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Design of Sewage Treatment Plant in Hospital

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ABSTRACT: This project outlines the design of a sewage treatment plant (STP) for CM Hospital Pandalam, with a designated capacity of 0.943 MLD. The STP will utilize a biological treatment process to effectively remove pollutants from the hospital wastewater. The design incorporates essential components to ensure efficient treatment, guaranteeing the treated effluent adheres to the stringent guidelines established by the Central Pollution Control Board (CPCB). This enables the treated wastewater to be reused for non-potable applications such as irrigation or landscaping, promoting water conservation efforts in the region. The STP also contributes to responsible sewage disposal, minimizing environmental impact. In addition to the process description, the project also encompasses AutoCAD designs for the STP layout.

KEYWORDS: Total Dissolved Solids, Biological Oxygen Demand, Chemical Oxygen Demand, Dissolved Solids

I. INTRODUCTION

The design of a sewage treatment plant is a crucial task that ensures proper disposal of wastewater and safeguards the environment from pollution. This project aims to design a sewage treatment plant for CM Hospital Pandalam that is cost-effective, reliable, and sustainable, meeting local authorities' standards and regulations and the Central Pollution Control Board. The project involves analyzing site conditions, treatment technologies, and operational and maintenance aspects. The design phase includes selecting and sizing various components of the sewage treatment plant, such as screens, grit chambers, primary and secondary clarifiers, biological reactors, sludge handling units, and disinfection units. The project also considers environmental and social impacts such as the reuse of treated effluent for irrigation, disposal of sludge as fertilizer, and enhancement of the aesthetic and sanitary conditions of the town. This project's benefits include preventing waterborne diseases, conserving water resources, reducing greenhouse gas emissions, and improving the residents' quality of life. This project aims to demonstrate the feasibility and importance of designing a sewage treatment plant for a small town in India, where the lack of proper sanitation and wastewater management is a major challenge.

II. METHODOLOGY

2.1 - Quantitative analysis is the process of examining the physical, chemical, and biological properties of sewage. It includes parameters such as pH, temperature, turbidity, total solids, suspended solids, dissolved solids, biochemical oxygen demand (BOD), chemical oxygen demand (COD), dissolved oxygen (DO), nitrogen, phosphorus, etc. These parameters determine the quality and strength of the sewage and the degree of treatment it will need.

2.2 - Qualitative analysis involves assessing the physical, chemical, and biological characteristics of sewage to understand its composition and quality. This helps plant operators determine the appropriate treatment processes needed to remove contaminants and pollutants effectively before discharging the treated water back into the environment. Qualitative analysis includes methods such as visual inspection, odour assessment, and laboratory testing to identify the presence of various substances and pathogens in the sewage.

2.3 - Physical treatment is the process of removing the coarse and settleable solids from the sewage using physical methods such as screening, grit removal, and primary sedimentation. These units reduce the organic load and protect the downstream treatment units from clogging and abrasion.



2.4 - Chemical treatment involves adding chemicals to the sewage to enhance the removal of pollutants such as phosphorus, metals, and pathogens. Chemical treatment can also be used to adjust the pH, coagulate and flocculate the suspended solids, disinfect the effluent, etc. Chemicals such as lime, alum, ferric chloride, and chlorine are commonly used.

2.5 - Comparison with standards involves comparing the quality of the treated effluent with the standards or regulations prescribed by the authorities for the disposal or reuse of the effluent. Standards vary depending on the intended use of the effluent, such as irrigation, industrial, recreational, etc. The standards specify the permissible limits of various parameters such as BOD, COD, suspended solids, nitrogen, phosphorus, coliforms, etc.

2.6 - Design of particular treatments involves selecting and designing the appropriate treatment units for the removal of specific pollutants from the sewage, such as nitrogen, phosphorus, pathogens, etc. The selection and design depend on the influent quality, effluent standards, site conditions, operational and maintenance requirements, etc.

2.7 - Cost estimation and materials required involves estimating the capital and operating costs of the STP, as well as the materials required for the construction and operation of the STP. The costs and materials depend on the type and size of the treatment units, the land area, labor, energy, chemicals, sludge disposal, etc.

2.8 - Layout preparation involves preparing the layout or plan of the STP, showing the location and arrangement of the treatment units, the piping and pumping system, the electrical and instrumentation system, the sludge handling and disposal system, the access roads, the fencing, the landscaping, etc. The layout should consider the hydraulic gradient, the ease of operation and maintenance, aesthetics, safety, and environmental impact.

III. LOCATION OVERVIEW

LOCATION: CM HOSPITAL PANDALAM

COORDINATES: 9°13'06"N 76°40'44"E

The area was calculated using the google earth engine and the approximate area is 20,000 m² (4.9 acres)

IV. QUANTITATIVE ANALYSIS

4.1 POPULATION FORECAST

Population forecasting method used here is geometric increase method

$$P_t = P_o \left(1 + \frac{r}{100}\right)^n$$

$$P_{2054} = 375 \times \left(1 + \frac{87.5}{100}\right)^3 \\ = 2471.9 \sim 2472$$

4.2 CALCULATION OF SEWAGE GENERATION

Design Period = 30 Years

Forecasted Population after 30 years = 2471.9

Per capita water supply = 450 litres per head

Average water supply per day = 450 × 2471.9
= 1115595 LPCD = 1.11 MLD

Average sewage generation = 85% of supplied water
= 0.85 × 1.11 = 0.9435 MLD
= 943.5 KLD

Average sewage per hour = $\frac{943.5}{24} = 39.31 \text{ m}^3/\text{hr}$
= $\frac{39.31}{(60+60)} = 0.010 \text{ m}^3/\text{sec}$



V. QULITATIVE ANALYSIS

Characteristics of waste water such as pH, TSS, BOD, Oil & Grease, chloride and Nitrite and results are;

Table -1: Characteristics of waste water

Sl. No	Parameters	Unit	Wastewater
1	Ph	-	6.96
2	Total suspended solids (TSS)	mg/l	212
3	BOD	mg/l	8.9
4	COD	mg/l	228
5	Oil & grease	mg/l	7.12
6	Chloride	mg/l	191.9
7	Nitrite	mg/l	1.23

VI. DESIGN OF SEWAGE TREATMENT PLANT

6.1 DESIGN OF SCREENING

Maximum Flow = 0.9435MLD
 = 0.0109m³

Shape of bars = M S bars

Size = 10×50mm² (10mm facing flow)

Clear spacing = 30mm (6 – 40mm for medium screens)

Velocity of flow = 0.8m/sec (0.6 – 0.9m/sec)

Hence, net submerged area of screen opening = $\frac{0.0109}{0.8}$
 = 0.013625m²

Gross vertical area required = 0.01365sin60
 = 0.01179m²

Providing a depth of 0.3m

Width of the channel = $\frac{0.01179}{0.3}$ = 0.039m

The velocity of flow in screen chamber = $\frac{\text{maximum flow}}{\text{depth} \times \text{width of channel}}$
 = $\frac{0.0109}{0.3 \times 0.0393}$ = 0.92451m/sec

Which is greater than 0.42m/sec (self-cleansing velocity)

FiWith spacing of 30mm

Assume 30 bars are provided

Actual width of screens = 30×0.04
 = 1.2m

Actual depth of sewage flow = $\frac{0.01179}{1.2}$
 = 0.0098m

Providing a free board of 0.5m

Total depth of screen = 0.0098+0.5
 = 0.5098m ≈ 0.51m

Hence, size of screen = 1.2m×0.51m

6.2 DESIGN OF GRIT CHAMBER

To prevent scouring of already deposited particles the magnitude of ‘v’ should not exceed critical horizontal velocity v_c , and the above equation becomes

$$\frac{L}{h} = \frac{v_c}{v_o}$$

The critical velocity, v_c , can be given by the following equation

$$v_c = \sqrt{\frac{8g}{f} \beta (g_s - 1) d}$$

Where;

B = constant

= 0.04 for Uni granular sand

= 0.06 for non-uniform sticky material

f = Darcy-Weisbach friction factor = 0.03 for gritty matter g, gravitational acceleration

G_s = Specific gravity of the particle to be removed (2.65 for sand)

d = Diameter of the particle, m

$$v_c = \sqrt{\frac{8 \times 9.81 \times 0.06 (2.65 - 1) \times 0.2}{0.03 \times 1000}}$$

= 0.228m/sec (usually between 0.17-0.26m/sec)

The grit chamber is designed to remove the smallest particle of size 0.2mm with specific gravity around 2.65. For these particles, using above expression the critical velocity comes out to be, $v_c = 0.228$ m/sec

Keeping horizontal velocity 0.2m/sec (<0.228m/sec) to prevent scouring and detention time 40 sec (vary from 40-60sec)

To lower the velocity, hydraulic structures like Sutro weir and Parshall flume should be provided.

Length of grit chamber = velocity of flow \times detention time

= $0.2 \times 40 = 8$ m

Volume of grit chamber = discharge \times detention time

$$= 0.0109 \times 40$$

$$= 0.436 \text{m}^3$$

Cross sectional area of flow = $\frac{\text{volume}}{\text{length}} = \frac{0.436}{8}$

$$= 0.0545 \text{m}^2$$

Providing width of grit chamber = 0.5m

Depth of grit chamber = $\frac{0.0545}{0.5}$

$$= 0.109 \text{m}$$

Now assuming grit generation = 0.05m^3 per 1000m^3 of sewage

Even though grit is continuously raked, still 8 hours grit storage is provided for average flow

Storage volume required = $\frac{943.8 \times 8 \times 0.05}{(24 \times 1000)}$

$$= 0.015 \text{m}^3$$

Thus, grit storage depth = $\frac{0.015}{0.0545} = 0.27$ m

Also providing free board = 0.5m

Total depth of grit chamber = $0.5 + 0.27 + 0.109$

$$= 0.879 \approx 0.88 \text{m}$$

Hence, size of grit chamber = **8m \times 0.5m \times 0.88m**

6.3 DESIGN OF SKIMMING TANK

Detention time = 3 to 5 minutes

Compressed air required = 300 to 6000m^3 per million litres

Surface area required, A = $\frac{0.00622 \times q}{v_r}$

Where;

q = Rate of flow of sewage in m^3/day

v_r = Minimum rising velocity of greasy material to be removed in m/minute = 0.25m/minute in most cases.

$$A = \frac{0.00622 \times 943.5}{0.25} = 23.47 \text{m}^2$$

Providing a square tank

Side of tank = 4.8m

Hence provide a skimming tank of size = $4.8 \times 4.8 \text{m}^2$

Actual area of tank = 4.8×4.8

$$= 23.04 \text{m}^2$$

6.4 PRIMARY SEDIMENTATION TANK

Continuous flow tank is to be provided

Detention time = 2 hrs (1-2hrs)

Quantity of sewage treated in 2hrs

$$V = \frac{943.5 \times 2}{24} = 78.625 \text{m}^3$$

Now, assuming that the flow velocity through the tank is 0.3m/minute

Assuming the water depth in the tank (ie. effective depth of tank) = 5m

Assuming length of tank = 5.5m

$$\begin{aligned} \text{Cross-sectional area of the tank required} &= \frac{\text{capacity of tank}}{\text{length of the tank}} \\ &= \frac{78.625}{5.5} = 14.29 \text{m}^2 \end{aligned}$$

$$\begin{aligned} \text{The width of the tank required} &= \frac{\text{area of c/s}}{\text{depth}} \\ &= \frac{14.29}{5} = 2.85 \text{m} \end{aligned}$$

Since the tank is provided with mechanical cleaning arrangement, no space at bottom is required for sludge zone.

Now, assuming free board of 0.5m

The overall depth of the tank = 5+0.5 = 5.5m

Hence, a rectangular sedimentation tank with an overall size of $5.5 \times 2.85 \times 5.5 \text{m}^3$ can be used.

6.5 DESIGN OF AERATION TANK

Average flow = 943.5×10^3

BOD at inlet = 8.9 mg/l

Total applied BOD = $8.9 \times 943.5 \times 10^3$
= 8.4 kg/day

BOD loading rate = $3 \text{kg/m}^3/\text{day}$

Volume of aeration tank = $\frac{8.4}{3.2} = 2.625 \text{m}^3$

Assuming media filling factor = 0.5

Media volume = $2.65 \times 0.5 = 1.3125 \text{m}^3$

Hence, Aeration tank dimensions are $2\text{m} \times 2\text{m} \times 2\text{m}$

6.6 AERATOR SIZING

BOD applied to tank = 8.9mg/l

Average flow in tank = $943.5 \text{m}^3/\text{day}$

BOD to be removed in tank = $\frac{8.9 \times 943.5 \times 1000}{1000000}$
= 8.39kg/day
= 0.34kg/hr

Oxygen requirement = 1kg/kg BOD applied

Peak oxygen demand = 125%

Oxygen transfer capacity of the aerator in standard conditions = $1.9 \text{kg/kWh} = 1.41 \text{kg/HP/hr}$

Oxygen transfer capacity of aerators at field conditions
= $0.7 \times 1.41 = 0.98 \text{kg/HP/hr}$

Oxygen to be applied in tank = $1 \times 0.34 \times 1.25$
= 0.425kg

HP of aerator required = $\frac{0.425}{0.98} = 0.43 \text{HP}$

Thus, provide 2 generators of 0.21 HP each, with 1 generator at standby

6.7 DESIGN OF SECONDARY CLARIFIER

No of secondary clarifier = 1 No.

Average flow = $943.5 \text{m}^3/\text{day}$

Recirculated flow, assuming 50% = $471.75 \text{m}^3/\text{day}$



Total inflow = 1415.25m³/day

Provide hydraulic retention time = 2hrs

Volume of tank = $\frac{1415.25 \times 2}{24} = 117.93\text{m}^3$

Assume liquid depth = 5m

The area is calculated as;

$$\text{Area} = \frac{\text{volume}}{\text{depth}}$$

$$= \frac{117.93}{5} = 23.58\text{m}^2$$

Surface loading rate of average flow = 30m³/m²/hr (25-35m³/m²/hr)

$$\text{Surface area to be provided} = \frac{943.5}{30}$$

$$= 31.45\text{m}^2$$

Taking area whichever is higher = 31.45m²

Diameter of circular tank (d), $A = \frac{\pi}{4} d^2 = 31.45\text{m}^2$

d = 6.32m

Check for weir loading,

Average flow = 943.5m³/day

Recirculated flow = 471.75m³/day

Mixed liquor suspended solids in tank = 2500mg/l

$$\text{Thus, total solid in inflow} = \frac{1415.25 \times 2500 \times 31.45}{1000000}$$

$$= 111.27\text{kg/day}$$

$$\text{Solid loading} = \frac{111.27}{1000}$$

$$= 0.111\text{kg/day/m}^2$$

Providing a free board of 0.5m

Hence, **size of secondary clarifier = diameter of 6.23m with an overall depth of 5.5m**

6.8 DESIGN OF SLUDGE DIGESTION TANK

Average flow = 0.9435MLD

Total suspended solids = 212mg/l

$$\text{Mass of suspended solids in 0.9435MLD of sewage flowing per day} = \frac{0.9435 \times 10^6 \times 212}{10^6}$$

$$= 200.22\text{kg/day}$$

Assuming, that the fresh sludge has moisture content of 95% (95-98%) means 5kg of solids will make = 100 kg of wet sludge

$$= 130.01 \text{ will make}$$

$$= \frac{100 \times 130.01}{5}$$

$$= 2600.2\text{kg/day of wet sludge}$$

Assuming specific gravity of wet sludge as = 1.015

$$v_1 = \frac{2600.2}{1.015}$$

$$= 2.56\text{m}^3/\text{day}$$

The volume of digested sludge produced per day at 75% moisture content is given by formula,

$$V_2 = \frac{V_1(100-P_1)}{(100-P)}$$

$$= \frac{2.56(100-95)}{(100-75)}$$

$$= 0.512\text{m}^3/\text{day}$$

Now, assuming the digestion period as 30 days,

$$\text{Capacity of digestion tank required} = V_1 t - 0.67(V_1 - V_2)t$$

$$= 2.56 \times 30 - 0.67(2.56 - 0.512) \times 30$$

$$= 35.64\text{m}^3$$

Now, providing 5.5m depth of the cylindrical digestion tank

$$\text{Cross-sectional area of the tank} = \frac{35.64}{5.5} = 6.48\text{m}^2$$

$$\text{Area of the tank} = \frac{\pi}{4} d^2 = 6.48\text{m}^2$$

$d = 2.87\text{m}$

Hence, provide a cylindrical sludge digestion tank 5.5m deep and 2.87m diameter, with an additional hopper bottom of 1:1 slope for collection of digested sludge.

6.9 ESTIMATION OF GAS PRODUCED FROM DIGESTER TANK

Total suspended solids in sewage = 212mg/l

Assuming that 60% of suspended are removed in the primary sedimentation tank, we have

The suspended solids removed as sludge = 60% of 212mg/l
= 127.2mg/l

Now, assuming that the volatile solids present are 70% of the suspended solids, we have

Volatile solids removed = 70% of 127.5
= 89.04mg/l

Further, assuming that the volatile matter is reduced by 65% sludge digestion.

Volatile matter reduced = 65% of 89.04mg/l
= 57.876mg/l

Hence, Volatile matter reduced in 943.5 cum of sewage = $\frac{57.876 \times 943.5 \times 1000}{1000000}$
= 546.06m³

Now, assuming that 0.9m³ of gas is produced per kg of volatile matter reduced,

Total quantity of gas produced = 0.9 × 546.06 = 491.45m³

Assuming that the produced gas contains 65% methane and 30% CO₂,

Methane produced = 0.65 × 491.45
= 319.44m³

CO₂ produced = 0.30 × 491.45 = 147.43m³

Now, assuming that the methane in the sludge gas has a fuel value of 36000kJ/m³

The fuel value = 36000 × 319.44
= 11.49MkJ

Now, assuming a boiler efficiency of 80%, the amount of heat that can be furnished by the boiler = 80% of 11.49 = 9.192 Million kJ

= $\frac{9.192}{4.12}$ = 2.2. Million kilo calories

The gas collected may be utilized for operating engines, and for heating sludge to promote quick digestion.

6.10 DESIGN OF FLOCCULATION TANK

Average hourly flow = $\frac{943.5}{(24 \times 60)}$ = 0.655m³/minute

Detention time ranges between 10 to 30 minutes

Let us take detention time = 10 minutes

Volume of flocculation tank = 10 × 0.655 = 65.5m³

Assuming depth of flocculation tank as 1.5m and length of flocculation tank = breadth of flocculation tank

Hence, size of flocculation tank = 6.6m × 6.6m × 1.5m

6.11 DESIGN OF SECONDARY SEDIMENTATION TANK

Average flow = $\frac{943.5}{24}$ = 39.312m³/hr

Surface flows rate = 15m³/hr/m²

Surface area = $\frac{39.312}{15}$ = 2.62m²

Assume depth, D = 2.5

Assuming length: breadth ratio as 2:1

Hence, size of secondary sedimentation tank = 2.28m × 1.14m × 2.5m

6.12 CHLORINE DOSAGE

Assuming chlorine dosage of 30 ppm

Amount of chlorine required to disinfect 39310 litre/hr = 39310×30
 = 1179300mg/hr

Amount of chlorine solution required = 1179300mg/hr
 = $\frac{1179300}{150000} = 7.86$ litres/hr

Chlorine dosage per day = $7.86 \times 24 = 188.64$ litres/day

6.13 DESIGN OF FILTER FEED TANK

Hourly flow rate = $\frac{943.5}{24} = 39.31 \text{m}^3/\text{hr}$

Retention time = 3hr

Volume of filter feed tank = $39.31 \times 3 = 177.93 \text{m}^3$

Assume height = 3m

Area required = $\frac{177.93}{3} = 39.31 \text{m}^2$

Assume L: B = 1:1

Size of filter feed tank = $6.26 \text{m} \times 6.26 \text{m} \times 3 \text{m}$

6.14 DESIGN OF PRESSURE SAND FILTER

Average hourly flow rate = $39.31 \text{m}^3/\text{hr}$

Assume loading rate = $12 \text{m}^3/\text{m}^2/\text{hr}$

Area = $\frac{39.31}{12} = 3.27 \text{m}^2$

Area = $\frac{\pi}{4} d^2 = 3.27 \text{m}^2$

d = 2.04m

Size of pressure sand filter = 2.04m diameter and 2.5m height

6.14 DESIGN OF ACTIVATED CARBON FILTER

Average hourly flow rate = $39.31 \text{m}^3/\text{hr}$

Assume loading rate = $12 \text{m}^3/\text{m}^2/\text{hr}$

Area = $\frac{39.31}{12} = 3.27 \text{m}^2$

Area = $\frac{\pi}{4} d^2 = 3.27 \text{m}^2$

d = 2.04m

Size of activated carbon filter = 2.04m diameter and 2.5m height

6.14 DESIGN OF ACTIVATED CARBON FILTER

Average hourly flow rate = $39.31 \text{m}^3/\text{hr}$

Retention time = 3hr

Volume of treated water tank = $39.31 \times 3 = 117.3 \text{m}^3$

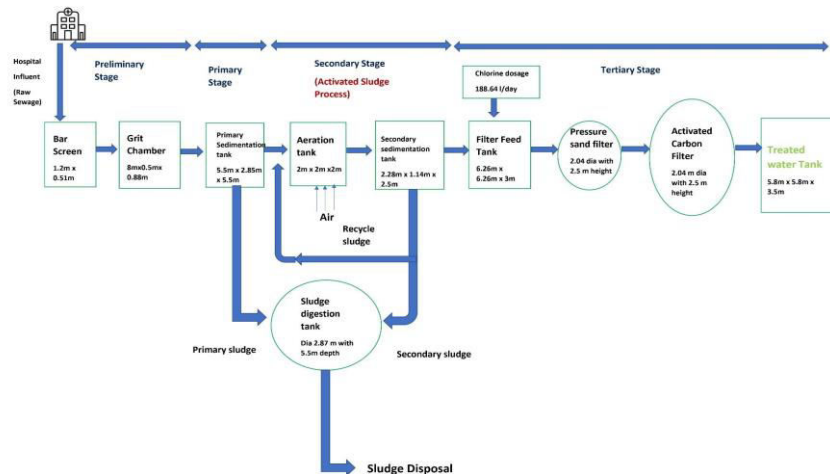
Assuming height = 3.5m

Area required = $\frac{117.93}{3.5} = 33.69 \text{m}^2$

Assuming L: B = 1:1

Size of treated water tank = $5.8 \text{m} \times 5.8 \text{m} \times 3.5 \text{m}$

VII. LAYOUT OF SEWAGE TREATMENT PLANT



VIII. CONCLUSION

The sewage treatment plant project in C M Hospital, Pandalam is aimed at providing a sustainable and cost-effective solution for managing domestic wastewater in the town. The project involves several steps, including assessing the existing sewage system and wastewater generation in the area, selecting the most suitable location, capacity, and technology for the plant, and designing the MBBR-based sewage treatment plant. By implementing this project, the town can address the challenges of rapid urbanization, population growth, and inadequate sanitation facilities. Additionally, this project contributes to national and global goals of achieving clean water and sanitation for all while mitigating the impacts of climate change and enhancing the resilience of communities.

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