Water treatment plant designer document

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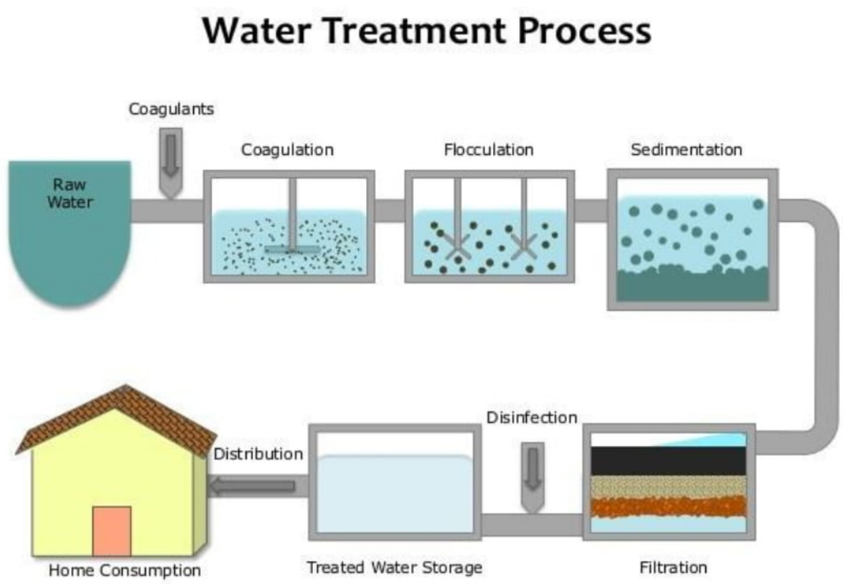
Design a water treatment plant.

Link to all the calculations: Water treatment plant designer

Link to all autoCAD designs: Water treatment plant design details

Assumptions: Majority of Water to be treated comes from a dam(distance ~10 km)

Since we need to treat river water the basic outline of our filtration system will be like follow:



Current Population: 101005
Design period: 15 years
Design population: Assume geometric population growth with a growth rate of 7.5%

$P_n = P_0(1+r/100)^n$

$P_0 = 101005$

$R = 7.5\%$

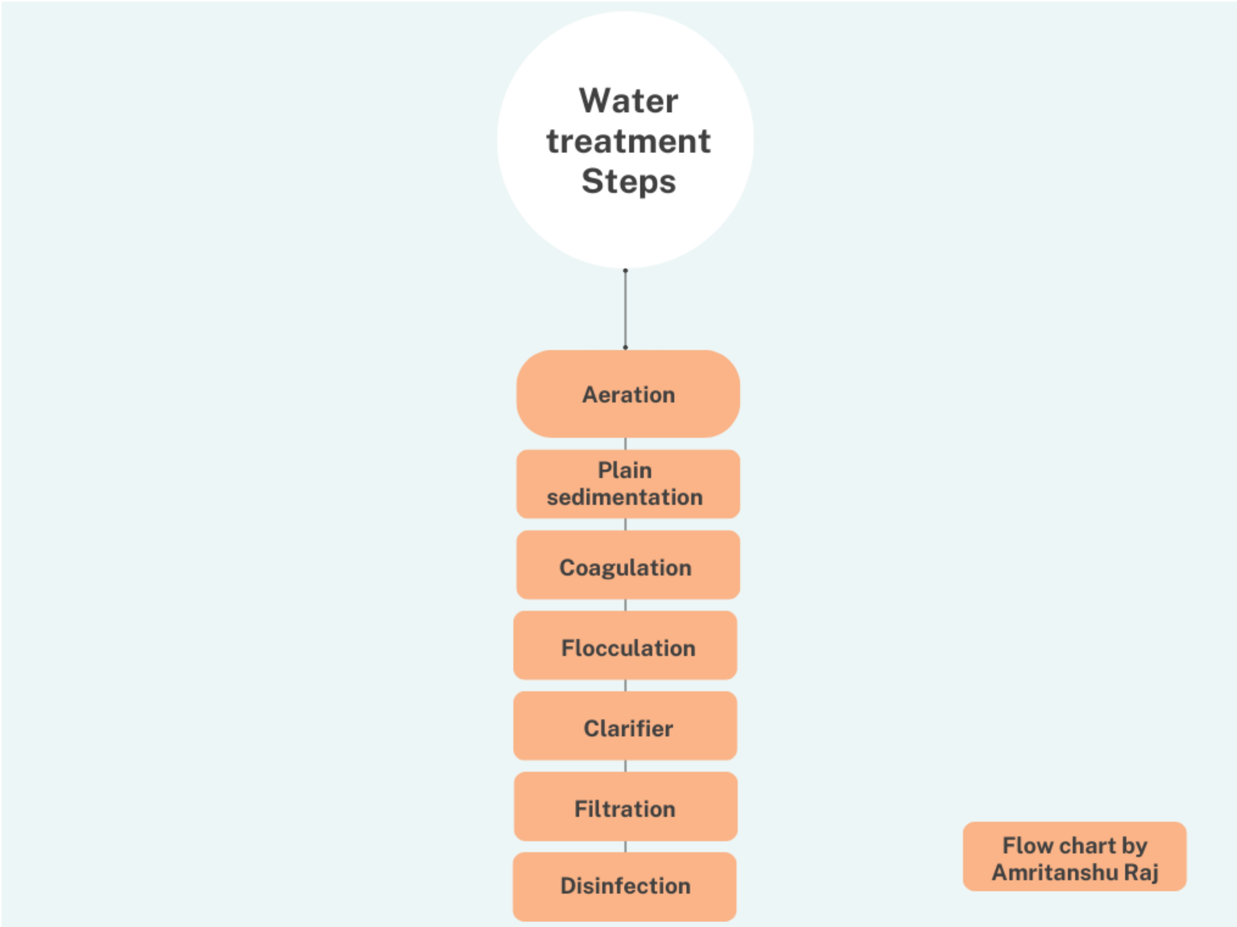
n = 15/10 = 1.5

$P_n = 112579$

Net water demand (liters/day) = design population x per capita demand
= 112579 x 135 = 15198165 liters/day

Assume 20% water is lost in various treatment processes,
now net water required a day = 1.2 x 15198165 liters/day = 18237798 liters/day
= 18.237798 MLD = 0.211085625 m^3/s

Now summarizing the water treatment steps we need to follow:



1. Aeration
   Since the influent water is coming from a dam, It will have high organic content. Aeration becomes an important step as bacteria used in water treatment and stabilization are given oxygen by aeration. The bacteria require oxygen for biodegradation to take place. Bacteria in the wastewater use the given oxygen to break down organic stuff that contains carbon to produce carbon dioxide and water. Bacteria cannot biodegrade the incoming organic materials in desired time if there is insufficient oxygen present. Degradation must take place under septic conditions, which are sluggish, unpleasant, and result in incomplete conversions of pollutants, in the absence of dissolved oxygen. Also it will help remove dissolved CO2 and H2S like gasses.

Design of aerator:

Q = 759.90825 m^3/h

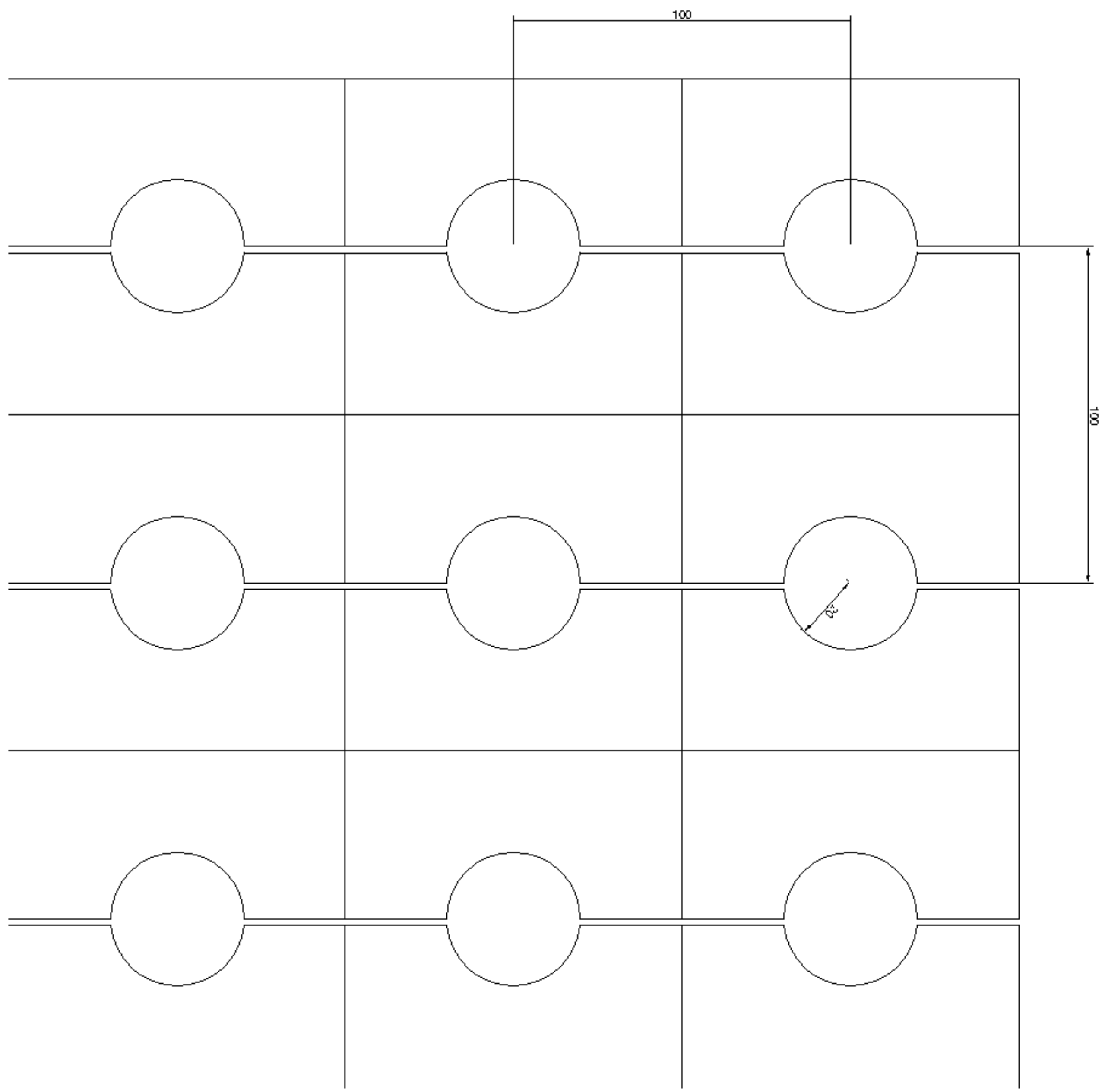
Type of aerator: Diffused aerator

A diffused aerator is highly energy efficient since only air needs to be pumped into the tanks from the diffusers placed at the bottom of tanks. The ability to save energy of 30% to 70% over surface aerators depending on the type of surface aerator used is a big benefit of fine bubble diffusers.

Dimensions of aerator tanks = (l x b x h) = (10 * 8* 5)m^3
Plan area of aerator tank: 80 m^2
Volume of aerator tank = 400 m^3
No of tanks required = 2
No. of diffusers per tank = 80 (1 per 1 m^2 of plan area)
Total diffusers required = 2 x 80 = 160
Radius of a diffuser = 0.2 m

Detailed distribution of diffusers in the aeration tank is given in the following diagram:
Detailed Auto-CAD plan of aerator tank showing distribution of diffusers can be found here:
(https://drive.google.com/drive/folders/1u1UNCUCInDnFk1iY33DjOfc5qn0hssFo?usp=sharing)

(All units in ‘cm’)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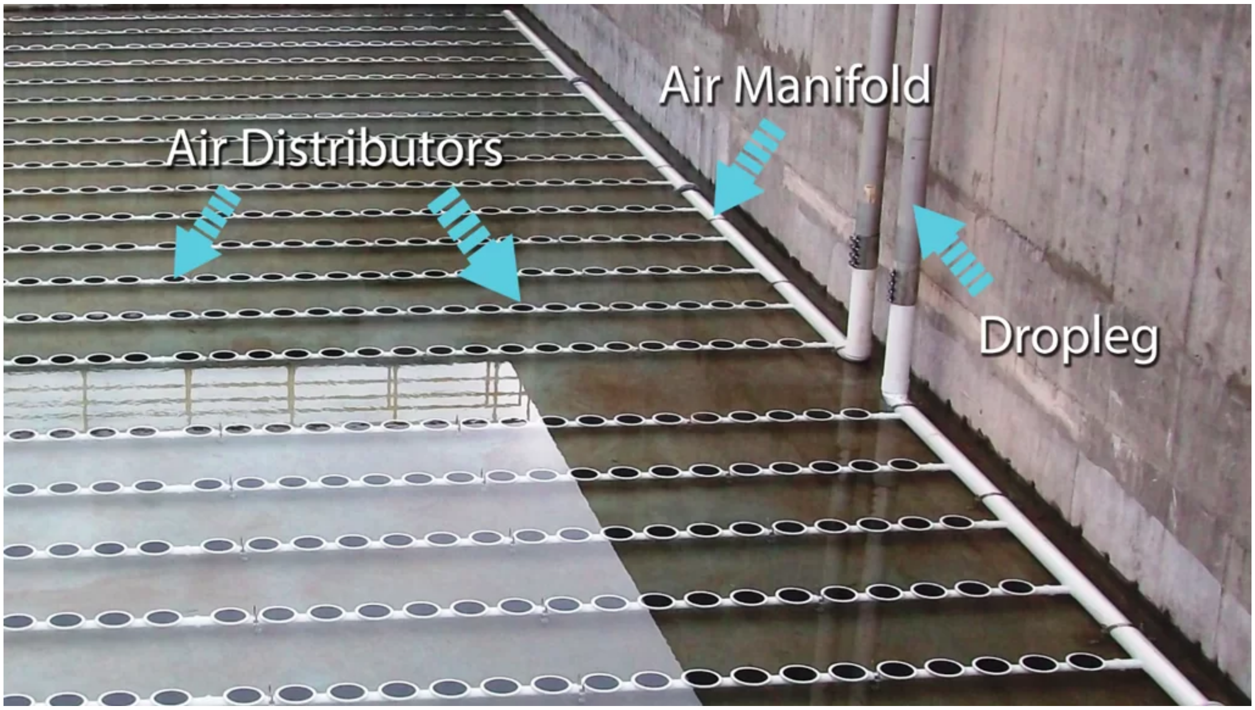
100
20



Diffuse type aerator set-up

Coagulation:

Coagulation is the chemical water treatment process used to remove solids from water, by manipulating electrostatic charges of particles suspended in water. This process introduces small, highly charged molecules into water to destabilize the charges on particles, colloids, or oily materials in suspension. The influent water will have colloidal content as clayey particles. We will use Alum as coagulant and limestone to provide optimum pH for coagulation to occur.

Rapid mixing unit:

Q = 759.90825 m^3/h

Assume Mixing time to be 2 min (standard practice)

Volume of mixing basin = (759.90825 x 2)/60 m^3 = 25.330275 m^3

Generally rapid mixing unit is cubical in shape:

Assume the Rapid mix unit be with side length ‘a’ : a^3 = 25.330275 m^3, a = 2.936837 m

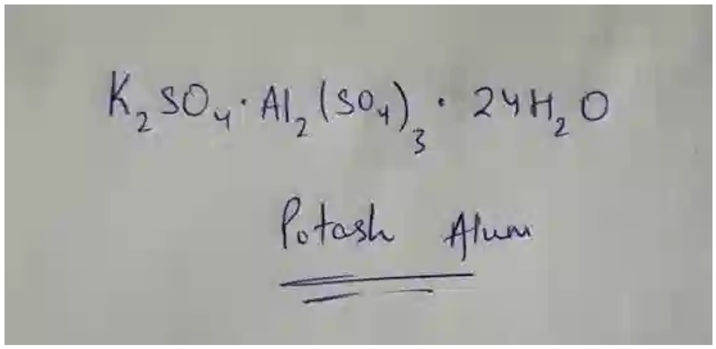
Assume a to be nearest integer for ease of construction: a = 3 m

Volume of rapid mix tank = 27 m^3

At room temperature T = 20 deg C, Dynamic viscosity will be $\mu = 1.0016 * 10^{-3}$ $N.s/m^2$
Assume G = 800/s

Power required:
$P = G^2\mu V = 800^2*1.0016 * 10^{-3}*27 = 17.307648$ Kwatt

Monthly Alum dosage requirements:
Chemical formula:



Optimal dosage: 20 mg/liter as A study has showed that the removal efficiencies for turbidity and optimum alum dosage were (93% at 20 mg/l, 92% at 20 mg/l, 85% at 30 mg/l, 88% at 30 mg/l and 89.3% at 30 mg/l). And hence we will use 20 mg/l for maximum turbidity removal efficiency.

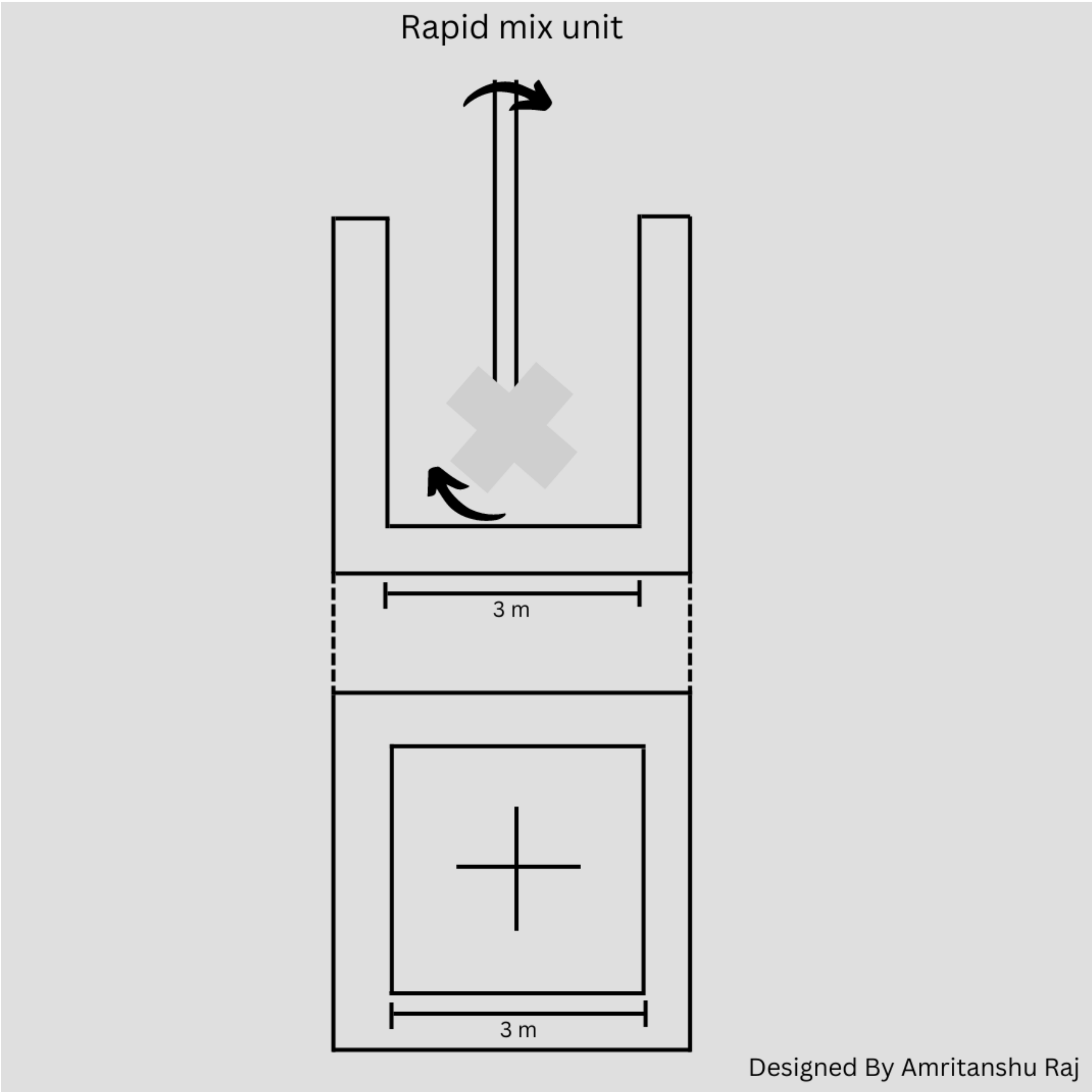
Q = 759.90825 m^3/h = 547133.94 m^3/month
Optimal Alum dose = 20 mg/l = 0.02 kg/m^3
Alum required = 0.02 x 547133.94 = 10942.6788 kg/month ~ 11,000 kg/month

The pH during coagulation should be maintained in the range of 7-8 for proper flocc formation. This can be achieved by using lime (Ca(OH)2).

The coagulation is generally required only during floods. However to treat water with high efficiency we can perform coagulation year round.

Rapid mix unit design:

Diagram on next page



Sedimentation

Sedimentation refers to physical separation of solid concentration from water. The driving force of sedimentation is gravitational force.

Design:
Shape: rectangular sedimentation tank
Q = 759.90825 m^3/h
We will divide this flow into 4 tanks. So the each tank will have of 189.98 m^3/h

Assumptions:
1) Maximum size of the particle to be removed d = 0.02 mm
2) Expected removal efficiency = 75%
3) Specific gravity G = 2.65
4) Assumed performance of tank = good = n = 1⁄4
5) Dynamic viscosity $\mu = 1.0016 * 10^{-3}$ $N.s/m^2$ (T = 20°C)
6) Kinematic viscosity = 1.00×10^{-6} m^2/s

1) Vertical settling velocity
Settling Velocity $V_s = (g \times (\rho_p - \rho_w) \times d^2) / (18 \times \mu)$

$= (9.81 \times 1000 \times (2.65 - 1) \times (0.02 \times 10^{-3})^2) / (18 \times 1.0016 \times 10^{-3})$

$= 3.591253994 \times 10^{-4}$ m/s

$V_s = 3.59 \times 10^{-4}$ m/s

Reynolds's number $Re = (\rho \times V_s \times d) / \mu$

$= (1000 \times 3.59 \times 10^{-4} \times 0.02 \times 10^{-3}) / 1.0016 \times 10^{-3}$

$= 7.17 \times 10^{-3} < 1$

Hence stoke's law is applicable and thus computed settling velocity is true.

2)Surface overflow rate

For ideal settling basin and complete removal of particles, we will equate settling velocity to theoretical surface overflow rate.

$V_0 = V_S = 3.59 \times 10^{-4} \times 3600 \times 24 = 20.68$ m/day

$Y / Y_0 = 1 - (1 + (n \times (V_0/(Q/A))))^{-1/n}$

As removal efficiency is 75% $Y / Y_0 = 0.75$

n = ¼ (for good performance of tank)

$0.75 = 1 - (1 + (0.25 \times (V_0/(Q/A))))^{-1/0.75}$

$V_0/(Q/A) = 1.657$

$Q/A = V_0 / 2.6 = 12.48$ m/day

3) Dimensions of tank

Surface area of tank A = Q/(Q/A)= (189.9770625 x 24) / 12.48

A = 365.2298904 m^2

Assuming length to width ratio as 3

A = Length x width = 3w x w

w = √(365.2298904/3) = 11.03373449m

w = 11.03373449 m

L = 3w = 33.10120347 m

L = 33.10120347 m

Assuming a detention period of 4 hours

Depth of tank = Q x t / A = 189.9770625 x 4 / (11.03 x 33.10)

D = 2.08 m

4) Check against resuspension of deposited particles

Velocity that can initiate resuspension of deposited

Particles or scour velocity $V_d = \sqrt{(8 \times \beta / f') \times g \times (G - 1) \times d}$

β = 0.04 for uni granular sand
f' = Darcy Weisbach friction factor = 0.03

$V_d = \sqrt{(8 \times 0.04 / 0.03) \times 9.81 \times (2.65 - 1) \times (0.02 \times 10^{-3})}$

$V_d = 5.8 \times 10^{-2}$ m/s

Horizontal velocity V_h = Q/ (w x D)

= (189.9770625 /(60 x 60)) / (11.03373449 x 2.080629954)

$V_h = 0.229 \times 10^{-2}$ m/s < V_d

Hence O.K.

FLOCCULATION

Flocculation is the process of slow mixing in a flocculator so that the particles of the precipitate collide and form a large and dense cluster that will readily settle.

DESIGN

Flocculation Basin Dimensions

Let us assume two trains with three compartments each.

Assumption $GT = 4.5 \times 10^4$

For first compartment $G_1 = 50/s$

For second compartment $G_2 = 40/s$

For third compartment $G_3 = 30/s$

For fourth compartment $G_4 = 20/s$

For fifth compartment $G_5 = 10/s$

Average compartment $G = (50+40+30+20+10)/5 = 30/s$

Average Detention Time $T = GT/G = (4.5 \times 10^4)/30 = 25$ min

$T = 25$ min

$Q = 759.90825\ m^3/h = 18237.798\ m^3/day = 12.6651375\ m^3/min$

=> Volume $V = 12.6651375 \times 25 = 316.6284375$ m3 $\quad V = 316.63\ m^3$

Let the depth of the tank $d = 5$ m

Area $A = V / d = 316.63 / 5 = 63.3256875\ m^2 \quad A = 63.3256875\ m^2$

Total number of compartments = 2 x 5 = 10

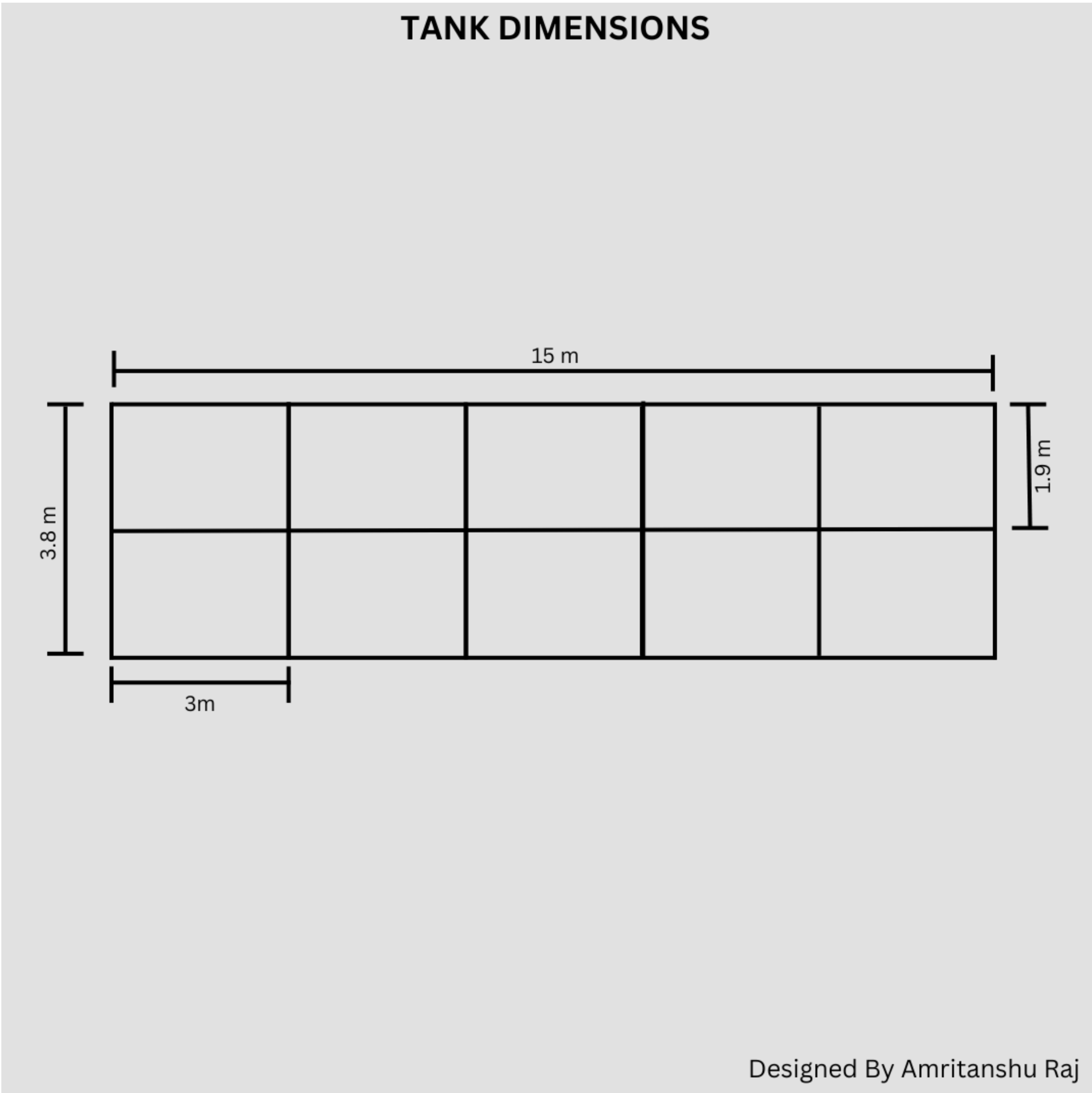
Area of one compartment = $63.3256875 / 10 = 6.33\ m^2$

Assume the length of one stage as 3 m

Width of each stage will be

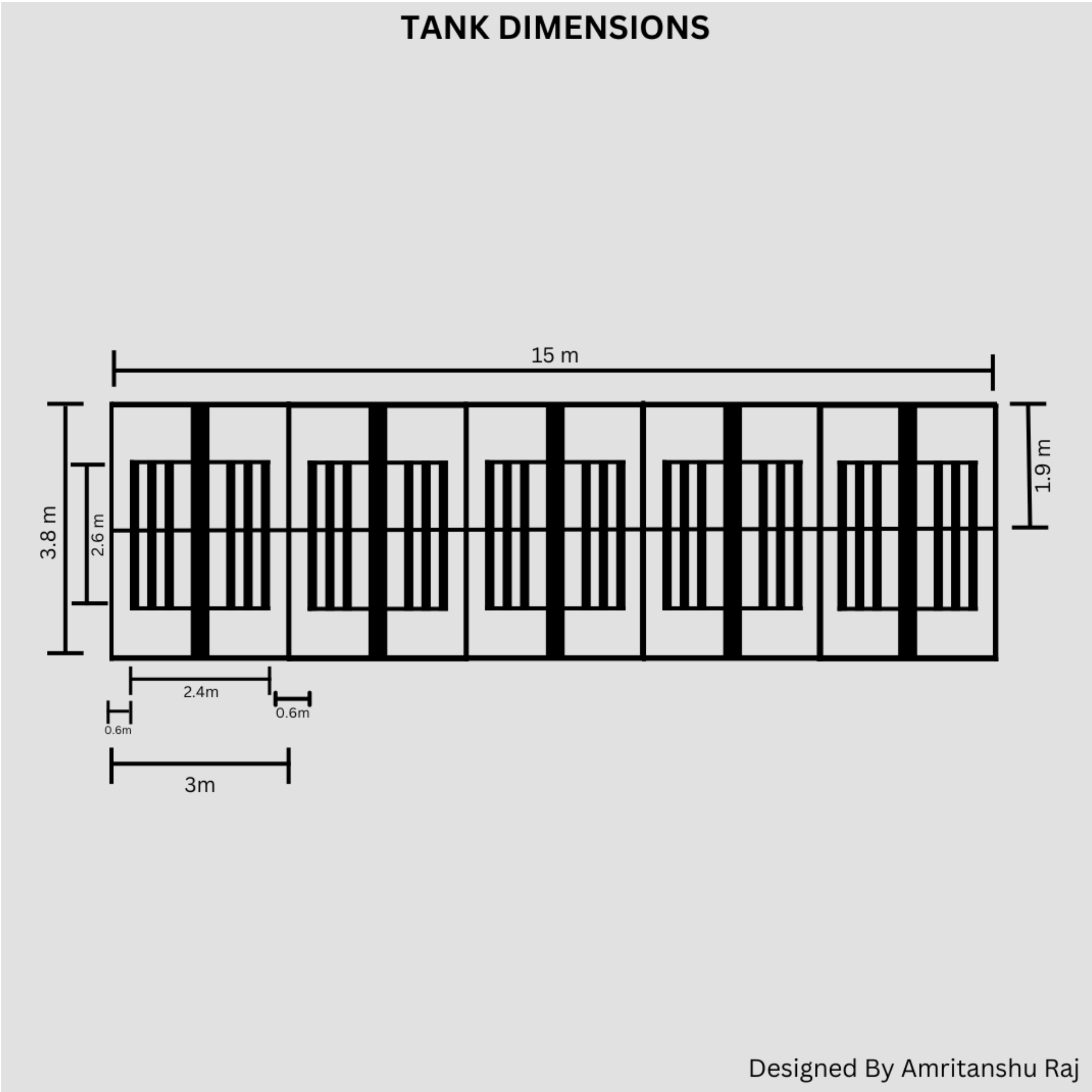
Width = 6.33 /3 = 2.11m

TANK DIMENSIONS



TANK CONFIGURATIONS

Assuming a clear cover of 0.6 m between two wheels of compartment



Power in each Compartment

For first compartment G_1 = 50/s

For second compartment G_2 = 40/s

For third compartment G_3 = 30/s

For fourth compartment G_3 = 20/s

For fifth compartment G_3 = 10/s

Volume = 316.6284375 m^3

Volume in each stage Vs = 316.6284375 / 5 = 63.3256875 m^3 = 63.33 m^3

At T = 20°C, Dynamic Viscosity $\mu = 1.0016 \times 10^{-3}$ N.s/m^2

We know, Power $P = G^2\mu V$

$P_1 = G_1^2\mu V = 50^2 x\ 1.0016 \times 10^{-3} x\ 63.33$= 158.5675215 Watt

P_1 = 158.5675215 Watt

$P_2 = G_2^2\mu V = 40^2 x\ 1.0016 \times 10^{-3} x\ 63.33$ = 101.4832138 Watt

P_2 = 101.4832138 Watt

$P_3 = G_3^2\mu V = 30^2 x\ 1.0016 \times 10^{-3} x\ 63.33$ = 57.08430774 Watt

P_3 = 57.08430774 Watt

$P_4 = G^2\mu V = 20^2 x\ 1.0016 \times 10^{-3} x\ 63.33$ = 25.37080344 Watt

P_4 = 25.37080344 Watt

$P_5 = G_5^2\mu V = 10^2 x\ 1.0016 \times 10^{-3} x\ 63.33$ = 6.34270086 Watt

P_5 = 6.34270086 Watt

Paddle Design

$P = (C_d \times A_p \times \rho \times V_p^3) / 2$

For first compartment:

P_1 = 158.5675215 Watt

Cd = 1.8 for flat blades

ρ = 1000 kg/m3

Assume actual speed of paddle V_p' = 0.67 m/s

$V_p = V_p' \times 0.75$ =0.5 m/s

Assuming width of paddle board as w

A_p = 2 wheels per compartment x 12 boards per wheel x length of paddle board x width of paddle board

$= 2 \times 12 \times L \times w = 2 \times 12 \times 1.30 \times w = 31.2w\ m^2$

$A_p = 31.2\ m^2$

$A_p = (P_1 \times 2) / (Cd \times \rho \times V_p^3)$

$= (158.5675215 \times 2) / (1.8 \times 1000 \times 0.5^3) = 1.388556418\ m^2$

w = 1.39/ 31.2 = 0.066 rev/s   w = 4.00 rev/min

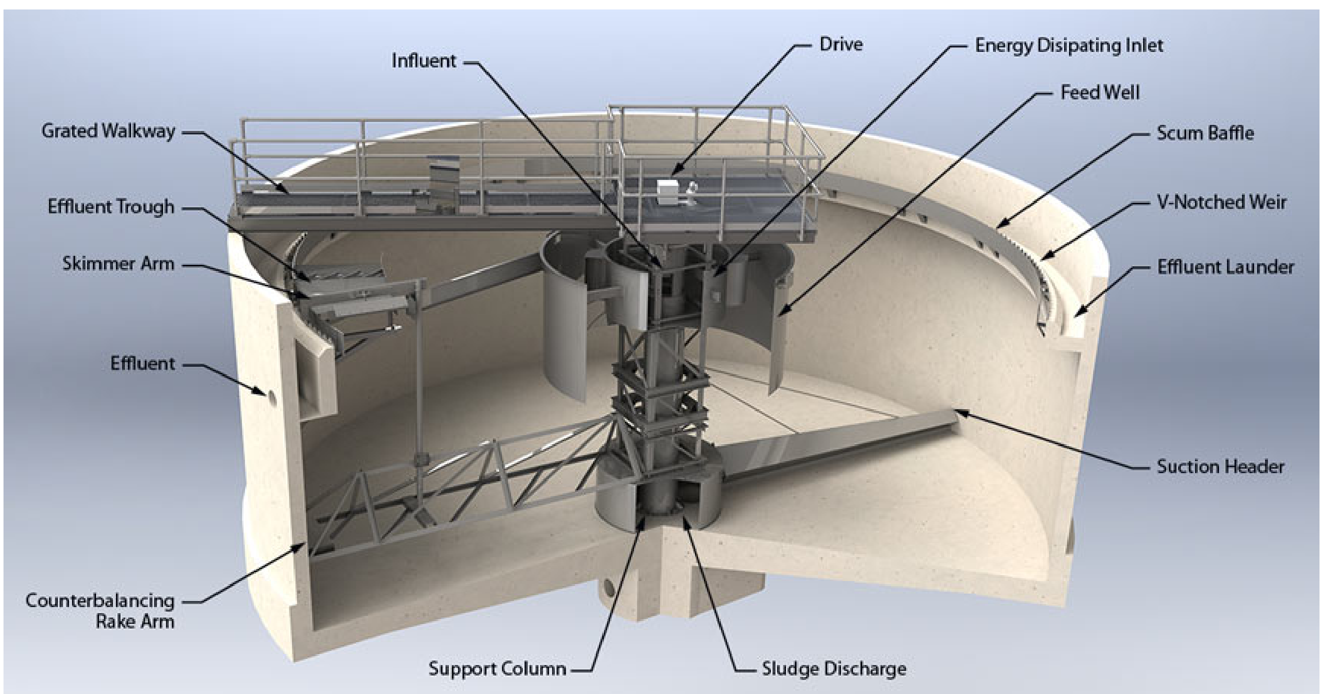
<u>For Second, third, fourth, fifth compartment</u>

Calculations had been given in the attached spreadsheet.

CLARIFIER

Clarifier tank is optional as we have already provided a sedimentation tank, but if it is required then sedimentation tank designs can be reused.

Details:



FILTRATION :

Filtration, the process in which solid particles in a liquid or gaseous fluid are removed by the use of a filter medium that permits the fluid to pass through but retains the solid particles.

DESIGN :

Rapid sand Filter:

Q = 18.237798 MLD = 18.24 x 10^6 liter/day = 759.90825 m^3/hr

We will design 4 filter tanks. Same design for all 4 tanks

So, Q for each tank = 189.9770625 m^3/hr

Assumptions:

1) Dynamic Viscosity $\mu = 1.0016 \times 10^{-3}$ $N.s/m^2$ (T = 20°C)

2) Filter Medium - Uniform Sand of depth 0.67 m

3) Filtering Velocity V_a = 5 m/hr

4) Depth of particles dp= 0.6mm

5) Specific Gravity G = 2.65

6) Porosity e = 0.4

7) Shape factor ψ = 0.85

8) backwash water required for backwash = 5 %

9) Time for washing the filter = 1 hr

1) Area of filter:

Design flow rate = (Q + 5% Q) m^3/hr = 199.4759156 m^3/hr

Design flow rate/day = (Q + 5% Q) x 24

It is assumed that 1 h is lost every day in washing the filter

Q = (Q + 5% Q) x 24/23 m^3/hr = 208.1487815 m^3/hr

$Q = V_a \times A$

=> $A = Q / V_a$ = 208.1487815/5 = 41.6297563 m^2

Assuming L/B = 1.25 (of a rectangular filter)

$A_f = L \times B = 1.25B^2 =$ 41.6297563 m^2

B = 5.7709449 m (5.8m)   L = 7.213681126 m (7.25 m)

2) Frictional Head Loss :

$H_f = (f' \times L \times (1-e) \times V_a^2) / (e^3 \times g \times dp)$
$f' = 1.75 + (150 \times (1-e)) / Re$
$Re = (\psi \times \rho \times V_s \times dp) / \mu$ $(V_s = V_a = 1.4 \times 10^{-3} m/s)$
$Re = (0.85 \times 1000 \times 1.4 \times 10^{-3} \times 0.6 \times 10^{-3}) / (1.0016 \times 10^{-3}) = 0.70 < 1$

$\Rightarrow f' = 1.75 + (150 \times (1-0.4)) /0.70 = 129.0121176$

$\Rightarrow H_f = (129 \times 0.67 \times (1-0.4) \times (1.4 \times 10^{-3})^2) /(0.4^3 \times 9.81 \times 0.6 \times 10^{-3})$

Hf = 265.57 mm

Minimum Thickness for sand bed given by Hudson Formula

$l = V_a \times dp^3 \times H_f/B_i \times 29323 = 5 \times 2 \times 0.6^3 \times 0.265 / 4 \times 10^{-4} \times 29323$

l = 0.058m < 0.67m   hence ok

3) Estimation of gravel and size gradation

Assume a size gradation of 2 mm at top to 40 mm at the bottom. The depth of gravel layer can be determined by using empirical formula

l = 2.54 k log d

k =constant varies from 10 to 14

Size (mm)	2	5	10	20	40
Depth (cm)	9.2	21.3	30.5	40	49
Increment (cm)	9.2	12.1	9.2	9.5	9

4) Design of under drainage system:

Plain area of filter = 7.213681126 m x 5.7709449 m

Assumption: Total area of the perforations in all laterals should be 0.5% of the filter area

Total area of perforations =0.5% x 41.6297563 m^2 = 2081.487815 cm^2

Total cross-sectional area of laterals = 3 x Total area of perforations = 6244.463446 cm^2

Area of manifold ($A = \pi D^2 / 4$)= 2 x area of laterals

Diameter of central manifold = 126.1327119 cm = 127 cm

Number of laterals (assume the distance between the laterals = 15 cm) = 7.21 m x 100 (cm/m) / 15 cm= 49 (for two sides = 98)

So, cross sectional area of one lateral = 6244.463446/ 98 =31.85950738 cm^2

Diameter of lateral = ($A = \pi D^2 / 4$)= 6.370663993 cm

Length of lateral = 1⁄2 (width of filter – diameter of manifold)

= 1⁄2 (5.7709449 – 1.27) = 2.25 m

Check Length of each lateral / Diameter of lateral < 60

(2.25/0.0623) = 36 < 60

Hence ok

<u>4) Design of back water trough:</u>

Let us assume that the rate of washing of the filter be 60 cm rise/ minute or 0.60 m/minute

The plan area of the filter = 41.6297563 m^2

Wash water discharge = 0.60m/min x 60 min/h x 41.6297563

= 1498.671227 m^3/hr = 0.416297563 m^3/sec

Assume a spacing of 1.6 m for wash water trough which will run parallel

No. of trough = 5.77 / 1.6 = 4
Discharge per unit trough = 0.1040743908 m^3/sec

The dimension of a concrete V- bottom trough can be designed by using the empirical formula $Q = 1.376\, b\, h^{3/2}$

Q = discharge in m3/s (0.1040743908 m^3/sec)

b = width of trough in m

h = water depth in the trough in m

Assume b = 0.4 m; h = 0.33 m

Assume freeboard of 0.1 m, total depth = 0.4 m

Total depth of filter = Depth for under drains + gravel + sand + water depth + free board

= 0.9 + 0.5 + 0.6 + 1.2 + 0.4 = 3.6 m

However instead of going for sand as filter media, we may also opt for anthracite as Sand tends to have a higher uniformity coefficient and therefore, can clog more often when used in a singular media filter configuration. Sand particles tend to be more spherical in shape while anthracite is sharp and angular. Studies have shown that more effective back washes occur with more angular structures.

6) DISINFECTION :

Disinfection describes a process that eliminates many or all pathogenic microorganisms, except bacterial spores, on inanimate objects (Tables 1 and 2). In health-care settings, objects usually are disinfected by liquid chemicals or wet pasteurization.

Chlorination, ozone, ultraviolet light, and chloramines are primary methods for disinfection.

We will go with chlorine disinfection.

DESIGN :

Assuming Chlorine Dosage = 0.3 mg/L = 0.3×10^{-3} kg/m3

=> C x Q = $0.3 \times 10^{-3} \times 18237.798 = 5.4713394$ kg/day

Consumed chlorine = 5.5 kg/day

The end