### TABLE III

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

<table>
<thead>
<tr>
<th>Sl. No.</th>
<th>Pipe size in mm</th>
<th>Total head in m</th>
<th>Class of CI pipes required</th>
<th>Rate* per m length</th>
<th>Cost of 7000 m pipe line</th>
<th>1st stage flow of 7.5 MLD</th>
<th>2nd stage flow of 10 MLD</th>
<th>Grand total of capitalized cost for 30 years (16) = (11) + (15)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>300</td>
<td>115</td>
<td>A</td>
<td>640.77</td>
<td>4486</td>
<td>520</td>
<td>1192</td>
<td>9067</td>
</tr>
<tr>
<td>2</td>
<td>350</td>
<td>80</td>
<td>LA</td>
<td>804.92</td>
<td>5635</td>
<td>360</td>
<td>829</td>
<td>6306</td>
</tr>
<tr>
<td>3</td>
<td>400</td>
<td>75</td>
<td>LA</td>
<td>918.72</td>
<td>6431</td>
<td>340</td>
<td>777</td>
<td>5910</td>
</tr>
<tr>
<td>4</td>
<td>450</td>
<td>60</td>
<td>LA</td>
<td>1106.51</td>
<td>7746</td>
<td>280</td>
<td>622</td>
<td>4731</td>
</tr>
<tr>
<td>5</td>
<td>500</td>
<td>55</td>
<td>LA</td>
<td>1302.66</td>
<td>9119</td>
<td>260</td>
<td>570</td>
<td>4336</td>
</tr>
</tbody>
</table>

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

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

---

8* Cost of pumpsets

10* Energy charges capitalised = \( C_{CI} = 7.606 \text{ CR}_1 \)

12* Cost of pump sets

14* Energy charges capitalised = \( C_{CI} = 7.606 \text{ CR}_1 \)

9* Annual cost of energy charges = \( 700/1.65 \text{ KW}_1 = \text{ CR}_1 \)

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

13* Annual cost of energy charges = \( 7348.63 \text{ KW}_2 = \text{ CR}_2 \)

15* Initial capital investment for pumpsets and annual electrical charges = \( \text{(Col. 12 + Col. 14) / 4.177} \)
APPENDIX 6.6

DESIGN OF THRUST BLOCKS

To design a thrust block for 900mm diameter main conveying water at 11kgs/cm² pressure (P).

The deviation angle $\alpha$ is 45° and density of concrete is 2300 kgs/m³. Soil density is assumed to be 1800 kgs/m³ and angle of internal friction $\phi = 30^\circ$.

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.

$Sin \ alpha/2 = 0.382$

$F = 2 \ (11) \ (6364) \ (0.382) \ (10^{-5}) = 53.48$ Tonnes

(i) Lateral resistance to counteract the horizontal thrust:

Try a thrust block of size = $3.2M \times 3.2 \ M \times 3.2 \ M$

Weight of thrust block = $3.2M \times 3.2 \ M \times 3.2 \ M \times 2.3 = 75.36$ 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:

$fp = p \ (H)^2 / 2 \ \ L \ [1 + Sin \ theta / Sin \ theta + 2CHL \ \ V (1 + Sin \ theta) / (1 - Sin \ theta)]$

By assuming cohesion as 0, the above equation

Yields = $1.8 \ (3.2)^2 / 2 \ (3.2) \ (1.5) / 0.5 = 88.47$ 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:

\[ \frac{f_a}{\gamma} = h \left( \frac{1 - \sin \theta}{1 + \sin \theta} \right) - 2C \sqrt{\frac{1 - \sin \theta}{1 + \sin \theta}} \]

\[ = (1.8)(0.9) \left( \frac{0.5}{1.5} \right) = 0.54 \text{ tonnes} \]

Total lateral resistance = 24.15 + 88.47 + 0.54 = \textbf{113.16 tonnes/m}^2

Total horizontal thrust = \textbf{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.
THRUSTR 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

\[ H_o = \text{Absolute Head} = \text{Total head} + \text{Atmospheric Head} = 135 + 10 = 145 \text{ m} \]

\[ V_o = \text{Initial velocity} \]

\[ C = \text{Water Hammer Wave velocity}, \quad C = \sqrt{\frac{1425}{1 + \frac{kd}{Ec}}} \]

(Refer 6.17.2 of Manual)

\[ = \sqrt{1 + \left[ \frac{2.07 \times 10^8}{2.1 \times 10^{10} \times 0.01} \right]} = 896.27 \text{ m/s} \]

Water Hammer head

\[ h = \frac{cV_o}{g} = \frac{896.27 \times 1.03}{9.81} = 94.10 \text{ meters} \]

Pipeline Parameter

\[ \rho = \frac{cV_o}{2gH_o} \]

\[ = \frac{896.27 \times 1.03}{2 \times 9.81 \times 145} = 0.324 \]

\[ 2\rho = 0.65 \]
AIR VESSEL PARAMETER = 2C_oC/Q_oL

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

From chart for 2p = 1.00x(2C_oC/Q_oL)=10.50
For 2p = 0.5, 2C_oC/Q_oL = 6.50
By interpolation for 2p = 0.65 and for k_e =0.5
Air vessel parameter 2C_oC/Q_oL =7.70
Volume of air C_o = 7.7x1.944x18000
  2x896.27
  = 150.31 Cubic meters

Volume of Air Vessel = C_o[H_o/H_max]^1/2p
  = 150.31[(145)/0.5x(145)^1/2p]
  = 150.31x(2)^1/1.2
  = 267.20 Cum

Increase the capacity by 20% to cater for upsurge of 1.20 H_o
  = 267.20 x 1.2
  = 320 Cumns

WATER COLUMN SEPERATION LENGTH

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

\[ V_1^2 - V_2^2 = (2g/L)\{(t_{1} - t_{2}) V_1 \{H + F (V_1^2 / V_o)\}\} \]

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_o) is almost Zero. The water column separation length l is given by Laws =

\[ lwcs \sum [V_1 + V_2 + \ldots \ldots V_n] (t_2 - t_1) \]

613
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

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

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

Repeat n times till \( V_n = 0.01 \ m/sec. \)

Then \( V_1 + V_2 + V_3 + \ldots \ldots \ldots V_n \ (0.20) \approx \text{say } = 6.1 \text{ meters.} \)

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

\[
\frac{\pi \times (1.55)^2 \times 6.10}{4} = 11.51 \text{ Cum}
\]

**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 x 1.2 = 192.42 meters

Pressure = 19.25 kg/cm²

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

\[
d = \frac{2f_e \times e \times t}{P}
\]

\( f_e = \text{Permissible tensile strength in steel plates } = 1260 \text{ kgs/cm}^2 \)

\( e = \text{Weld efficiency say 0.9} \)

\( t = \text{Thickness in cms of plate } = 2.2 \text{ cm} \)

\( P = \text{Pressure in kgs/cm}^2 \)

\[
\frac{2 \times 1260 \times 1.90 \times 2.20}{19.25} = 250.20 \text{ cms} \\
= \text{ Say 260 cms}
\]

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

Volume of (two hemispheres) spherical portion = \( 4 \times \pi (1.3)^2 = 9.2 \text{ Cum} \).

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 m³/hr.
   Pipe used = 70 mm dia ‘B’ Class C.I. Pipe with a C value of 100
   \( S = 3.60 \text{ meters } /1000 \text{ meters} \)
   \( V = 1.35 \text{ m/s} \)
2. Iron present in raw water = 1.8 mg/l.
3. Saturation concentration of \( O_2 \) at 28 °C = 7.92 mg/l.
4. Aeration constant (to the common base) at 28 °C = 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° to 5° 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 \( 1.25 \times 10^{-3} \) to \( 3.75 \times 10^{-3} \text{ m}^2 \) per \( \text{m}^3/\text{day} \) of design flow.

III. SOLUTION
1. Design flow = 6000 m³/day.
2. Assuming 25-mm dia. nozzle with an inclination of 3° 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. \( Fe^{++} + 3O_2 = 2Fe_2O_3 \)
   \( 1.7 \text{ mg/l of Fe requires } 1.7 \times 96/224 = 0.7286 \text{ mg/l of } O_2 \).
5. By applying ‘Gas absorption’ equation in 7.2.2 in the form
\[ \log_{10}\left[\frac{(C_i - C_o)}{(C_i - C_s)}\right] = \frac{KAt}{V} \]

where

\[ C_s = 7.92 \text{ mg/l at 28}^\circ\text{C, } C_o = 0.0 \text{ mg/l.} \]
\[ C_i = 0.73 \text{ mg/l, } K = 70 \text{ cm/hr} \]
\[ \frac{A}{V} = \frac{6}{d} = \frac{6}{2.5 \left(\text{cm}^{-1}\right)} \]

\[ \therefore \log_{10}\left(\frac{7.92}{7.19}\right) = \frac{70}{60 \times 60} \times \frac{6}{2.5} \times t = \frac{7}{150} t \]

\[ \therefore t = \frac{150}{7} \times \log_{10}\left(\frac{7.92}{7.19}\right) \]
\[ t = \frac{150}{7} \times 0.042 \]
\[ t = 0.9 \text{ seconds} \]

say \( t = 1 \text{ second & small case} \)

\( t_r = \text{time of rise} = \frac{t}{2} = 0.5 \text{ seconds} \)

\( V = \text{nozzle velocity and } \alpha = \text{inclination to horizontal.} \)

\( V \sin \alpha = g t_r \)

\[ \therefore V = \frac{g t_r}{\sin \alpha} \]

\[ = \frac{(980 \times 0.5)}{\sin (90.3)^\circ} \]
\[ = \frac{(980 \times 0.5)}{\sin 87^\circ} \]
\[ = 4.91 \text{ cm/s} \]

6. **Number of nozzles**

Assuming:

\( N = \text{No. of nozzles required} \)

\( q = \text{Discharge through each nozzle} = C_d \times V \times a \)

where,

\( C_d = \text{Coefficient of discharge} = 0.9 \) (assuming)

\( V = \text{nozzle velocity} = 4.91 \text{ mps} \)

\( a = \text{nozzle area} = (3.14/4) d^2 \)

\( d = \text{dia of nozzle} = 25 \text{ mm} \)
\[ \text{Discharge through "N" number nozzles} = N \times C_d \times V \times a \]
\[ = N \times 0.9 \times 4.91 \times (3.14/4) \times [25 \times 10^{-3}]^2 \text{ m}^3/\text{sec.} \]

But design flow i.e. discharge through N nozzles \[ = 6000 \text{ m}^3/\text{day} \]
\[ N \times 0.9 \times 4.91 \times (3.14/4) \times [25 \times 10^{-3}]^2 \times 60 \times 60 \times 24 = 6000 \text{ m}^3/\text{day} \]
\[ \therefore N = 32 \]
\[ \therefore \text{Nozzles required} = 32 \text{ Nos of 25 mm dia each.} \]

7. Spacing of Aerator Pipes

Radius of spray \[ = V \cos \alpha \times 2t = 4.91 \cos 87^\circ \times 2 \times 0.5 = 0.257 \text{ m} \]
Assuming wind velocity \[ = 8 \text{ km/hr.} \]
Wind Drag \[ = C_d \times V_w \times t \ (\text{assuming } C_d = 0.6) \]
\[ = 0.6 \times [(8 \times 10^3)/(60 \times 60)] \times 1 = 1.33 \text{ m} \]
\[ \therefore \text{Minimum spacing required} = \text{Radius of Spray + Wind drag} \]
\[ = 0.257 + 1.33 \]
\[ = 1.587 \text{ m} \quad \text{say } = 2 \text{ m apart} \]

8. Arrangement of nozzles

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

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

618
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 m²

\[ \text{Area provided per m}^3/\text{day of design flow} = \frac{12}{(250\times24)} = 2.0\times10^{-3} \text{m}^3/\text{m}^3/\text{day of design flow} . \]

(O.K. since it is between 1.25 x 10⁻³ to 3.75x10⁻³m³/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 = \((250\times24)/4 = 1500 \text{ m}^3/\text{d}\)

Assuming \( h = \text{head loss at each nozzle} \)

\[ V = C_v \sqrt{2gh} = 4.91 \text{ m/s} \]

\[ h = (4.91)^2 / [(0.9)^2 \times 2 \times 9.81] \quad \text{(Assuming} \ C_v = 0.9) \]

Assuming variation of head = 2%

\[ m_i = \frac{\text{discharge through last nozzle in the pipe}}{\text{discharge through first nozzle in the pipe}} = 0.98 \]

\[ H = h (1-m_i^2) = 1.52(1-0.98^2) = 0.06 \text{ m} \]

Head loss in the pipe for gradually diminishing flow = \(H = 0.06 \text{ m}\)

\[ \therefore \text{Corresponding head loss for uniform flow} = H_u = 3H = 3 \times 0.06 \]

\[ = 0.18 \text{ m (per aerator pipe length)} \]

\[ \therefore \text{Head loss /1000 m} = (0.18 /1.5) \times 1000 = 120 \]

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:
<table>
<thead>
<tr>
<th>Pipe Section</th>
<th>Design Flow (m³/d)</th>
<th>Length (m)</th>
<th>Dia. (m)</th>
<th>Velocity (m/sec)</th>
<th>Head Loss (m)</th>
<th>Total Head Loss (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>AB</td>
<td>3000</td>
<td>2.5</td>
<td>200</td>
<td>1.1</td>
<td>0.01</td>
<td>0.025</td>
</tr>
<tr>
<td>BC</td>
<td>3000</td>
<td>2.0</td>
<td>200</td>
<td>1.1</td>
<td>0.01</td>
<td>0.020</td>
</tr>
<tr>
<td>C₁C₂</td>
<td>1500</td>
<td>3.0</td>
<td>125</td>
<td>1.42</td>
<td>0.03</td>
<td>0.090</td>
</tr>
<tr>
<td>CD</td>
<td>1500</td>
<td>2.0</td>
<td>125</td>
<td>1.42</td>
<td>0.03</td>
<td>0.060</td>
</tr>
<tr>
<td>D₁D₂</td>
<td>1500</td>
<td>3.0</td>
<td>125</td>
<td>1.42</td>
<td>0.03</td>
<td>0.090</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>0.285</strong></td>
</tr>
</tbody>
</table>

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 m³/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 (>300 S⁻¹)
7. Assume temperature of 20° C.

2. SOLUTION

(i) Determine dimensions of tank
Volume = Flow x detention time
= 250 x (30/3600) = 2.083 m³
Diameter of the tank, D, is calculated from \((\pi/4)D²(1.5D) = 2.083\)
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, \(P\) = \(\mu G²\) (Volume of tank)
= 1.0087×10⁻³×(600)²×2.083 = 756 watts
Power per unit volume = 756/2.083 = 362.94
= Say 363 watts/m³
Power per unit flow of water = 756/250 = 3.02 watts/m³/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 = \((2.π.r.n/60)\) m/s
= \((2π x 0.48)/2\) x \(120/60\) m/s
\[ = 3.02 \text{ m/s (O.K.)} \]

Determine the area of blades \( A_p \) of impeller by the equation.

Power spent = \((1/2).C_D.p.A_p.V_t^3\)

Assuming Newtons coefficient of drag, \( C_D = 1.8 \) for flat blades and relative velocity of paddle, \( V_o \) 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 m$^3$ / hr
4. Detention period
   = 20 minutes
5. Average value of velocity gradient, G
   = 408$^{-1}$

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

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

$\frac{\pi}{4} (D^2 - D_p^2) = 34$, $\frac{\pi}{4} (D^2 - 0.3^2) = 34$

D = 6.57 m
Provide a tank diameter of 6.6 m.

V. DIMENSIONS OF PADDLES

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

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

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

Where

\[
C_D = \text{Newtons coefficient of drag, 1.8}
\]

\[
\rho = \text{Density of water at 25}^{\circ}\text{C, 997kg/m}^3
\]

\[
V = \text{Velocity of the tip of blades}
\]

\[
= 0.4 \text{ m/s (recommended range 0.3-0.4 m/s)}
\]

\[
v = \text{Velocity of water at tip of blade}
\]

\[
= 0.25 \times 0.4 \text{ (25% of V)}
\]

\[
= 0.1 \text{ m/s.}
\]

\[
122 = 1.8 \times 997 \times \rho \cdot (0.4 - 0.1)^3 / 2
\]

\[
A_p = 5.04 \text{ m}^2
\]

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

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

\[
= (5.04) / (\pi \cdot (6.6 - 0.3) \times 2.5) = 0.102 \text{ or 10.2%}
\]

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 \((6.6 - 0.3)/4 = 1.58 \text{ m from the centre line of clarifier flocculator}\). 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.

\[
V = (2\pi r n) / 60
\]

\[
0.4 = (2 \cdot \pi \cdot r \times 4) / 60
\]

\[
\therefore r = 1 \text{ m}
\]

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

\[
\text{Area} = 250 / (0.3 \times 60) = 13.9 \text{ m}^2
\]
Depth below partition wall
\[ = \frac{13.9}{(\pi \times 6.6)} = 0.67\text{m} \]

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
\[ = 0.3 + 2.5 + 0.67 + 0.63 = 4.10\text{m} \]

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 = 153.06 m\(^2\)

Diameter of the clariflocculator, \(D_{cf}\) is given by
\[ \frac{\pi}{4} \left[ r_{cf}^2 - (6.6)^2 \right] = 153.06 \quad \therefore \quad D_{cf} = 15.44\text{m} \]

Length of weir = \(\pi D_{cf} = \pi \times 15.44 = 48.53\text{m} \)

Weir loading = \((255.1 \times 24)/(48.83)\) m\(^3\)/day.m = 126.2 m\(^3\)/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 m³/hr.
2. Water lost in desludging = 2%
3. Design Average flow = (250x100)/(100-2) = 255.1 m³/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 = 1.01x10⁻⁶ m²/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.
   \[ v_s = \frac{g(S_r-1)d^2}{18 \times v} \]
   \[ = 9.81(2.65-1)(0.02 \times 10^{-3})^2 / (18 \times 1.01 \times 10^{-6}) \]
   \[ = 3.56 \times 10^{-4} \text{ m/s} \]
   Reynolds number = \( \frac{v_s \cdot d}{v} \)
\[
= 3.56 \times 10^{-4} \times (0.02 \times 10^{-5}) / (1.01 \times 10^{-6}) \\
= 704 \times 10^{-3} < 1
\]
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 overflow rate for 100% removal.

\[
V_s = V_o \\
V_o = 3.56 \times 10^{-4} \text{ m/s} \\
= 3.56 \times 10^{-4} \times 3600 \times 24 = 30.76 \text{ m/d}
\]

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.

\[
y / y_o = 1 - [1 + n. (V_o / (Q/A))]^{-1/n} \\
\text{For } y / y_o = 0.75, \ n = 1/4 \text{ (good performance of tank)} \\
V_o / (Q/A) = 1 / n \{1 - y / y_o\}^{n} - 1 \\
= 4 \times [(1 - 0.75)^{1/4} - 1] = 1.66
\]

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

\[
Q/A = (V_o / 1.66) = 30.76 / 1.66 = 18.53 \text{ m/d}
\]

Typical values for design surface overflow rate range between 15 and 30 m³/m²/d for plain sedimentation tanks.

3. **Calculate Dimensions Of Tank**

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

\[
= 255.1 \text{[m}^3/\text{hr}] \times 24 / 18.53 \\
= 330.4 \text{ m}^2
\]

Assume length to width ratio as 4

Length x width = surface area

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

Length of tank, \(L = 36.36 \text{ m}\)

Assume detention period, t, as 4 hrs.

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

\[
= 255.1 \times 4 / (36.36 \times 9.09) = 3.09 \text{ m}
\]

627