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Design of Wastewater Treatment System for Ugwuoba Bbattoir and Settlement at OJI River Local Government Area in Enugu State, Nigeria

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Abstract -- The discharge of untreated abattoir wastewater and domestic waste to the stream can have adverse effect on the environment and human health. This study was carried out to design a wastewater treatment system for Ugwuoba abattoir and the settlements around it. Population data was collected from the abattoir and the settlements which gave average design flow of 500.5m^3/d and peak design flow of 2002m^3/d. The designed wastewater system was in five stages from screen, aerated grit chamber, primary sedimentation, aeration basin for biological treatment and secondary clarifier. The dimensions obtained for each chamber was based on the design criteria for wastewater treatment system.

Indexed Terms: Abattoir, Wastewater, Treatment, Design

I. INTRODUCTION

Water is one of the most abundant natural resources to man. The importance of water cannot be over-emphasized as there is no substitute to it in many of its uses. It is essential for everything to grow and prosper in. Rivers are primary sources of potable water for mankind all over the world. They are sources of food as millions of tons of edible fish, lobsters, crabs and other aquatic animals and plants are taken from them [7]. They are used for the generation of power, recreational activities and for irrigation amongst others. In spite of the important role of water, water bodies are constantly being polluted through various activities, one of which is human activity. In Nigeria, environmental problems increase continually due to improper disposal of wastes. Abattoirs in the country have been generally known to dispose their wastes into surface water bodies without any prior treatment of the effluent. This impairs the surface of water bodies thus polluting the environment directly or indirectly. Similarly, the discharge of wastewater from bathroom, laundry has been used to explain the deterioration of most tropical rivers as they pass through inhabited places [8]. Wastewater from slaughter houses typically contains fat, grease, hair, feathers, flesh, manure, grit, undigested feeds, blood, bones which are characterized by high organic levels [3]. During processing of slaughtered animals, the animal blood is released untreated into the flowing stream while the undigested parts of the slaughtered animal are washed directly into the flowing water. As a result of these activities, the abattoir generates large quantities of biodegradable wastewater with high strength and complex composition which may elevate the pollution status of the receiving surface waters. Significant increase in biochemical oxygen demand and other nutrients could result in excess nutrient, which causes the water body to become choked with organic substances and organisms. The organic matter frequently exceeds the capacity of the micro-organism in water that breaks down and recycles the organic matter. This encourages the rapid growth and blooms of algae [4]. The improper disposal of wastes from slaughterhouses could lead to the transmission of pathogens to humans and can cause zoonotic disease such as coli bacillosis, salmonellosis, brucellosis and helminthes [2]. Due to the amount of wastewater discharged daily into Iheocha Stream from Ugwuoba abattoir and the consequences it posses to human health, environment and the economy, therefore the aim of this study is to design a wastewater treatment system for pre-treatment of wasterwater from Ugwuoba abattoir and the settlements around it.

II. DESCRIPTION OF THE STUDY AREA

Ugwuoba abattoir is located close to Iheocha stream. Iheocha stream drains from Enugu state to Mamu river, the river serves as a geographical dermacation between Enugu state and Anambra state. Ugwuoba abattoir is situated at Ugwuoba village in Enugu state. The employees at Ugwuoba abattoir are 42 in number. Average number of kill per day at Ugwuoba abattoir is about 12 - 15 cows. Fats, blood and tiny bones from the slaughter slab are washed down to stream while at some distance away from the slaughter slab, the processing takes place which generates soot from the roasted skin and ends up being emptied into the stream. Close to the slaughtering slab is a heap where paunch materials are dumped and have accumulated over the years. There are about 1388 of human population squating close to the abattoir whose source of water is from Iheocha stream, wastewaters generated from bathrooms, kitchens, restuarants and other activities are washed down to the stream. The mean annual rainfall at the area is 1450mm with a relative humudity of 80%. The mean annual temperature is about 22°C to 34°C. The land area around Iheocha stream is mostly of marshland. There are two main seasons the dry season (October-March) and rainy season (April-September).

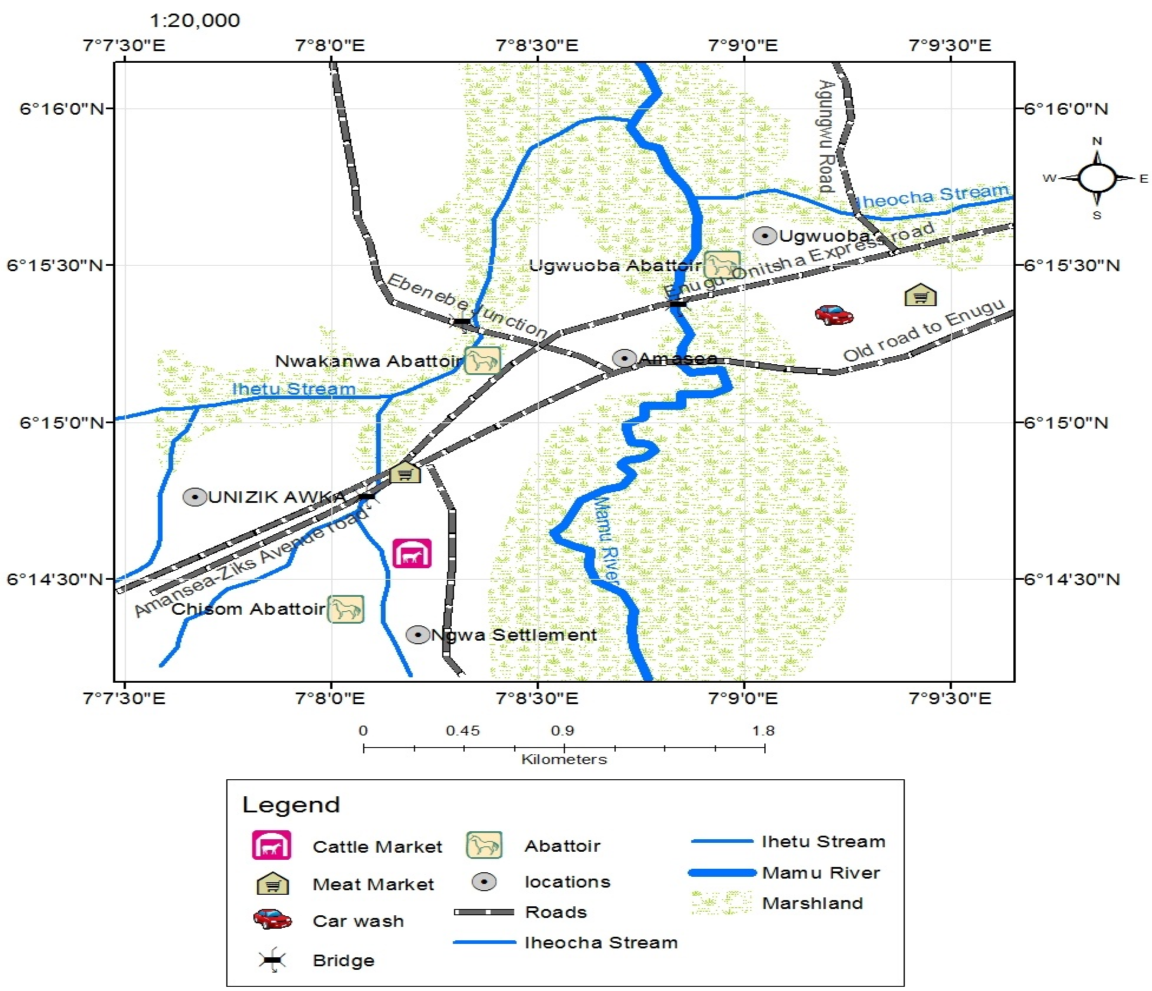

Fig 1.1 Map Showing Mamu River, Ihetu Stream, Iheocha Stream and two settlements

NGSA 2017

III. LITERATURE REVIEW

Treatment Processes

The preliminary treatment units include unit operations such as:

- Screen: it is the first unit operation used at wastewater treatment plant to remove large objects such as rags, paper, plastics, metals and etc. Screens are classified into three types, coarse screen (50-150mm) spacing, medium screen (20-

50mm) spacing and fine screen (5-20mm) spacing. Screen is used to prevent damage to pumping and sludge removal equipments. Screens can be manually or mechanically cleaned

- Grit Chamber: it is the second unit operation used to remove materials such as bone chips, seeds, eggshells and etc. grit is used in order to prevent mechanical equipments and pumps from unnecessary wear and abrasion. Prevent clogging in pipes, prevent cementing effects on the bottom of sludge digesters and primary sedimentation tanks, and etc. There are different types of grit removal; velocity controlled grit removal and aerated grit removal. Grit chamber can be cleaned manually or mechanically.
- Primary Treatment: it is the unit where primary sedimentation takes place. Its purpose is to remove settleable solids. Normally a primary sedimentation tank removes 50-70 percent total suspended solids and 30-40 percent biochemical oxygen demand. The design of sedimentation tank falls into three categories: horizontal flow, solid contact and inclined surface.
- Biological Treatment (Secondary Treatment): biological treatment involves bringing the active microbial growth in contact with wastewater so that they can consume the impurities as food. its remove soluble organics that escape the primary sedimentation and further remove suspended solids. Normally secondary treatment removes 85 percent of biochemical oxygen demand and suspended solids. Biological treatment has two processes, the suspended growth and the attached growth biological treatment.
- Sludge Collection: the floor of the rectangular and circular tanks is sloped toward the hopper. The slope helps in draining of the tank and to move the sludge. In mechanized sedimentation tank, the type of sludge collection varies with size and shape of the tank. In rectangular tank, the sludge collection equipment consist of a pair of endless conveyor chains running over sprockets attached to the staff or moving bridge sludge collectors having a scraper to push the sludge into the hopper. The sludge is removed from the hopper using pump.

IV. METHODOLOGY

- Field Work:

A preliminary visitation was done to obtain the population of employees in Ugwuoba abattoir and the settlements around it from the last ten years to the present year. Population data obtained was used to calculate the average daily flow from the abattoir and the settlement respectively. Water samples were collected from the stream to determine the biochemical oxygen demand and the total suspended solids.

- Population Analysis:

Population data was obtained from Ugwuoba abattoir and Ugwuoba settlements for the last ten years and the present year. The data obtained was used to project the future population using Arithmetical Increase method: $P_n = P + n.C$

Where: P_n is the population after "n" decades. P is the present population.

Year	Population	Increment
1998	0	-
2008	11	11
2018	42	31
		Average increment = 21

Table 1: Population Projection of Ugwuoba Abattoir

Future Population, $P_n = 42 + (21)5 = 147 persons$.

Year	Population	Increment
1998	0	-
2008	499	499
2018	1388	889
		Average increment = 694

Future Population for Ugwuoba settlements, $P_n = 1388 + (694)5 = 4858 persons$

Total Future Population = Future population of persons working in Ugwuoba abattoir + Future population of persons living in Ugwuoba settlements.

Therefore, $P_n = 147 + 4858 = 5005$ Persons.

Determination of Design Flowrate
The average flow per day was obtained by multiplying the supply of water per capita per day (100LPCD) with the total population.
Average flow,
$Q_{av} = 5005 persons \times 100 LPCD = 500500 LPCD = 500.5 m^3/d$
.
Peak flow was obtained by multipling the peaking factor (4.0 from peaking factor curve by Metcalf et al, 2003) with the average flow.
Peak flow,
$Q_{pek} = 4.0 \times 500.5 m^3/d = 2002 m^3/d.$

V. RESULTS AND DESIGN OF PLANT UNITS

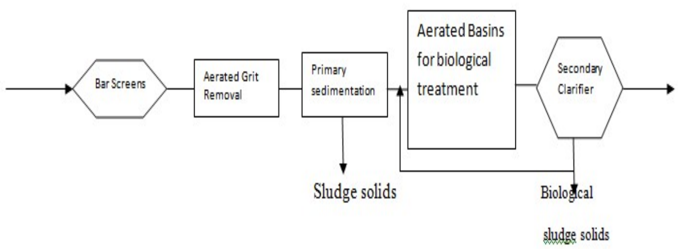

Fig. 2: Schematic Diagram for Wastewater Treatment System

a) Design Calculations for Screen:
Design Criteria Used

1. Velocity through rack at maximum flow = 0.8m/s
2. Velocity through the rack at minimum/average flow = 0.3m/s.
3. Assume manual cleaning and angle of inclination of bars with horizontal as 30°.
4. Assume size of bars 9mm x 50mm, 9mm facing the flow.
5. Clear spacing between bars = 30mm.
6. Average flow = 500.5m^3/day = 0.0058m^3/s
7. Max. flow = 2002m^3/day= 0.023 m^3/s

Design of Rack (Screen) Chamber
Net submerged area of the screen:
At average flow: Area = $\frac{average\ flow}{velocity\ through\ rack} = \frac{(0.0058) m^3/s}{(0.3) m/s} = 0.019 m^2$

At maximum flow: Area $= \frac{0.023 m/s}{0.8 m/s} = 0.029 m^2$.

Provide net submerged area of = 0.029m^2.
Gross submerged area of the screen:

Using 'n' number of bars as a ratio of opening to the gross width. (n) is 20 bars.
$\frac{(n+1)30}{(n+1)30+9(20)} = \frac{(20+1)30}{(21)30+9(20)} = 0.78 m^2$.

The gross submerged area of the screen = $\frac{0.029}{0.78} = 0.037 m^2$.

The submerged vertical cross sectional area of the screen = $0.037 m^2 \times \sin 30^0 = 0.019 m^2$

This is equal to cross-section area of the screen chamber, therefore velocity of flow in screen chamber = $\frac{0.023 m/s}{0.019 m^2} = 1.21 m/s$

Since the velocity through the screen is greater than the bar screen self clensing velocity 0.42m/s. the design is ok.

Provide 20 bars of 9mm x 50mm at 30mm spacing.
The gross width of screen = $(20 \times 0.009 m) + (21 \times 0.03 m) = 0.81 m$
Therefore the liquid depth at average flow = $\frac{0.019 m^2}{0.81 m} = 0.023 m$
Provide freeboard of 0.03m.

Total depth of screen = 0.03m + 0.023m = 0.053m
The size of the channel = 0.81m(width) x 0.023m(depth).
Bed slope
$R = \frac{A}{P} = \frac{(0.81 m \times 0.053 m)}{2(0.81 m + 0.053 m)} = \frac{0.043}{1.73} = 0.025 m$

But, $V = \left(\frac{1}{n}\right) R^{2/3} S^{1/2}$

$$S^{1/2} = \frac{V.n}{R^{2/3}} = \frac{1.21m/s \times 0.0012}{0.025^{2/3}} = 0.017$$

$S^{1/2}$ = 0.017.

The bed slope is nearly 1 in 59. (n is mannings rugosity constant ranging from 0.009-0.015).
Head Loss through the Clean Screen

The head loss through the clean screen is calculated

$$h_L = \beta \left(\frac{w}{b}\right)^{4/3} h_v \sin\theta$$

Where h_L = head loss (m). β = bar shape factor (2.42 for sharp-edged rectangular). w = maximuim width of the bar (m), (9mm). b = minimuim clear spacing of the bar (m),

(30mm). h_v = velocity head of the flow $\left(\frac{v^2}{2g}\right)$ approaching the rack (0.0746m). θ = angle of the rack with the horizontal, (30°).

$$h_L = 2.42 \left(\frac{0.009}{0.030}\right)^{1.333} (0.0746 \times \sin 30) = 0.0181$$

m = 18.1mm < 150mm
Head Loss through Half Clogged

The head loss through the screen when is half clogged is calculated

$$h_L = 0.0729(V^2 - v^2)$$

Where V = velocity through the screen (m/s). v = velocity before the screen (0.3m/s).

$h_L = 0.0729(0.8^2 - 0.3^2)$ = 0.040m = 40mm.

b) Design Calculations for Aerated Grit Chambers:
Design Criteria Used.

1. Provide two chambers to help in cleaning and maintenance.
2. Depth is 2m.
3. Width to depth ratio 1:1.1.
4. Air supply of 0.15m^3/min per in of length.
5. Detention time is 5mins.
6. Maximuim design flow through chamber = $= 0.023m^3/s$

Volume of each chamber for 5 minutes detention period = $\frac{Q \times detention\ time}{2}$ = (0.023)m^3/s x (5)min x (60)secs = 6.9m^3 = $\frac{6.9}{2}$ = 3.45m^3

Dimension of aeration basin:
Depth to width ratio is 1:1.1.
Since depth is 2m, width is 2 x 1.1 = 2.2m.
Length = $\frac{volume}{Area} = \frac{3.45}{2 \times 2.2} = 0.78m$

Increasing length by 20% for inlet and outlet conditions.
Total length = 0.78 x 1.2 = 0.94m.

Design for Air Supply Requirement
Provide air supply at 0.15m^3/min per meter length of the chamber.

Theoretical air required per chamber = 0.15m^3/min x 0.94m = 0.14m^3/min per chamber.

Quantity of Grit
Consider grit collection rate of 0.015m^3/10^{3}m^3
Volume of grit = 0.023 x 60 x 60 x 24 x 0.015 x 10^{-3} = 0.0298m^3/day

Check for surface overflow rate (SOR)
SOR in the grit = $\frac{0.023}{2.2 \times 0.78} = 0.013m/s$

SOR is less than settling velocity of smallest particle 0.024m/s. Design is ok.

c) Design Calculations for Primary Sedimentation Tanks:

Design Criteria Used

1. Overflow rate and detention time shall be based on average design flow of 500.5m^3/day.
2. The overflow rate shall be less than 35m^3/day at (average design flow).
3. The detention time shall not be less than 1.5hrs and not more than 2.5hrs.
4. The weir loading shall be less than 300m^3/day/m.
5. The liquid depth in the basin shall not be less than 2m.
6. Influent BOD$_5$ and TSS to the plant = 250mg/l and 260mg/l.

Design Calculations

Surface area of the tank = $\frac{500.5m^3/day}{35m^3/m^2.day} = 14.3m^2$

Check for peak flow:

SOR at peak flow = $\frac{500.5m^3/day x 4}{13.90} = \frac{2002\,m^3/d}{14.3m^2} = 140m^3/m^2.day$

This is less than the recommended peak flow.

Assume wide of basin = 2.8m.

Theoretical length = $\frac{14.3}{2.8} = 5.11m.$

Total length of tank = 5.11m + 2m (inlet) + 2m (outlet) = 9.11m

Using detention time of 1.5hrs.

Depth of basin = $\frac{500.5\times 1.5}{24\times 14.3} = 2.2m.$

Flow through velocity = $\frac{0.0058m^3/s}{2.2m\times 2.8m} = 0.00094m/s.$

Flow through velocity < 1cm/s, therefore design is ok.

Flow through velocity at peak flow = $\frac{0.023\,m^3/s}{2.2m\times 2.8m} = 0.00037m/s$.

Provide total depth = 2.2m + 0.5m freebord + 0.25 space for sludge = 2.95m

Weir loading rate = $\frac{500.5m^3/d\times 4}{2.8m\times 2} = 357.5\,m^3/m.d$

Length of weir = $\frac{500.5m^3/d}{357.5m^3/m.d} = 1.4m$.

d) Design Calculation for Biological Reactor/Aeration Basin:

Design Criteria Used

1. Provide complete mix activated sludge process using diffused aeration system.
2. The effluent shall have BOD_5 and TSS of 20mg/l or less.
3. Two basins are provided.
4. The assumed biological kinetic coefficients are; mean cell residence of solids in aeration basin θ_c = 10day. Yield coefficient over finite period of log growth, Y = 0.5mg/mg. concentration of MLVSS in aeration basin, X = 3000mg/l. endogenous decay coefficient, $K_d = 0.05\ d^{-1}$
5. BOD_5 for the effluent (SS) = 0.63.
6. Influent BOD_5 and TSS = (200 and 150) respectively.
7. Maximum flow = 2002m^3/day

Design Calculations for Aeration Basins.

The Concentration of Soluble in the Effluent.

BOD_5 exerted by the solids in the effuent = 20mg/l x 0.63 = 12.6mg/l.

Soluble portion of BOD_5 in the effluent = (20 – 12.6) mg/l = 7.4mg/l.

Treatment Efficiency of biological Treatment.

Efficiency = $\frac{(200-7.4)mg/l}{200} \times 100 = 96 percent.$

Determination of Reactor Volume.

$$XV = \frac{YQ\theta_c(S_o-S)}{1+k_d\theta_c} = \frac{0.5\times 2002000l/d\times 10d\times(200-7.4)mg/l}{1+0.05\times 10d} = 1.285\times 10^9 mg$$

But, $XV = 1.285\times 10^9 mg$.

Therefore,

$$V = \frac{1.285\times 10^9 mg}{X} = \frac{1.285\times 10^9 mg}{3000mg/l} = 428428 litres = 428.4m^3$$

Dimensions of Aeration Basin.

Provide two rectangular aeration basins.

Volume for each basin = $\frac{428.4m^3}{2} = 214.2m^3$

Provide water depth of 3.0m.

Provide freeboard = 0.8m.

Total depth = 3 + 0.8 = 3.8m.

Surface Area for each basin = $\frac{214.2m^3}{3m} = 71.4m^2$

Provide length to width ratio = 2:1. Area = $2w^2$.
Width of basin = 6m.
Length of basin = 2 x 6 = 12m.
Hydraulic detention Time

$$Detention\ time = \frac{volume}{Q} = \frac{428.4m^3 \times 24}{2002\,m^3} = 5.1 = 5hrs$$

Computation of sludge to be wasted, P_x

$$\frac{\Delta x}{\Delta t} = \frac{XV}{\theta_c} = \frac{1.285 \times 10^9 mg}{10} = 1.28 \times 10^8 mg/d$$

$$P_x = 128.5kg/d$$

The above microbial mass production represents volatile solids (VSS) in the tank.

Assuming SS contains 70 percent volatile matter.
Therefore, SS production/sludge growth = $\frac{128.5kg/d}{0.7} = 183.6kg/day.$

The underflow solids concentration = 15000mg/l (SS).

$$Q_w = \frac{183.6 x 10^6/d}{15000mg/l} = 12240l/d = 12.24m^3/d$$

Q_w for each basin = $\frac{12.24m^3/day}{2} = 6.12m^3/day$.

Determination of recirculation flow, Q_r.

$$Q_r.X_r = (Q + Q_r)X$$

$$Q_r = \frac{Q.X}{(X_r - X)} = \frac{(3000mg/l)\times(2002\,m^3/l)}{(15000-3000)mg/l} = 500.5m^3/day$$

(for all basins).

Q_r for each basin = $\frac{500.5m^3/l}{2} = 250.3m^3/day$.

Therefore, $\frac{Q_r}{Q} = \frac{500.5m^3/d}{2002m^3/d} = 0.25, ok.$

Calculation of Oxygen Requirement.

O_2 demand = $1.47(S_o - S) \times Q - 1.42P_x$

Q = $2002m^3/d$ = $2.0 \times 10^6 l/d$.

$$S_o - S = (200 - 7.4) = 192.6mg/l.$$

$$P_x = 1.28 \times 10^8 mg/d.$$

Therefore, O_2 demand = $1.47(192.6mg/l) \times 2.0 \times 10^6 l/d - 1.42 \times 1.28 \times 10^8 mg/d$

= 3.84×10^8 mg/d = 384.5kg/d.

Volume of air required:

Assuming air weight is $1.2kg/m^3$ and contains 23.2 percent oxygen weight.

Theoretical air = $\frac{O_2\,demand}{0.232 \times 1.2kg/m^3} = \frac{384.5kg/day}{0.232\times 1.2kg/m^3} = 1381m^3/day$

Assuming oxygen transfer efficiency of porous tube diffuses as 8%.

Actual volume of air = $\frac{1381m^3/day}{0.08} = 17262m^3/day$. $17262m^3$/day per basin.

e) Design Calculations for Secondary Clarifiers: Design Criteria Used

1. Two circular clarifiers are provided.

2. Clarifiers are designed based on average design flow plus the recirculation.
3. The overflow rate at peak flow conditions shall not exceed $40m^3/m^2/day$.

Design Calculations

Surface Area of Secondary Clarifier = $Q_{pek} + Q_r + Q_w$
= $2002m^3/day + 500.5m^3/day + 12.24m^3/day$
= $0.0291m^3/sec$

Design flow for each secondary clarifier = $\frac{0.0291\,m^3/sec}{2} = 0.0145m^3/sec$

Assume SOR = 15m/day

Area = $\frac{Q}{SOR} = \frac{0.0145\,m^3/sec}{0.000174\,m^3/sec} = 83.64m^3$

Diameter of the secondary clarifier = $\sqrt{\frac{83.64}{\pi}} = 2.92m$

Provide two clarifiers each of (2.92m) diameters.

Check the overflow rate at peak design flow = $\frac{Q}{Area} = \frac{2002\,m^3/day}{83.64} = 23.94 \Rightarrow ok.$

Depth of Secondary Clarifier
Provide average side water depth = 2.5m.
Freeboard = 0.5m.
Total depth = 2.5m + 0.5m = 3.0m.
Detention Time

Peak volume of the clarifier = $\frac{\pi}{2} \times (2.92^2)m \times 3.0m = 125.46m^3$

Detention time for peak design flow = $\frac{peak\ volume}{design\ flow} = \frac{125.46m^3}{0.023\,m^3/sec}$

$$= 5454.78m^3/sec = 1.5hrs = 2hrs$$

VI. DISCUSSION AND CONCLUSION

Design of wastewater treatment system is highly influenced by the population density, and population growth. The designed wastewater system is for pre-treatment of wastewater before discharge. The system removes 85 percent of biochemical oxygen demand and suspended solids. Further advance treatment can be done to remove other constituents order than BOD and TSS. It is recommended that pre-treatment of wastewater should be adopted by abattoir owners before being discharged into the stream.

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