$ sq. cms.

$\sin \alpha/2 = 0.382$

$F = 2\ (11)\ (6364)\ (0.382)\ (10^{-3}) = 53.48$ Tonnes

(i) Lateral resistance to counteract the horizontal thrust:

Try a thrust block of size = 3.2M × 3.2 M × 3.2 M

Weight of thrust block = 3.2M × 3.2 M × 3.2 M × 2.3	= 75.36 tonnes
Weight of water in the pipe = $0.785 \times (0.9)^2 \times 1 \times 3.2$	= 2.03 tonnes
Weight of earth = $0.9 \times (3.2)\ (0.6)\ (1.8)$	= 3.11 tonnes
Total Weight	80.50 tonnes

Total force available considering frictional resistance of soil = 80.5 (0.3) = **24.15 tonnes**

(ii) Lateral resistance of soil against the block:

$$fp = \gamma_s \frac{(H)^2}{2} . L . \frac{1 + \sin\theta}{1 - \sin\theta} + 2CHL \sqrt{\frac{1 + \sin\theta}{1 - \sin\theta}}$$

By assuming cohesion as 0, the above equation

$$\text{Yields} = 1.8\ \frac{(3.2)^2}{2}\ (3.2)\ \frac{(1.5)}{0.5} = 88.47\ \textit{tonnes}$$

(ii) Lateral resistance of soil when the thrust block is free to yield away from the soil mass i.e., the portion of projected pipes :-

$$fa = \gamma_s \, h \, \frac{1 - Sin\ \theta}{1 + Sin\ \theta} - 2C\sqrt{\frac{1 - Sin\ \theta}{1 + Sin\ \theta}}$$

$$= (1.8)(0.9)\,\frac{(0.5)}{1.5} = 0.54\ tonnes$$

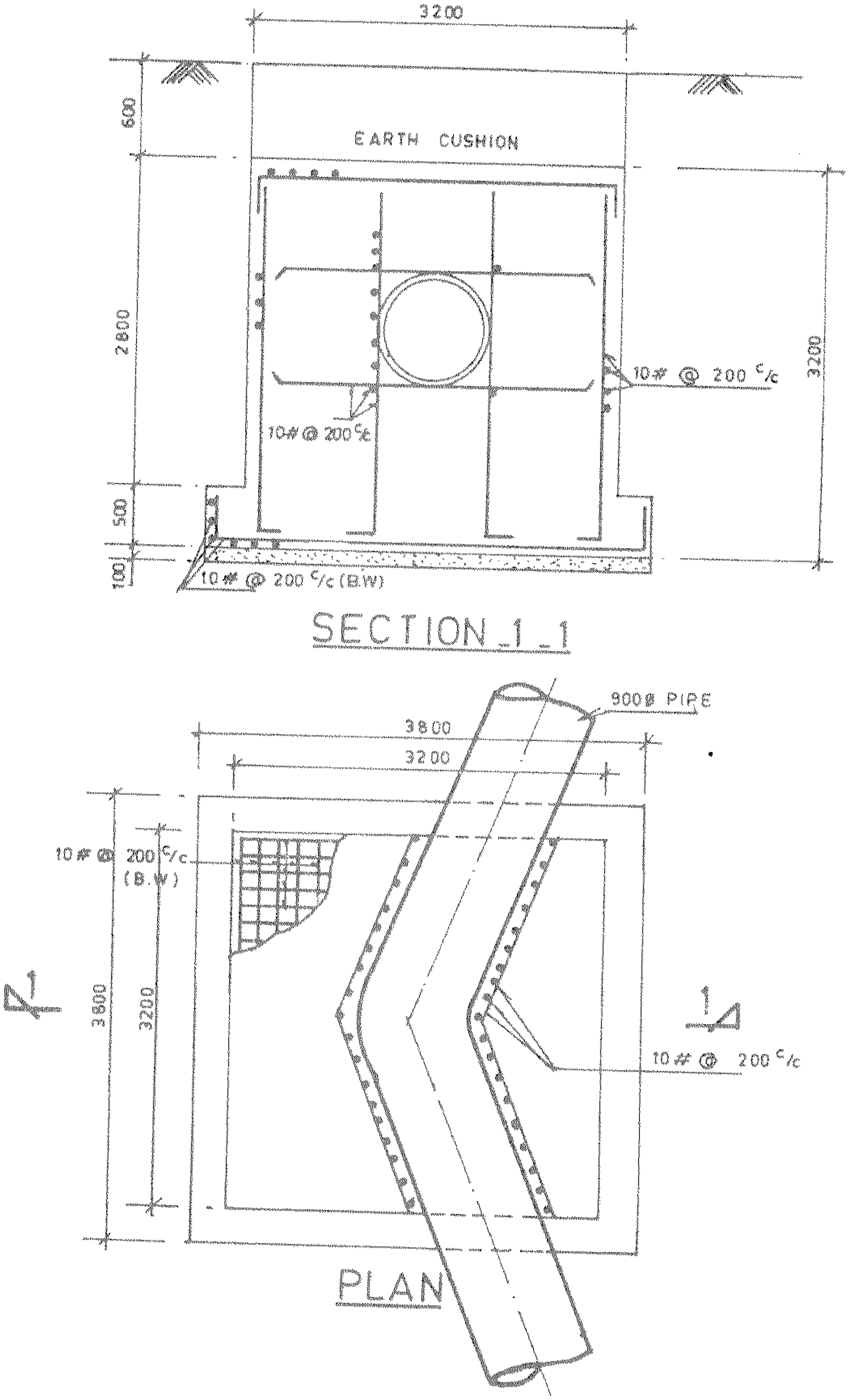
Total lateral resistance = 24.15 + 88.47 + 0.54 = **113.16 tonnes/m^2**

Total horizontal thrust = **53.48 tonnes**

Factor of safety = 113.16/53.48 = 2.19 which is O.K.

REINFORCEMENT:

The minimum surface reinforcement in all thrust blocks shall be 5 kgs/sq.m(as per IRG 21-1972 Article 306.4). The spacing of these bars is not to exceed 500mm. Hence provide 10 bars at 200 c/c which is more than 5 kgs/sqm.



THRUST BLOCK FOR 45° HORIZONTAL BEND

APPENDIX 6.7

DESIGN OF AIR VESSEL

DATA

Discharge through pipe line	= 1.944 cumecs
Material of pipe line	= steel
Diameter of pipe line	= 1550 mm
Thickness of pipe (ct)	= 10 mm
Static Head	= 120 m
Length of pumping main	= 18000 m
Total Head including friction Head & other losses of 15 meter.	= 135 m
Design head of pumps.	= 150 m
Atmospheric Head	= 10 m
H_o = Absolute Head = Total head + Atmospheric Head	= 135 + 10 = 145 m

V_0 = Initial velocity $= \dfrac{1.994}{(\pi/4)(1.55)^2} = 1.03\ m/\sec$

C = Water Hammer Wave velocity.

$$= \frac{1425}{\sqrt{1+\dfrac{kd}{Ec_t}}}$$ (Refer 6.17.2 of Manual)

$$= \frac{1425}{\sqrt{1+\left[\left(2.07\times10^{8}\right) / \left(2.1\times10^{10}\times0.01\right)\right]}} = 896.27\ m/s$$

Water Hammer head $= \dfrac{cV_0}{g} = \dfrac{896.27\times1.03}{9.81}$
= 94.10 meters

Pipeline Parameter $\rho = \dfrac{cV_0}{2gH_0}$

$$= \frac{896.27\times1.03}{2\times9.81\times145}$$
= 0.324

2ρ = 0.65

AIR VESSEL PARAMETER = $2C_oC/Q_oL$

Referring Chart f_o, k_c as 0.50 for limiting Upsurge to 1.20 H_o and Downsurge to 0.5 H_o Air vessel parameter for $2\rho = 0.65$ is calculated as follows:

From chart for $2\rho = 1.00x(2C_oC/Q_oL)=10.50$
For $2\rho = 0.5$, $2C_oC/Q_oL = 6.50$

By interpolation for $2\rho = 0.65$ and for $k_c = 0.5$

Air vessel parameter $2C_oC/Q_oL = 7.70$

$$\text{Volume of air } C_o = \frac{7.7 \times 1.944 \times 18000}{2 \times 896.27}$$

$$= 150.31 \text{ Cubic meters}$$

$$\text{Volume of Air Vessel} = C_o[H_o/H_{min}]^{1/1.20}$$
$$= 150.31[(145)/0.5x(145)^{1/1.20}]$$
$$= 150.31x(2)^{1/1.2}$$
$$= 267.20 \text{ Cum}$$

Increase the capacity by 20% to cater for upsurge of 1.20 H_o

$$= 267.20 \times 1.2$$
$$= 320 \text{ Cums}$$

WATER COLUMN SEPERATION LENGTH

The water column separation is calculated on the basis of the following formula.

$$V_1^2 - V_2^2 = (2g/L)\{(t_1-t_2)\, V_i\, \{H+F\,(V_1^2/V_o)\}\}$$

H= Static Head, (Absolute Head)

F= Loss of head due to friction

V_1, V_2=Velocities at instances $t_1$and t_2.

$(t_1 - t_2)$ = Period between time intervals in seconds .

V_o = Initial Velocity.

L=Length of pipeline

Initial velocity will come to rest over a time period after the stoppage of pumps. Assuming a time interval of 0.20 seconds and by using above formula the subsequent velocities are calculated till the final velocity (V_n) is almost Zero. The water column separation length l is given by Laws =

$$l_{wcs}\ \Sigma\ [V_1+V_2+\ldots\ldots V_n]\ (t_2-t_1)$$

For the given diameter of pipe and for the calculated water column separation Length the volume of water required to be stored in Air vessel is calculated.

For Worked Example

$$(1.03)^2 - V_2^2 = 2\times\frac{9.81}{18000}(0.20)(1.03)\left[145+15\frac{(1.03)^2}{1.03}\right]$$

$$(1.01)^2 - V_3^2 = 2\times\frac{9.81}{18000}(0.20)(1.01)\left[145+15\frac{(1.03)^2}{1.01}\right]$$

Repeat n times till $V_n = 0.01$ m/sec.

Then $V_1+V_2+V_3+\ldots\ldots\ldots\ldots V_n$ (0.20)=say =6.1 meters.

For a pipe of 1.55 per dia volume of water required to fill this separation length

$$= \frac{\pi}{4}(1.55)^2(6.10) \quad = \quad 11.51 \text{ Cum}$$

Fixing The Size Of Vessel And Levels Of Water And Air In Airvessel Chamber

(i) Air And Water Volume

Air Vessel volume required = 320 Cum.

If two vessels are provided volume of each vessel = 160 Cum.

Provide 90 Cum of Air and 70 Cum of water in each vessel.

(ii) Determination Of Size Of Air Vessel

Absolute Head at working head of pumps = 150 + 10.35 = 160.35 meters.

Maximum upsurge permitted 160.35 x1.2 = 192.42 meters

Pressure = 19.25 kg/ cm^2

Using 25 mm thick M S Plate i. e 22 mm +3mm for corrosion allowance

$$d = \frac{2f_t \times e \times t}{p}$$

f_t = Permissible tensile strength in steel plates = 1260 kgs/cm^2

e = Weld efficiency say 0.9

t = Thickness in cms of plate = 2.2 cm

p = Pressure in kgs/cm^2

$$= \frac{2 \times 1260 \times 1.90 \times 2.20}{19.25}$$

$= 259.20$ cms

$=$ Say 260 cms

Provide 2.60-m dia of vessel with a length L and two hemi-spherical ends.

Volume of (two hemispheres) spherical portion $= \frac{4}{3} \times \pi(1.3)^3 = 9.2$ Cum.

Total Volume of cylinder = 160 cum – 9.20 = 150.80 cum

Length of vessel of 2.6 m dia with volume 150.80 cum is = 28.40 meters

Provide 2 vessels each of 2.6-m dia and 28.40 m long with hemi-spherical ends.

(iii) Fixing Of Levels Of Water And Air In The Vessel

The levels are fixed by trial by assuming a depth and calculating volume in cylindrical and spherical portions.

(a) Normal Working Level

Volume of Air = 90 cum

Volume of Water = 70 cum

The normal working level is fixed by trial by assuming 1.15 meter of water depth from bottom. Volume of water = 70.95 Cum which is more than required 70 Cum. Hence normal working level will be at 1.15 m from bottom of vessel.

(b) Upper Emergency Level

Air dissolves in water in the vessel. Assuming that 10% Air dissolves in water the level of water rises by 10% of volume of Air i.e.

Volume of water = 70 Cum +10% of 90 Cum = 79 Cum.

The depth of water from bottom will be 1.35 m which gives volume of water as 79 Cum. Hence upper emergency level will be 1.35 m from bottom of vessel.

(c) Lower Emergency Level

When pumps trip as per water column separation about 11.51 Cum of water is required to fill the pipeline. As calculated volume of water at a depth of 1.00 m from bottom of vessel = 56.43 Cum . Volume of water at normal working level is 71 Cum.

Quantity of water available is the difference between normal working level and lower emergency level.

APPENDIX 7.1

DESIGN OF SPRAY TYPE AERATOR

(Removal of Iron & Manganese)

I. PROBLEM STATEMENT

Design a spray aerator given the following data:

1. Design flow = 250 m^3/hr.

   Pipe used = 70 mm dia 'B' Class C.I. Pipe with a C value of 100

   S = 3.60 meters /1000 meters

   V = 1.35 m/s

2. Iron present in raw water = 1.8 mg/l.
3. Saturation concentration of O_2 at 28 ^0C = 7.92 mg/l.
4. Aeration constant (to the common base) at 28 ^0C = 70 cm/hour.

II. DESIGN CRITERIA

1. Nozzle dia. usually 10 to 40 mm – spaced in the pipe at intervals of 0.5 to 1.0 m
2. Nozzles are usually tilted 3^0 to 5^0 to the vertical to avoid interference due to falling water.
3. Nozzle discharges should be uniform as far as possible. Variation in no case should be greater than 5% i.e. the discharge ratio between the first and the last nozzle, should not be less than 0.95(a variation 2 to 5% may be allowed).
4. Velocity of water in the aerator pipe should be between 1 and 1.5 m/s .
5. Pressure required at the nozzle varies from 2 to 9 meter of water (usually 7m).
6. Discharge ratings per nozzle vary from 300 to 600 lpm.
7. Aerator area should be 1.25×10^{-3} to 3.75×10^{-3} m^2 per m^3/day of design flow.

III. SOLUTION

1. Design flow = 6000 m^3/day.
2. Assuming 25-mm dia. nozzle with an inclination of 3^0 to the vertical, dia of one drop is 25 mm.
3. Iron present in raw water = 1.8 mg/l.

   Permissible limit of iron in treated water = 0.1 mg/l.

   Iron to be removed = (1.8-0.1) mg/l = 1.7 mg/l.

4. $Fe^{++}+3O_2 = 2Fe_2O_3$

   1.7 mg/l of Fe requires 1.7 x96/224 = 0.7286 mg/l of O_2.

5. By applying 'Gas absorption equation in 7.2.2 in the form

$$Log_{10}[(C_s - C_0)/(C_s - C_t)] = \frac{K.At}{V}$$

where

C_s = 7.92 mg/l at 28°C, Co = 0.0 mg/l.

C_t = 0.73 mg/l, K = 70 cm/hr

$$\frac{A}{V} = \frac{6}{d} = \frac{6}{2.5}\left(\frac{1}{cm}\right)$$

$$\therefore \log_{10}\frac{7.92}{7.19} = \frac{70}{60x60} \times \frac{6}{2.5} \times t = \frac{7}{150}t$$

$$\therefore t = \frac{150}{7} \times \log_{10}\frac{7.92}{7.19}$$

$$t = \frac{150}{7} \times 0.042$$

t=0.9 seconds

say t = 1 second & small case

t_r = time of rise = t/2 = 0.5 seconds

V = nozzle velocity and α = inclination to horizontal.

V Sin α = $g.t_r$

$$\therefore V = \frac{gt_r}{Sin\alpha}$$

= (980x0.5)/Sin (90-3)°

= (980x0.5)/Sin 87°

= 4.91 cm/s

6. Number of nozzles

Assuming:

N = No. of nozzles required

q = Discharge through each nozzle = C_d x V x a

where,

Cd = Coefficient of discharge = 0.9 (assuming)

V = nozzle velocity = 4.91 mps

a = nozzle area = (3.14/4)d^2

d= dia of nozzle = 25 mm

$\therefore$ Discharge through "N" number nozzles = N x C_d x V x a

= N x 0.9 x 4.91x (3.14/4) x $[25 \times 10^{-3}]^2$ m^3/sec.

But design flow i.e. discharge through N nozzles = 6000 m^3/day

N x 0.9 x 4.91x (3.14/4) x $[25 \times 10^{-3}]^2$x 60 x 60 x 24 = 6000 m^3/day

$\therefore$ N = 32

$\therefore$ Nozzles required = 32 Nos of 25 mm dia each.

7. **Spacing of Aerator Pipes**

Radius of spray = V Cosα x $2t_r$ = 4.91 Cos 87^0x2x0.5 = 0.257 m

Assuming wind velocity = 8 km/hr.

Wind Drag = Cd x Vw x t (assuming C_d = 0.6)

= 0.6 x $[(8 \times 10^3)/(60 \times 60)]$ x 1 = 1.33 m

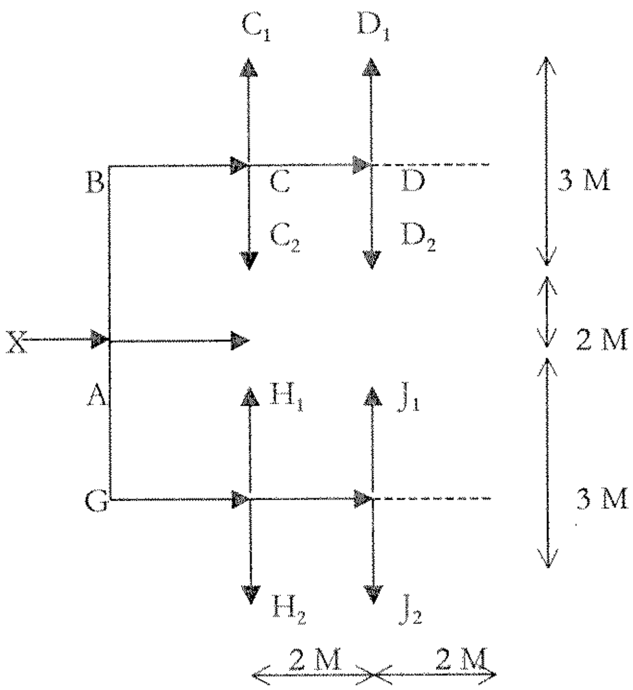
$\therefore$ Minimum spacing required = Radius of Spray + Wind drag

= 0.257 + 1.33

= 1.587 m say = 2m apart

8. **Arrangement of nozzles**

Nozzles are fixed on 4 rows of pipes as shown below;



No. of nozzles in each pipe = 32/4 = 8

Providing a spacing of 0.3 m c/c of nozzles and spacing in two adjacent rows and in staggered position.

Provide 4 pipes each of length 3m at a spacing of 2m.

Allowing 2m space on all the sides, the size of the aerator tray will be 8m x 6m.

Checking

Aerator pipes enclose an area of 2 x (3x2) = 12 m^2

$\therefore$ Area provided per m^3/day of design flow = 12/(250x24)= $2.0\times10^{-3} m^2/m^3$/day of design flow .

(O.K. since it is between 1.25×10^{-3} to $3.75\times10^{-3} m^2$/day of design flow).

9. Uniformity in distribution

The uniformity in distribution of water is maintained by arrangement of aerator pipes as in figure above.

Discharge through each pipe = (250x24)/4 = 1500 m^3/d

Assuming h = head loss at each nozzle

$V = C_v \sqrt{(2gh)} = 4.91$ m/s

$h = (4.91)^2 / [(0.9)^2 \times 2 \times 9.81]$ (Assuming $C_v = 0.9$)

Assuming variation of head = 2%

$$m_1 = \frac{\text{discharge through last nozzle in the pipe}}{\text{discharge through first nozzle in the pipe}} = 0.98$$

$H = h (1-m_1^2) = 1.52(1-0.98^2) = 0.06$ m

Head loss in the pipe for gradually diminishing flow = H = 0.06 m

$\therefore$ Corresponding head loss for uniform flow = H_u = 3H =3x0.06

= 0.18 m (per aerator pipe length)

$\therefore$ Head loss /1000 m = (0.18 /1.5) x1000 = 120

10. Design of pipes and head losses:

The arrangement of pipe is shown in Figure. The aerator pipes are so chosen that the velocity remains within 1 to 1.5 mps and corresponding head losses for pipes (C.I.) are calculated and are shown in the following table:

Pipe Section.	Design Flow (m^3/d)	Length (m)	Dia. (m)	Velocity (m/sec)	Head Loss(m)	Total Head Loss(m)
1	2	3	4	5	6	7
AB	3000	2.5	200	1.1	0.01	0.025
BC	3000	2.0	200	1.1	0.01	0.020
C_1C_2	1500	3.0	125	1.42	0.03	0.090
CD	1500	2.0	125	1.42	0.03	0.060
D_1D_2	1500	3.0	125	1.42	0.03	0.090
					Total	0.285

Total Head loss = 0.285 + 10% for valves and specials

= 0.314 m

say = 0.32 m

Head at 'A' = Terminal head + Total Head loss

= 1.52 + 0.32

= 1.84 m

APPENDIX 7.2

DESIGN OF MECHANICAL RAPID MIX UNIT

1. PROBLEM STATEMENT

Design a mechanical rapid mix unit using following data:

1. Design flow to be treated = $250\ m^3/hr$
2. Detention time = 30 secs (20-60 s)
3. Ratio of tank height to diameter = 1.5:1 (1-3:1)
4. Ratio of impeller diameter to tank diameter = 0.4:1(0.2 – 0.4:1)
5. Rotational speed of impeller = 120 rpm (>100 rpm)
6. Velocity gradient = $600S^{-1}(>300S^{-1})$
7. Assume temperature of 20^0 C.

2. SOLUTION

(i) Determine dimensions of tank

Volume = Flow x detention time

= 250 x (30/3600)= 2.083 m^3

Diameter of the tank, D, is calculated from $(\pi/4)D^2(1.5D) = 2.083$

Therefore, diameter of tank = 1.20 m

and height of tank = 1.80 m

Total height of the tank = 2 m which will provide a free board of 0.2 m

(ii) Compute power requirements

Power spent, P = μG^2. (Volume of tank)

= $1.0087 x 10^{-3} x (600)^2 x 2.083$ = 756 watts

Power per unit volume = 756/2.083 = 362.94

= Say 363 $watts/m^3$

Power per unit flow of water = 756/250 = 3.02 $watts/m^3/hr$ of flow

Determine dimensions of flat blades and impeller

Diameter of impeller = 0.4 x tank diameter

= 0.4 x 1.2 = 0.48 m

Velocity of the tip of impeller = $(2.\pi.r.n/60)$ m/s

= $((2\pi \times 0.48)/2) \times (120/60))$ m/s

$= 3.02$ m/s (O.K.)

Determine the area of blades A_p of impeller by the equation.

Power spent $= (1/2).C_D.\rho.A_p.V_r^3$

Assuming Newtons coefficient of drag, $C_D = 1.8$ for flat blades and relative velocity of paddle, V_r, as three fourths of the tangential velocity of the tip of the blade,

$756 = (1/2) \times (1.8 \times 1000 \times A_p) \times [0.75 \times 3.03]^3$

$A_p = 0.072 \text{ m}^2$

Provide 6 blades of size 0.1x0.12 m

(iv) Provide 4 Nos of baffles of length 1.9 m and projecting 0.10 m from the wall of the tank to reduce vortex formation.

(v) Provide inlet and outlet pipes of 200 mm diameter.

APPENDIX 7.3
DESIGN OF CLARIFLOCCULATOR

I. Problem Statement

Design clariflocculator using following data and design criteria:

1. Desired average outflow from clariflocculator = 250 m^3 / hr
2. Water lost in desludging = 2%
3. Design average flow = (250 x 100) / (100 - 2) = 255.1 m3 / hr
4. Detention period = 20 minutes
5. Average value of velocity gradient, G = $40S^{-1}$

II. Components to be designed

A circular clariflocculator is to be designed having vertical paddles. The water enters through a central influent pipe and is fed into the flocculation zone through parts. The effluent from flocculation zone passes below the partition wall dividing the flocculator portion and the clarifier portion. The clarified effluent is collected by a peripheral effluent launder. The components of clariflocculator to be designed include the influent pipe, the flocculator, the clarifier and the effluent launder.

III. Design of Influent pipe

Assuming a velocity of 1 m/s

$$\text{Influent pipe diameter} = \sqrt{\frac{255.1 \times 4}{3600 \times 1 \times \pi}} = 0.24$$

Provide an influent pipe of 300 mm diameter

IV. Design of Flocculator

Dimensionless Parameter, G.t = 40 x (20 x 60) = 4.8 x 10^4

This is acceptable as G.t = 2 to 6 x 10^{-4} for alum coagulants

Volume of flocculator = (255.1 x 20) / 60 m^3 = 85 m^3

Provide a water depth of 2.5 m

Plan area of flocculator = 8.5 / 2.5 m^2 = 34 m^2

Let D be the diameter of flocculator and D_p the diameter of the inlet pipe. Then

$$\frac{\pi}{4}\left(D^2 - D_P^2\right) = 34, \qquad \frac{\pi}{4}\left(D^2 - 0.3^2\right) = 34$$

D = 6.57 m

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Provide a tank diameter of 6.6 m.

V. Dimensions Of Paddles

Total power input to flocculator, $P = G^2 \mu$ (vol.)

$(40)^2$ x $[0.89 \times 10^{-3}]$ x $[\pi \times (6.6)^2 \times 2.5/4]$ = 122 watts

Power input = $(1/2) . C_D . \rho . A_p (V - v)^3$

Where

C_D = Newtons coefficient of drag, 1.8

ρ = Density of water at 25^0C, $997 kg/m^3$

V = Velocity of the tip of blades

= 0.4 m/s (recommended range 0.3-0.4 m/s)

v = Velocity of water at tip of blade

= 0.25 x 0.4 (25% of V)

= 0.1 m/s.

122 = $1.8 \times 997 \times Ap (0.4 - 0.1)^3 / 2$

A_p = $5.04 m^2$

Ratio of area of paddles to cross-sectional area of flocculator

= $A_p / \pi (D - D_p) \times h$

= $(5.04) / (\pi . (6.6 - 0.3) \times 2.5) = 0.102$ or 10.2%

This is acceptable as it is within the limits of 10 to 25%

Provide 8 Nos. of paddles of height 2.0 m and width of 0.32 m

Two shafts will support eight paddles, each shaft supporting 4 paddles. The shaft will be at a distance of (6.6 - 0.3)/4 = 1.58 m from the centre line of clariflocculator. The paddles will rotate at a rpm of 4.

Distance of paddle edge, r, from the centre line of vertical shaft is given by the equation.

$V = (2.\pi.r.n) / 60$

$0.4 = (2. \pi.r \times 4) / 60$

$\therefore r = 1$ m

Let the velocity of water below the partition wall between the flocculator and clarifier be 0.3 m / minute. Therefore area of opening required for a velocity 0.3 m/min below the partition wall will be

Area = $250 / (0.3 \times 60) = 13.9 m^2$

Depth below partition wall

= 13.9/ (π x 6.6) = 0.67m

Provide 25% additional depth for the storage of sludge in case the mechanical scraper is out of order.

Depth provided for sludge storage = 0.25 x 2.5 = 0.625 m say 0.63 m

Provide 8% slope for the bottom.

Total depth of tank at the partition will assuming a free board of 0.3 m

= 0.3 + 2.5 + 0.67 + 0.63 = 4.10 m.

VI. Design Of Clarifier

Assume a surface overflow rate of 40 $m^3/m^2/day$

Surface area of clarifier = 255.1 x 24 /40 = 153.06m2

Diameter of the clariflocculator, D_{cf} is given by

$$\frac{\pi}{4}\left[D_{cf}^2 - (6.6)^2\right] = 153.06 \qquad \therefore D_{cf} = 15.44m$$

Length of weir = $\pi.D_{cf}$ = π x 15.44 = 48.53m

Weir loading = (255.1 x 24)/(48.83) $m^3/day.m$ = 126.2m^3/day m (< 300 m^3/day m) O.K

APPENDIX 7.4
DESIGN OF RECTANGULAR PLAIN SEDIMENTATION TANK

I. PROBLEM STATEMENT

Design rectangular sedimentation tank with following data.

1.	Desired Average Outflow from sedimentation tank	= 250 m^3/hr.
2.	Water lost in desludging	= 2%
3.	Design Average flow	= (250x100)/(100-2) = 255.1m^3/hr
4.	Minimum size of the particle to be removed	= 0.02 mm
5.	Expected removal efficiency of min. size particle	= 75 %
6.	Nature of particles	= discrete and non flocculating
7.	Specific gravity of particles	= 2.65
8.	Assumed performance of the settling tank	= good (n = 1/4)
9.	Kinematic viscosity of water at 20 ° C	= 1.01×10^{-6} m^2/s

II. DESIGN PROCEDURE

For the given diameter and specific gravity of minimum size particles to be removed in settling tank, vertical settling velocity of the particle is calculated initially using Stoke's law. The computed settling velocity is used to determine Reynolds number to check whether Stoke's law is applicable. If Reynolds number exceeds 1, Hazen's formula is used to determine the settling velocity of particle. The settling velocity thus calculated is employed for computation of surface over flow rate for expected removal efficiency of minimum size particles and assumed performance of the settling basin. Alternatively the surface over flow rate for average design flow may be assumed on the basis of data presented in Table in section 7.5.6. The plan area is determined next, followed by tank dimensions. The depth of tank may be determined using detention period. Sizing of components of inlets and outlets is done using relevant design criteria & assumptions.

III. DESIGN STEPS

1. Compute vertical settling velocity of minimum size particles.

$$v_s = g(S_s-1).d^2 / (18 \times \nu)$$
$$= 9.81(2.65-1)(0.02 \times 10^{-3})^2 / (18 \times 1.01 \times 10^{-6})$$
$$= 3.56 \times 10^{-4} \text{ m/s}$$

Reynolds number = $(v_s.d) / \nu$

$= 3.56 \times 10^{-4} \times (0.02 \times 10^{-3}) / (1.01 \times 10^{-6})$

$= 704 \times 10^{-3} < 1$

Hence Stoke's law is applicable and computed settling velocity is correct.

2. Determine Surface Overflow Rate

For Ideal settling basin and complete removal of minimum size particles, equate settling velocity to theoretical surface over flow rate for 100% removal.

$V_s = V_o$

$V_o = 3.56 \times 10^{-4}$ m/s

$= 3.56 \times 10^{-4} \times 3600 \times 24 = 30.76$ m/d

However due to short circuiting, there is reduction in efficiency and decrease in surface overflow rate. To obtain design surface overflow rate, which would give expected removal efficiency of minimum size particles in real basin, use following relationship.

$y / y_o = 1 - [1 + n. (V_o / (Q/A))]^{-1/n}$

For $y / y_o = 0.75$, $n = 1/4$ (good performance of tank)

$V_o / (Q/A) = 1/ n\{[1- y / y_o]^{-n} - 1 \}$

$= 4 \times [(1 - 0.75)^{-1/4} - 1] = 1.66$

Hence Design Surface overflow rate at average design flow, Q/A

$Q/A = (V_o / 1.66) = 30.76/1.66 = 18.53$ m/d

Typical values for design surface overflow rate range between 15 and 30 $m^3/m^2/d$. for plain sedimentation tanks.

3. Calculate Dimensions Of Tank

Surface area of tank, A $= (Q / (Q/A))$

$= 255.1[m^3/hr] \times 24 / 18.53$

$= 330.4\ m^2$

Assume length to width ratio as 4

Length x width = surface area

$Width,\ B = \sqrt{(330.4/4)} = 9.09$

Length of tank, L = 36.36 m

Assume detention period, t, as 4 hrs.

Water depth of settling zone at average flow $= Q \times t / A$

$= 255.1 \times 4 / (36.36 \times 9.09) = 3.09$ m