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Design of Sewage Treatment Plant for CBN Housing Estate Trans-Ekulu Enugu Nigeria

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Authors' contributions

This work was carried out in collaboration between all authors. Author OPC designed the study, performed the analysis, wrote the protocol, wrote the first draft of the manuscript and managed the literature searches. Authors OBS, CJL and UDC managed the analyses of the study, literature searches and improved the final manuscript. All authors read and approved the final manuscript.

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ABSTRACT

CBN quarters Trans-Ekulu, Enugu has been upgraded to Housing Estate status, the steady increment in the Estate population results to the increase in domestic sewage generation. Presently there is no sewage treatment plant, so, it is required to construct a sewage treatment system with sufficient capacity to treat the increased sewage generation. The project deals with the design of the sewage treatment plant and its major units such as inlet chamber, grit chamber, comminutor, primary settling tank, trickling filter, secondary sedimentation tank, sludge digester and sludge drying bed for the Housing Estate. It also involves the sizing of each components of the treatment plant. The project takes into cognizance the housing estate size in land mass, number of housing units, residents' population and finally it is designed to serve the housing estate for the next 30 years as the residents' population increases. CBN Housing Estate Enugu is a residential estate and

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is at a distance of 7 km North East of 82 DIV. Enugu and 5 km south of School of Dentistry, Enugu. With regards to the housing Estate, almost the entire area and environment are plain and the general slope is from West to East. The estate is located at the latitude of N06° 28.669' (N06.48°) and longitude of E 007° 29.808' (N007.50°). The soil of the area is gravel and a large proportion of sandy-gravel. All the aspects of the Estate's climate, and topography, its population growth rate are will all considered while designing the project. By the execution of the project, the entire sewage of the Housing Estate can be treated effectively and efficiently.

Keywords: Sewage; treatment; housing; tank; analysis; design; wastewater; sludge.

1. INTRODUCTION

The need for adequate sewage treatment system is a global problem and has great impact on individuals, households, families, physical and biological environment. The steady increase in population results in the increase of domestic sewage generation. Thus, no treatment plants for the Housing Estate. Proper waste management has been universally accepted as one of the essential human need for a clean and healthy environment. However, many researchers believe that much has to be done in the mechanism of domestic sewage treatment [1].

Since the rapid increase in the population of housing estate occupants which result in the increase of sewage generation, the liquid water will require treatment before they are discharged into the water body or otherwise disposed off without endangering the public health or causing offensive conditions. The collection of waste water from occupied areas and conveying them to some point of disposal requires a mechanism for the treatment [2].

[3] stated that the purpose of a sewage collection system is to remove wastewater from points of origin to a treatment facility or place of disposal. The collection system consists of the sewers (pipes and conduits) and plumbing necessary to convey sewage from the point(s) of origin to the treatment system or place of disposal. It is necessary that the collection system be designed so that the sewage will reach the treatment system as soon as possible after entering the sewer. If the length of time in the sewers is too long, the sewage will be anaerobic when it reaches the treatment facilities.

In the past, the trend has been to design the most efficient unit processes, each, at a lowest cost and then combine the units to form an optimum wastewater treatment system. [4] conducted design studies of the activated sludge subsystem (aeration tank and secondary clarifier). The system provided excellent method of treating either raw sewage or more generally, the settled sewage. It offers secondary treatment with minimum area requirement, and an effluent of high quality is obtained. Though normally, it is found that for towns or small cities or estates (like CBN Housing Estate, Enugu) with medium sized plants, trickling filters are better; whereas in big cities with large sized plants, the activated sludge plant is better.

[5] analyzed the settling of particles using the ideal basin concept. He assumed that; the direction of flow is horizontal in uniform velocity throughout the settling zone, the concentration of suspended particles is uniform over depth at the inlet of the settling zone, and Particles reaching the bottom remain discrete. His work demonstrated that the efficiency of sedimentation is governed by the surface area measured parallel to the direction of flow. [5] concluded that the efficiency of primary sedimentation basin is independent of the basin depth but dependent on overflow rate. They have also proposed that for optimum efficiency, settling tanks should be long, narrow (minimize the effect of inlet and outlet disturbances, cross winds, density currents and longitudinal mixing) and relatively shallow. [5] did not consider flocculation in his analysis.

Most wastewater contains both soluble and particulate organic and inorganic matter. [6] Proposed that domestic wastewater contains more organic carbon in colloidal and suspended form than the dissolved form. [6] Also found that particulate fraction is 66% to 83% organic and contributes 58% and 63% of volatile solids for domestic wastewater. He also found that the ratio of Chemical Oxygen Demand (COD) to volatile solids for the particulate fraction is approximately 1.5 : 1.0 while for the soluble fraction varies from 0.6 : 0.8 to 1.0.

The aim of this paper is to develop a low cost design procedure for wastewater treatment systems, which will generally allow domestic effluents to be disposed of without danger to human health or unacceptable damage to the natural environment, satisfy a set of specified constraints, and minimize life time costs. Life time cost includes capital, operation and maintenance costs.

To realize this aim, the following specific objectives were pursued: Physical, chemical and biological treatment of the domestic sewage from CBN housing Estate Enugu Nigeria, Provide treatment at a minimal cost while satisfying specific requirements, to attain a total discounted cost at the lowest possible level while satisfying a set of constraints (these constraints include: a specified effluent quality, and various physical & biological constraints), Design of the sewage treatment plant, and also to set out a model for further subsequent designs of STP for cities and Estates.

This paper is concerned with the design of a sewage treatment plant for CBN Housing Estate, Trans-Ekulu Enugu Nigeria. The scope is limited to the design of the plant and its components, no construction of the plant or production of prototype will be made. The data used in this work were collected from the occupants of the Estate, and local Estate attendants.

This study developed a least cost design procedure for wastewater treatment systems, which will generally allow domestic effluents to be disposed of without danger to human health or unacceptable damage to the natural environment.

2. DESIGN ANALYSIS

2.1 Design Elements

It is common practice to control sewage treatment plant by reference to matters such as waste disposal, site selection, protection of surface waters and impact on neighborhood amenity. Accordingly, sewage treatment plants need to be located in areas remote from residential development with sufficient available land for sustainable wastewater reuse. Proper design and construction will ensure effective wastewater reuse procedures and can be managed on a sustainable basis.

2.2 Site Analysis

The research team's several field visits to the CBN Quarter Estate Enugu Nigeria availed them the leverage to agree that distance of the sewage water treatment will be 1.5 km away from the residential buildings. The exact location of the site in the estate as measured with the hand held GPS equipment was given as: Latitude; N: 06° 28.668$^{/}$, Longitude; E: 007° 29.808$^{/}$; Elevation; 210 m.

From the contour map of the estate studied by the team in Fig. 2, observations were made which include the following: The land formation is skewed (sharp slope), There is a running stream at the foot of the slope, and existence of the vegetations along the bank of the stream.

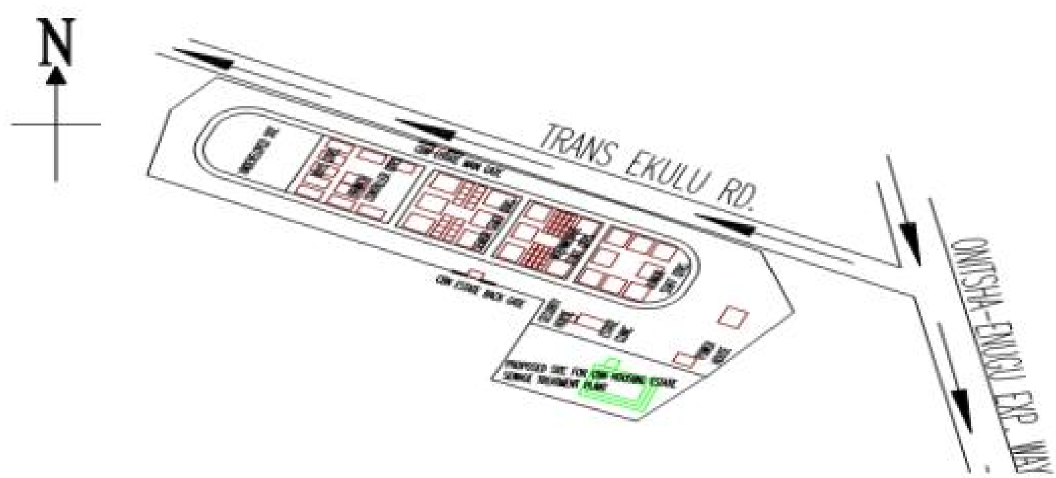

Fig. 1. The plan layout of CBN housing Estate Trans Ekulu, Enugu Nigeria

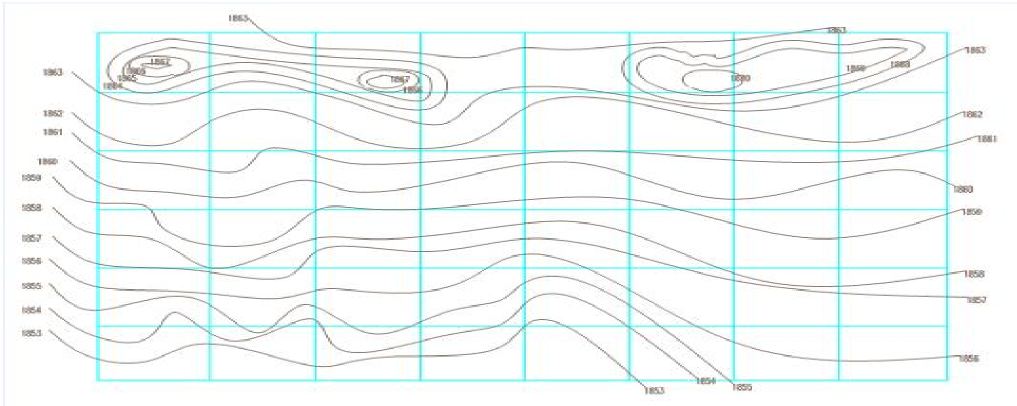

Fig. 2. Part of CBN housing estate layout contour

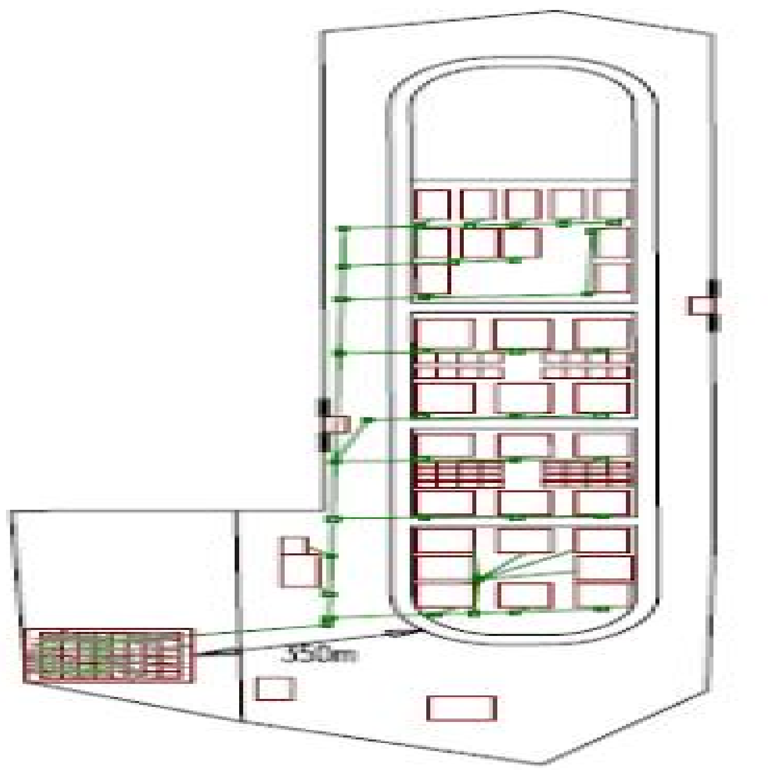

Fig. 3. Proposed sewer pipe network for CBN housing estate

Due to this skewed nature of the estate land form, the sewage water treatment plant was considered, sited so that most of the effluent flow to the settling tank will be by gravity and hence pump work will be reduced. The utility lines were observed in the site plan of the estate run in alignment with the footing of the estate's perimeter fence. Hence it is easier by that to carry out excavation work without tampering them. The already existing sewage conduit piping network makes for easy connection to the supply pipe to the treatment plant.

2.3 Geotechnical Investigation

2.3.1 Field work

Five (5) test borings were dug, to depth ranging from (0.1 - 0.5 m), soil samples were taken intervals. After these, the soil samples were taken to the laboratory for determination of the required parameters with respect to soil properties.

2.3.2 Laboratory testing

Laboratory classification tests were carried out on the undisturbed and disturbed samples obtained from the boreholes to improve on field identification and classification tests. The tests carried out include: Moisture Content Determination (MCD), Atterberg Limit Tests (ALT), Particle Size Distribution Tests (PSTD), pH value of Water in Soils, Sulphate Content of Water in Soils, Bulk density, Specific Gravity; (SG), Undrained Triaxial Compression Test, and Consolidation (Odometer) test.

2.3.3 Analysis of geotechnique

The Geotechnical properties of the soils encountered at the various strata formation of the overburden were obtained from the tests conducted in laboratory. The summary of the results are given in Table 1.

2.3.4 Bearing capacity analysis

The Allowable bearing pressure imposed on a foundation is a function of characteristics of the shear strength of the soil as well as the depth and dimensions of the foundation. The bearing capacities for selected boring locations were based on the SPT N_{30} value obtained from the Standard Penetration Test field results and the laboratory strength properties of the recovered samples. However, the ultimate bearing capacity values given in Table below are deduced from SPT N_{30}-values.

2.4 Estate Population

The population of the estate was gotten to be two thousand (2,000) persons. Allowing for 25% safety factor mark up in the estate's population will make the total population to be; $1.25x2000 = 2500\, persons$. Hence, the population's peak factor is determined by the formula,

$$PF = 14P^{-(1/6)} \tag{2.1}$$

Where; P = the population served, PF = 14 x $(2500)^{-(1/6)}$ = 3.8L/s.

2.5 Design Configuration

The CBN Housing Estate Trans Ekulu, Enugu Nigeria is treated as a small town based on population size. Hence, the sewage treatment plant that will be befitting to its inhabitants is that of single stage configuration (See Fig. 4).

2.6 Designs of the Various Parts That Make Up the Sewage Treatment

2.6.1 Design parameters

Estate Area = 100 hectares, Population = 2500 person, Peak factor = 3.8, Rate of water supply = 300 liters per capital per day.

2.6.2 Design calculation for the discharge in sewer line

Time of concentration = 50 minutes, Average impermeability coefficient for the entire area = 0.3, this sewer line will be designed for a flow equivalent to the Wet, Weather flow (W.W.F) plus twice the dry weather flow (D.W.F).

Assume that the sewage flow is equal to 80% of rate of water supply.

Hence sewage flow (D.W.F.) = 0.8 x 300 = 240 litres/capital day = $\frac{2500\ x\ 240}{24\ x\ 60\ x\ 60}$ = 6.94 litre/sec

The rainfall intensity is given by,

$$R_i = \frac{25.4a}{t+b} \tag{2.2}$$

Table 1. Summary of geotechnical properties of the soil

S/N	Property	Minimum	Maximum
1	Natural moisture content (%)	6	13
2	Liquid limit (%)	NP	NP
3	Plastic limit (%)	NP	NP
4	Plasticity index (%)	NP	NP
5	Passing # 200 Sieve (%)	1.34	52.85
6	Bulk density (KN/m^3)	15.85	18.10
7	Apparent cohesion (KN/m^2)	0	0
8	Angle of internal friction (Ø)	17	32
9	Coefficient at compressibility (m^2/KN)	-	-
10	Specific gravity	2.55	2.74

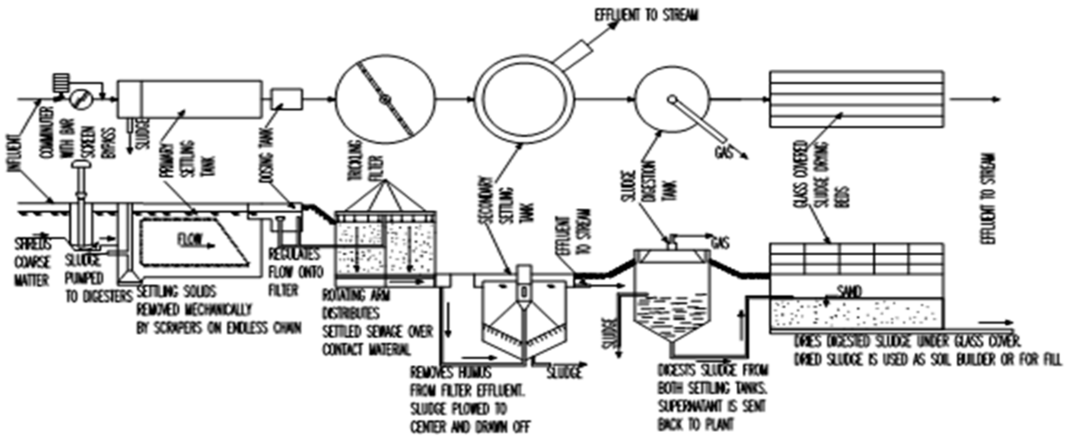

Fig. 4. Proposed sewage treatment plant configuration for CBN housing estate, Enugu

Where

t = 50 min; a = 40; b = 20; $R_i = 14.5\, mm/hr =$ 1.45 cm/hr

The W.W.F. is given by,

$$Q = 28\text{A.I. } R_i \tag{2.3}$$

Q = 28 x 100 x 0.3 (1.45) = 1218 litre/sec

Hence, design discharge

$$Q = 2\,(\text{D.W.F}) + \text{W.W.F} \tag{2.4}$$

Q = 2 x (6.94) + 1218 = 1231.88 liters/se, Ratio of DWF and WWF = $\frac{6.94}{1218} = \frac{1}{1.75.5}$

Since this ratio is very large, it is preferable not to use a combined sewer system.

Table 2. Bearing capacity values

Depth (m)	Bearing capacity values (kN/m^2)	
	BH1	**BH2**
0.0 - 0.05	40	80
0.1 – 0.2	320	100
0.27 –0.35	450	350
0.4 – 0.5	1000	620
	EB	EB

BH – Bore Hole and EB – End of Boring

2.6.3 Hydraulic design of sewers

The sewage, to be transported through the sewers, is mostly liquid (water), containing hardly (0.1 to 0.2%) of solid matter in the form of organic matter, sediments and materials. Hence, the general approach for the design of sewers is similar to the design of water mains. However, there are things to be considering in this design

1. Pressure of solid matters: This sewage flowing through the sewers contains particles of solid matters (both organic as well as inorganic). These solid particles settle at the bottom and have to be dragged during the sewage transport. In order that the sewers are not clogged, they are to be laid at such a gradient that self cleansing velocity is achieved, at all value of discharges. Also the inner surfaces of the sewer must be resistant to the abrasive action of the solid particles.
2. Pressure: Sewers may be considered as open channels in most cases, wherein, the sewage runs under gravity. The sewer should run full, and the hydraulic gradient line falls within the sewer. Hence, the sewer must be laid at continuous downward gradient. Sewers run under pressure only when they are designed as force mains and inverted siphons. Hence, consider the design calculations below:

Rugosity coefficient (Asbestos cement for plastic smooth conduit material), N = 0.011

The sewer is to be laid at a slope,

$$S = 1 \text{ in } 500 = \left(S = \frac{1}{500}\right) \tag{2.5}$$

Design based on a sewer running 0.8 times full at maximum discharge, Water supplied = 2500 x 300 = 750000 liters/day = $\frac{750000}{24\,x\,3600\,x\,1000} = 8.68\,x10^{-3}$ m^3/s.

Assuming that 80% of the water supplied to the Estate appear as sewage, then average discharge in the sewer = 0.8 x (8.68 $\times 10^{-3}$) = 6.944 $\times 10^{-3}$ m^3/s.

At a peak factor of 3.8; Maximum discharge = 3.8 x 6.944 $\times 10^{-3}$ = 0.0264 cumecs

Since the sewer is to be designed as running 0.8 times the full depth, $\frac{d}{D} = 0.8\ and$ Q_{max} = 0.0264 cumecs

For a sewer running partially full, consider the Fig. 5 circular sewer running partially full

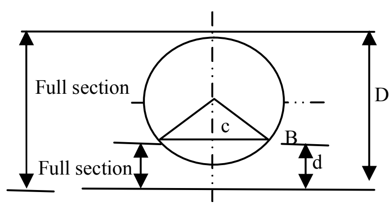

Fig. 5. Circular sewer running partially full

Where d = depth at partial flow, θ = central angle subtended as shown, D = internal diameter of circular sewer

Therefore,

$$\text{proportional depth} = \frac{d}{D} = \frac{1}{2}\left(1 - \cos\frac{\theta}{2}\right) \quad (2.6)$$

(Where; Sin θ = 0.960)

Area,

$$a = \frac{\pi}{4}D^2\left[\frac{\theta}{3600} - \frac{sin\theta}{2\pi}\right]d = 0.6736D^2 \quad (2.7)$$

Wetted perimeter,

$$P = \pi D\ \frac{\theta}{360} = 2.2143D \quad (2.8)$$

Hydraulic mean depth (HMD): $r = \frac{a}{p} = \frac{0.6736\text{D2}}{2.2143\text{D}}$ = 0.3042D

Where,

$$q = \frac{\text{I}}{N}\, ar\ ^2/_3\ \text{S}^1/_2 \quad (2.9)$$

Therefore; D = $(0.0213)^{3/8}$ = 0.236 m

Taking a markup of 6% of D = 1.06 x 236 = 250.16 mm

Also checking for self cleansing velocity at maximum discharge, r = 0.0718 m

$$\text{Velocity, } V = \frac{\text{I}}{N}\, r^{2/3}\, 5^{1/2} \quad (2.10)$$

V = 0.7023 mls (= 70.28 cm/s)

Checking for self cleaning velocity at minimum discharge, Assume minimum flow = 5/19 times the average flow: q_{min} = 1.8274 x 10^{-3} cumecs, Q_{max} = 0.0264, $\frac{qmin}{Q} = 0.069$.

Interpolating for the corresponding value of $\frac{q}{Q}$ = 0.069 for $\frac{Vmin}{V}$, from the table of Hydraulic elements of circular sewers running partially full: Let the value of $\frac{Vmin}{V}$ = x_v = 0.716, X_v = 0.401 + 0.153 = 0.554; V_{min} = 0.554 V = 0.389 m/s

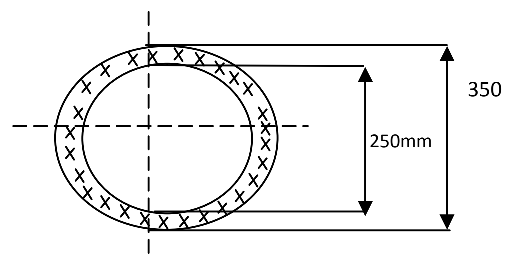

Fig. 6. Hydraulic sewers

2.6.4 Design calculation for structural requirement for sewer pipe

Pipe type = Asbestos cement pipe, Pipe diameter = 250 mm, to be laid in 1.5 deep trench of 0.6 m width. Assuming that the total vertical load will account for concentrated, Surcharge of 5t applied at the centre of the pipe.

Assume a type of bedding having load factor of 2.8 (from table of load factor for supporting strength in treach condition). Using a factor of safety of 1.5 for the saturated top soil take unit weight, γ = 2000 kg/m^2 and Kμ' = 0.150. Considering water load also, assuming the sewer to run 80% full. The three edge bearing strength for 250 mm diameter. Asbestos cement pipe is 4320 kg/m, Thickness of Asbestos cement pipe of 250 mm diameter = 50mm, Bc = 350 mm = 0.35 m, H = 1.5 – 0.35 = 1.15m; Bd = 0.6 m, $\frac{\text{H}}{Bd} = \frac{1.15}{0.6} = 1.92.$

The load coefficient for trench conduit is given by the equation

$$C_d = \left[\frac{1-e^{-2k\mu'(H/Bd)}}{2K\mu}\right] \quad (2.11)$$

$C_d = 1.46$, We = $C_d \gamma B_d^2$ = 1051.2 kg/m

Weight of water,

$$W_w = \left[\frac{\pi}{4}\ D^2 x L\right] W \times 0.8 \quad (2.12)$$

Were L = 1 m, W = 1000 kg/m^3, W_w = 39.37 kg/m, $\frac{L}{2H} = 0.4$; $\frac{Bc}{2H} = 0.152$

From table of values of load coefficient, C_s through the following parameters $\frac{Bc}{2H}$ = 0.2 and $\frac{L}{2H}$ = 0.4, Cs = 0.131 taking an impact factor of 1.5 and Lc = 1 m

$$W_{sc} = \frac{Cs \times Ie \times P}{Lc} \quad (2.13)$$

Were P = 5t, t =1000 kg, P =5000 kg, W_{sc} = 982.5 kg/m

Total W = Wc + Ww + Wsc = 2073.07 kg/m

Safe supporting strength of 250 mm diameter pipe = Three edge bearing strength x L_f

= 4320 x 2.8 = 12096 kglm

Since the actual load (2073.07 kg/m) does not exceed the safe supporting strength, the pipe is safe.

2.6.5 Design of the inlet/receiving chamber

The receiving chamber is where the effluent is received first before pumping it into the grit chamber. This is more or less like the cesspool in its structure.

Estate population = 2500, Water supply =300 litres per capital/day.

Assuming 80% of water supplied to the estate is converted into sewage.

Total sewage flow = 0.8 x 2500 x 300 = 600000 litres/day

Assume no detention period; Capacity required = $\frac{600000}{24\, x\, 1000}$ = 25 m^3

Assume an overflow rate of 30 m^3/d/m^2, Surface area = $\frac{600000}{30\, x\, 1000}$ 20m^2 = B x L =20 m^2

Taking L = 2B, B(2B) = 20; B = 3.2, L = 2B = 2(2.24) = 6.4 m.

Effective depth of tank = $\frac{25}{20} = 1.25\, m$, dimensions will be 6.4 m x 3.2 m x 1.25 m.

2.6.6 Design of the sewage pump

The centrifugal pumps are most widely used for pumping sewage and storm water, as these can easily be installed in pits and sumps and can easily transport the suspended matter present in the sewage without getting clogged too often. These pumps work on the principle of centrifugal force. They essentially consist of two main parts: (i) The casing and (ii) The impeller rotates with high speed inside the casing. The commonly used horizontal axial flow type pumps are fitted with either open or closed three-vane type impeller. The clearance between the vanes is kept large enough to allow any solid entering the pump to pass out with the liquid, thus preventing the clogging. See the design calculation for sewage pumping below: Peak sewage flow: Q_{max} = 0.0264 cumecs, Diameter of rising main: Assume a flow velocity of flow in rising main = 1 m/s, Area of cross-section = $\frac{Qmax}{V}$ = $\frac{0.0264}{1} = 0.026$ m^2, D = $\sqrt{\frac{0.026\, x\, 4}{\pi}} = 0.183\, m$, Provide a rising main of 18 cm diameter; Actual velocity of flow =

$$\frac{Qmax}{A} \quad (2.14)$$

Design of sump well: Sump will be designed for 2 hour low. Peak flow rate = 0.0264 cumecs; Quantity of sewage collection in 2 hours = 0.0264 x 2 60 x 60 = 190.08 m^3.

Assuming a separate sewer from the Estate enters the pumping station through a low level sewer at R. L. = 100 m. The same sewage will be pumped to a higher level sewer at R. L. = 115 m, Quantity of sewage in rising main = $\frac{\pi}{4}\ (0.18)^2 x 100 = 2.545$ m^3.

Total capacity of the sump well = 190.08 +2.545 = 192.63 m^3, 3 Sump wells will be provided, two for storing the above sewage and third as a standby. Let the depth of each unit = 3 m and Surface area of each unit = $\frac{192.63}{2\, x\, 3}$ = 32.105 m^2.

Diameter of sump well = $\sqrt{\frac{32.105\, x\, 4}{\pi}}$ = 6.4 m (Hence provide three units of sumps well, each of 6.4 m diameter and 3 m depth), Design of pumps: Each pump has to lift a sewage of $\frac{192.63}{2}$ = 96.315 m^3 in 2 hour, Capacity of each pump = $\frac{96.315}{2 \, x \, 60 \, x \, 60}$ = 0.0134 cumecs or 0.0134 m^3/s.

Assume Darcy's friction factor =

$$0.04: h_f = \frac{FLV^2}{2gd} \tag{2.15}$$

h_f = 1.23 m, Assume Losses in bends = 0.4 m; Total losses H_L = 1.23 + 0.4 = 1.63 m, Static lift, H = 115 – 100 = 15 m, Total lift = (H + $H_{L)}$ = 15 + 1.63 = 16.63.

$$\text{H. P of pump motor } \frac{QWH}{75} \tag{2.16}$$

Assume pump efficiency = 70%, Assume during unit efficiency = 80%, W = 1000 kglm^3; H. P. of pump motor = 6 Horse power.

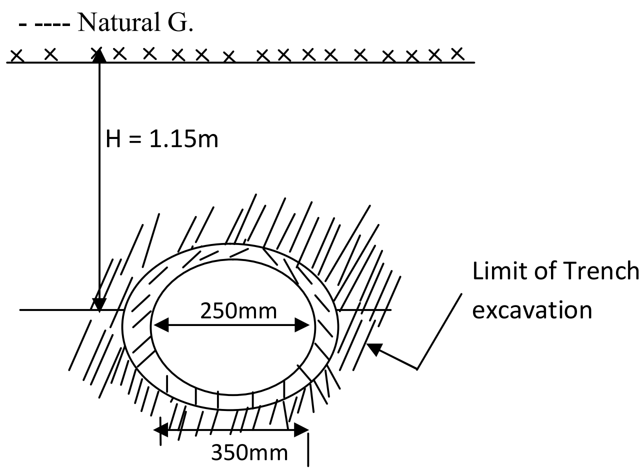

Fig. 7. Structural requirements for sewer

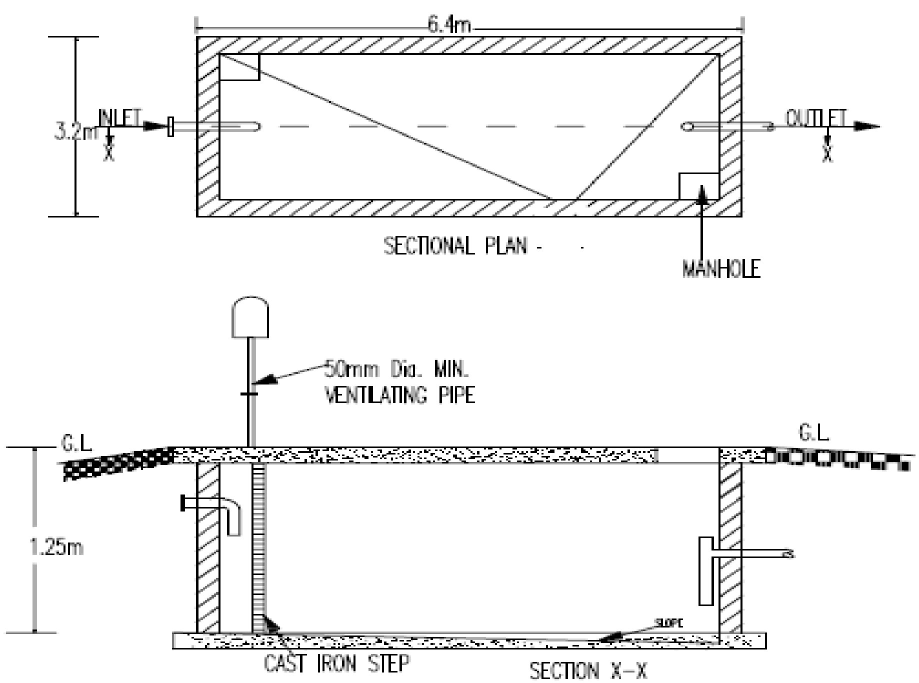

Fig. 8. Inlet/receiving chamber

2.6.7 Design of grit chamber

Grit chambers are provided to protect moving mechanical equipment from abrasion and accompanying abnormal wear. They reduce the formation of heavy deposits in pipelines, channels and conduits. They also reduce the frequency of digester cleaning that may be required as a result of excessive accumulations of grits in such units. High speed equipment such as centrifuges requires that practically, all grits be eliminated to prevent rapid wear and reduce maintenance. The removal of grits is also essential ahead of heat exchanger and high pressure diaphragm pumps. Grit channels, grit chambers or grit basins are intended to remove the grit present in the waste water. There are two general types of grit chambers, (i). Horizontal flow grit chambers, (ii). Aerated grit chambers.

To design Grit Chamber having rectangular cross – section and a proportional flow weir as the velocity control device, Max flow: 20 mLd, Diameter of the smallest grit particles to be removed: 0.2 mm, Average temperature: 25°C, Specific gravity of grit particle: 2.65.

For grit particles, the settling will be in the transition zone, for which settling velocity is given by Hazen's modified equation: V_s = 60.6 (S_S-I) $\frac{3t+70}{100}$ = 2.6 cm/sec, Critical Velocity is given by the modified shield's Equation:

$$4\sqrt{g\ (S_s - 1)}\text{d} \tag{2.17}$$

Velocity =22.8 cm/sec = 0.228 m/s, V_h = Vc = 0.228 m/sec, Q = 20 m/d = 0.231 m^3/s, Cross sectional area, A = $\frac{0.231}{0.228} = 1.0153$ m^2.

Providing a width of 1.25 m, liquid depth (H) required = 0.812 m. Provide a free board of 0.3m and a space of 0.25 m for sludge accumulation. Total depth = 0.812 + 0.3 + 0.25 = 1.362 m, depth = 1.4 m, ratio $\frac{H}{V} = \frac{Vs}{Vh} = \frac{2.6}{22.8} = \frac{1}{8.769}$ and L = 7.12 m.

This is the theoretical length. Allowing a 25% markup for inlet and outlet zone, hence total length = 9 m.

For the proportional flow weir as a control section to be used with the rectangular section of the above grit chamber, let a = 0.035 m, take C = 0.6

$$\text{Q} = 0.6\ \text{b}\sqrt{2ag}\left(ha - \frac{1}{3}a\right) \tag{2.18}$$

Therefore, b = 0.58 m

2.6.8 Comminutor

A comminuting device is a mechanically cleaned screen which incorporates a cutting mechanism that cuts the retained material (larger sewage solids) to about 6 mm in size, enabling it to pass along the sewage. Comminuting devices may be preceded by grit chamber to prolong the life of the equipment. Frequently, they are installed in the wet well of the pumping stations to protect the pump against clogging by rags and large objects. However, provision must be made to bypass comminutors incase flows exceed the capacity of the comminutor or incase there is a power or mechanical failure. The uses of comminutors tend to reduce odours, flies and unsightliness. A communitor consists of a vertical revolving drum-screen with 6mm to 10mm slots. The coarse material is cut by cutting teeth and shear bars on the revolving drum as solids are carried past a stationary comb as shown in the figure. The small sheared particles then pass through the slots of the drum and of a bottom opening through an inverted siphon. The head loss across comminutors depend s upon screen details and flow, the normal value being on the order of 50 to 100 mm. the grid intercepts the large solid particles whereas smaller solids pass through the space between the grid and cutting discs. The capacity of the comminutor for small town sewage treatment is rated between 1 – 2 hp (horsepower).

2.6.9 Primary settling tanks

These are usually large tanks in which solids settle out of water by gravity where the settleable solids are pumped away (as sludge), while oil float to the top and are skimmed off. It operates by means of the velocity of flow when reduced by 0.005 m so that the suspended material (organic settleable solids) will settle out. The usual detention time is 3 to 8 hours. Longer periods usually result in depletion of dissolved oxygen and subsequent anaerobic condition. Removal of suspended solid ranges from 50 to 65 percent and a 30 to 40 percent reduction of the five-day biochemical oxygen demand (BOD) can be expected.

For rectangular shape tank; Estate population = 2500, Water supply =300 litres per capital/day, Assuming 80% of water supplied to the estate is converted into sewage.

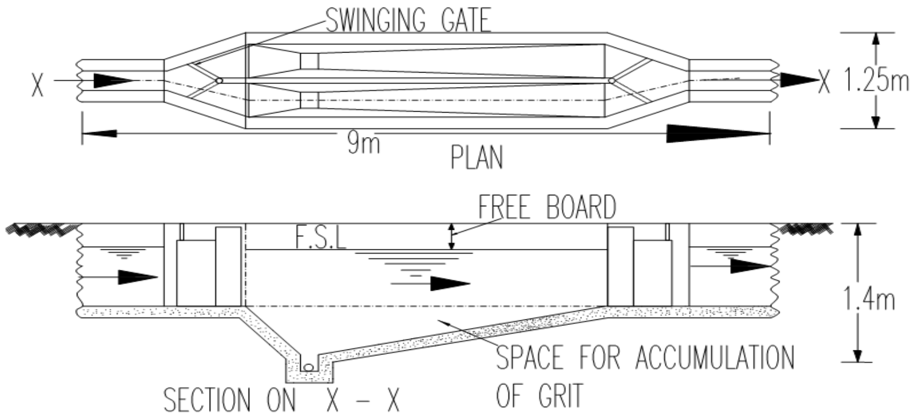

Fig. 9. Grit chamber

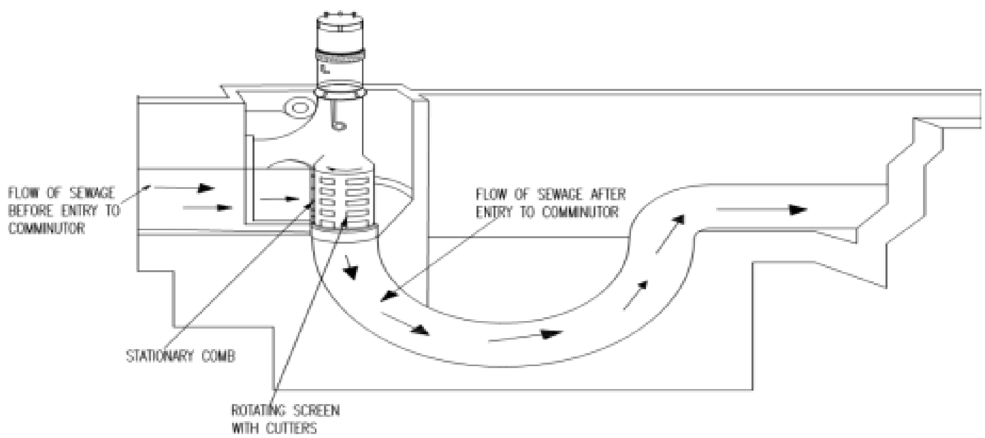

Fig. 10. Comminutor

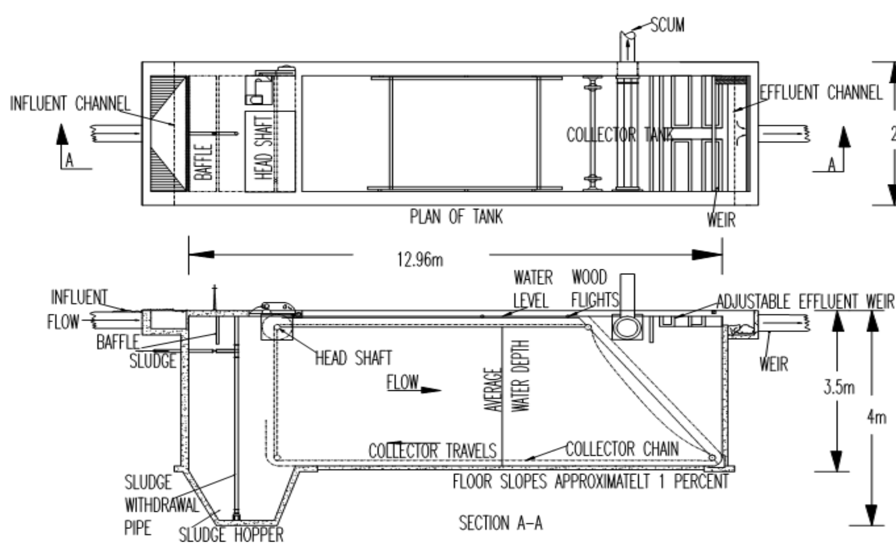

Fig. 11. Primary settling tank

Total sewage flow = 0.8x 2500 x 300 = 600000 litres/day, Assume a detention period of 3 hours; Capacity required = $\frac{600000\,x\,3}{24\,x\,1000}$ = 75 m^3.

Assume an overflow rate of 30 m^3/d/m^2 (from design parameters for settling tanks table), Surface area = $BxL = 20\,m^2$, L = 4B, B(4B) = 20, B = 2.24 m, L= 8.96 m.

Provide 4m for inlet and outlet arrangements; total length = 8.96 + 4 = 12.96 m, Effective depth of tank = $\frac{50}{20} = 2.5\,m$

Also provide 1 m extra depth of sludge accumulation and 0.5 m depth as free board; the tank dimensions will be 12.96 m x 2.24 m x 4 m.

2.6.10 Trickling filter

Trickling filter also known as percolating filter or sprinkling filters are similar to contact beds in construction, but their operation is continuous and they allow constant aeration. In this system, sewage is allowed to sprinkle or trickle over a bed of coarse, rough, hard filter media, and it is then collected through the under draining system. Spray nozzles or rotary distributors are used for this purpose. The biological purification is brought about mainly by aerobic bacteria which form a bacterial film known as bio film around the particle of the filtering media. The color of this film is blackish, greenish, and yellowish, and apart from bacteria, it may consist of fungi, algae, lichens, protozoa, etc. For the existence of this film, sufficient oxygen is supplied by providing suitable ventilation facilities in the body of the filter, and also to some extent, by the intermittent functioning of the filter. The straining due to mechanical action of the filter bed is much less. Organic removal occurs by biosorption from rapidly moving parts of the flow, and by progressive removal of soluble constituents from the more slowly moving portion.

Type of trickling fitter = Circular, Capacity = 1 million litres of sewage per day, Duration rate of Biochemical Oxygen Demand (BOD) = 5 day BOD of 120 mg/l.

Design the circular trickling fitter, the under drainage system as well as rotary system for the fitter. Suitable design data assumptions are made where necessary.

Design of fitter dimensions: Assuming it is hydraulic loading of 2 m^3/d/m^2 since the hydraulic loading of standard rate fitter varies between 1 to 4 m^3/d/m^2, Surface area required = 500 m^2, the organic loading of standard rate fitter varies from 80 to 320 g/d/m^3, assuming an organic loading of 150 g/d/m^3, total BOD present = (120 x10^{-3}) x (1000000) **=** 120000 g/day, Volume of fitter media required = $\frac{120000}{150} = 8000\,m^3$, Depth of fitter = $\frac{800}{500} = 1.6\,m$, diameter of fitter D = $\sqrt{\frac{500\,x\,4}{\pi}}$ = 25.23 m, Actual surface area = $\frac{\pi}{4}\,(25)^2$ = 490.9 m^2, Required Depth = $\frac{800}{490.9}$ 1.63 m, actual organic loading = $\frac{120000}{490.9\,x\,1.63}$ = 149.97/d/m^3, actual hydraulic loading = 2.04 m^2/d/m^2.

Design of rotary distributors: Design of central column; The pipe of rotary distributor is designed for a peak velocity of not greater than 2.0 mls and for average velocity not less than 1 mls. Hence let's assume a peak flow factor of 2.28 peak flow = 0.0264 m^3/sec, flow area of central column = $\frac{0.0264}{2}$ = 0.0132 m^2, Diameter of central column = $\sqrt{\frac{0.0132\,x\,4}{\pi}}$ = 0.13 m, Average flow = $\frac{0.0264}{2.28}$ = 0.0116 m^3/sec, Velocity of average flow = $\frac{0.0116}{0.0132}$ = 0.89 m/s.

This is less than permissible value of 1 mls. To bring it to the permissible value, the diameter of the central column must be reduced. However, reduction of diameter of central column will result in the increase in the velocity at the peak flow, which has to be restricted to a value of 2 mls. Hence provide 13 cm diameter. central column.

Design of arms; 4 arms for the rotary reaction spray type distributor will be provided, Peak discharge per arm = $\frac{0.0264}{4}$ = 0.0066 m^3/s, Length of arm = $\frac{25-0.13}{2}$ = 12.435 m.

Hence provide 12.44 m long arms with its size reducing from the centre to the end. For this purpose, 3 sections of arm will be provided, with first two sections of 4 m length and the third (end) section of 4.435 m length. The flow in these sections of each arm has to be adjusted in proportion to the filter area covered by these lengths of the arm. Let A_1, A_2 and A_3 be the circular filter area covered by each length of the arm. Hence, 0.33 m diameter in the centre for the central column was provided.

$A_1 = \pi[(4.11)^2 - (0.11)^2] = 53.03\,m^2$, $A_2 = \pi[(8.11)^2 - (4.11)^2] = 153.56\,m^2$, $A_3 = \pi[(12)^2 - (8.11)^2] = 245.76\,m^2$.

Hence proportionate areas served by each section of arm: $P_{a1} = \frac{A1}{A} x100 = 11.72\%$, $P_{a2} = \frac{A2}{A}$ x 100 $= 33.95\%$, $P_{a3} = \frac{A3}{A}$ x 100 = 54.33%

Discharge through each arm = 0.00066 m^3/s. The flow through velocity in the arm, at peak flow, should be less than 1.2 mls

Design of first section of the arm; Discharge = 0.0066 m^3/s, Design velocity = 1.2 mls, Area required = $\frac{0.0066}{1.2}$ = 5.5 x 10^{-3} m^2, Diameter required = $\frac{0.0055 \, x \, 4}{\pi}$ = 0.0837 m.

Area required = $\frac{0.0058}{1.2}$ = 0.0048 m^2, Diameter required = $\sqrt{\frac{0.0048 \, x \, 4}{\pi}}$ = 0.0782 m.

Design of third section of arm; Discharge = 0.0036m^3/s, Area required = $\frac{0.0036}{1.2}$ = 0.003 m^2, Diameter required = $\sqrt{\frac{0.003 \, x \, 4}{\pi}}$ = 0.0618 m.

Each arm is made up of three sections: The first section of 4 m length and 85 mm diameter, the second section of length 4 m and of 80 mm diameter, and the last section of length 4.435 m and of 60 mm diameter.

Design of orifices: Here 12 mm diameter orifices with a coefficient of discharge (C_d) equal to 0.6 and head causing flow equal to 1.5 m will be provided:

Discharge through each orifice =

$$Cd.a\sqrt{2gh} \tag{2.19}$$

Discharge = 3.6813 x $10^{-4} m^3/s$.

Number of orifices (n) in each section of the arm will be as under: First section, $n_1 = \frac{11.72}{100} \, x \, 18 = 2$, Second section, $n_2 = \frac{33.95}{100} \, x \, 18 = 6$, Third section, $n_3 = \frac{245.76}{100} \, x \, 18 = 10$.

The spacing (S) of orifices in each section will be as under: First section, $S_1 = \frac{4000mm}{2}$ = 2000 mm, Second section, $S_2 = \frac{4000mm}{6}$ = 6660.67 mm, Third section, $S_3 = \frac{4435mm}{10}$ = 443.5 mm.

Design of under drainage system: Peak flow = 0.0264 m^3/sec, let's provide central channel of rectangular section, fed by radial laterals of semi-circular section discharging into the central channel. The radial laterals, laid at a slope (S) of 1 in 40, will be in the form of under- drain block lengths containing semi-elliptical openings.

Design of rectangular efficient channel: The velocity of flow should not be less than 0.75 mls at peak instantaneous hydraulic loading or not less than 0.6 mls at average instantaneous hydraulic loading. Let's provide a flow velocity of 1mls at peak flow.

Peak flow = 0.0264 m^3/s, Area of channel = $\frac{0.0264}{1}$ = 0.0264 m^2, Assume a width of 0.15 m, Depth = $\frac{0.0264}{0.15}$ = 0.176 m.

Hence provide a width of 0.15 m and a depth of 0.18 m, Area, A = 0.15 x 0.18 = 0.027 m^2, Actual velocity = $\frac{0.0264}{0.027}$ = 0.98 mls, R = $\frac{A}{P} = \frac{0.027}{(0.15+2 \, x \, 0.18)}$ = 0.0529 m.

The bed slope of the channel is determined by manning's formula:

$$Q = \frac{I}{N} AR^{2/3}S^{1/2} \tag{2.20}$$

Assume N = 0.018, S = $\frac{1}{84459}$, say 1 in 84500.

Therefore providing the central efficient channel of width 15 cm and depth 18 cm below the level of lateral, and lay the channel at slope of 1 in 84500.

Design of radial laterals: Let S lay radial under-drain block length can be placed in rows, discharging into the effluent channel. In order to ensure proper ventilation, the laterals are designed to run approximately half full, d = $\frac{D}{2}$ or 0.5D. Where d = diameter of lateral when running half full, D = actual diameter of lateral when running full;

$$\text{Proportionate area} = \frac{a}{A} = \frac{\left(\frac{\pi}{4}\right)d^2}{\left(\frac{\pi}{4}\right)D^2} \tag{2.21}$$

$\frac{a}{A} = 0.25$, Corresponding to this, from the table of Hydraulic elements of circular sewers running partially full, by interpolation $\frac{\left(\frac{d}{D}\right)-0.2}{0.3-0.2} = \frac{0.25-0.43}{0.253-0.143}$, $\frac{d}{D}$ = 0.298, r/R = 0.482 + 0.202 (0.9817) = 0.680, $\frac{q}{Q}$ = 0.088 + 0.108 (0.9817) = 0.194.

Now permissible velocity at peak flow ≥ 0.75 mls; $\frac{q}{a}$ = 0.75 m/s, Q = $\frac{I}{N}$ $AR^{2/3}$ $S^{1/2}$ and q = $\frac{I}{N}$ $ar^{2/3}$ $S^{1/2}$;

$$\frac{q}{Q} = \frac{a}{A}\left(\frac{r}{R}\right)^{2/3} \qquad (2.22)$$

Therefore $\frac{r}{R}$ = 0.68, $\frac{Q}{A}$ = 0.9699.

Taking N = 0.015 and noting that R = D/4 and S = $\frac{I}{40}$, $D = 0.112\ m$

Discharge Q through a circular sewer of D = 0.12 m is 0.01151 m^3/s, q = 0.194Q = 0.00223 m^3/s, Number of laterals = $\frac{0.0264}{0.00223}$ = 12 laterals, V = $\frac{q}{a}$ = 0.789 mls (> 0.75 required), Average discharge = $\frac{0.0264}{2.28}$ = 0.0116 m^3/s, Average per lateral q = 0.000967 m^3/s, $\frac{q}{Q} = \frac{0.000967}{0.01151}$ = 0.084, $\frac{d}{D}$ = 0.10 + 0.1 (0.9403) = 0.194, $\frac{a}{A}$ = 0.1376, q = 1.556 x $10^{-3}$$m^2$, V_{av} = $\frac{q_{av}}{q_{av}}$ = 0.621 m/s (> 0.6 required).

Hence provide 12 radial laterals of semi –circular section, of 12 cm diameter laid at a slope of 1 in 40, each discharging into the rectangular efficient channel of width 15 cm and depth 18cm.

2.6.11 Secondary settling tank

Secondary settling tank assumes considerable importance in the activated sludge process as the effluent separation of the biological sludge is necessary, not only for ensuring final effluent quality but also for return of adequate sludge to maintain the MLSS level in the aeration tank. The secondary settling tank of the activated sludge process is particularly sensitive to fluctuations in the flow rate and on this account; it is recommended that the units be designed not only for average overflow rate but also for peak overflow rates. The high concentration of the suspended solid in the effluent requires that the solid loading rates should also be considered.

MLSS = 3000 mg/L, Peak flow = 45 mld = 45000 m^3/d, Peak factor = 3.8

Average flow = $\frac{45}{3.8}$ = 11.84 mLd = 11840 m^3/d.

Adopting a surface loading rate of 20 $m^3/d/m^2$ at average flow; Surface loading at peak flow = $\frac{45000}{20}$ = 592 m^2, surface loading at peak flow = $\frac{45000}{592}$ = 76 m^3/dm^2

This is within the prescribed range of 40 to 50.

On the basis of solids loading of 125 k/day/m^2 at average flow; Area required = $\frac{1184000\ 0\ x\ 3000\ x\ 10^{-6}}{125}$ = 284.16 m^2.

On the basis of solids loading of 250 kg/day/m^2 at peak flow; Area required = $\frac{(45\ x\ 10^{-6})x(3000\ x\ 10^{-6})}{250}$ =540 m^2.

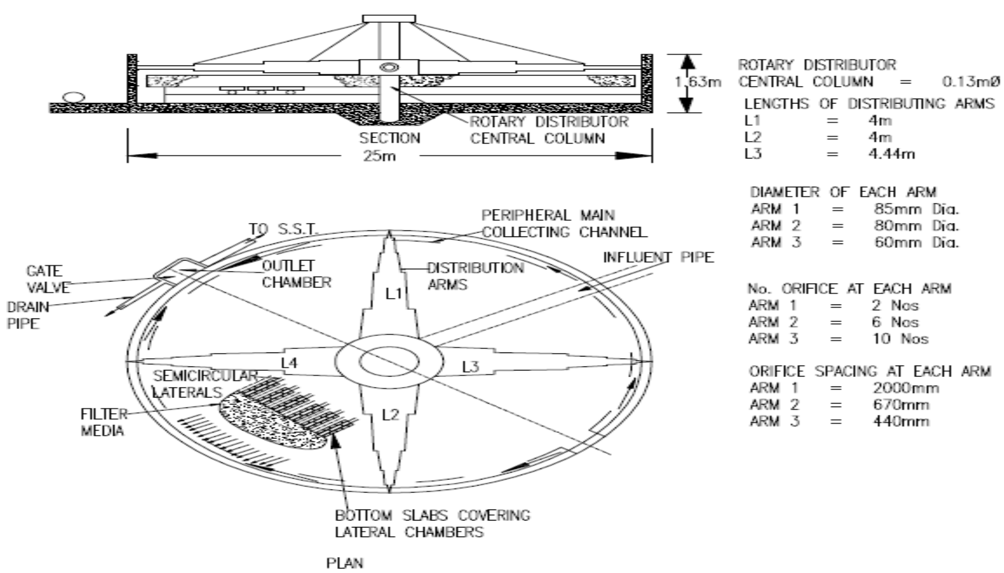

Fig. 12. Standard rate trickling filter

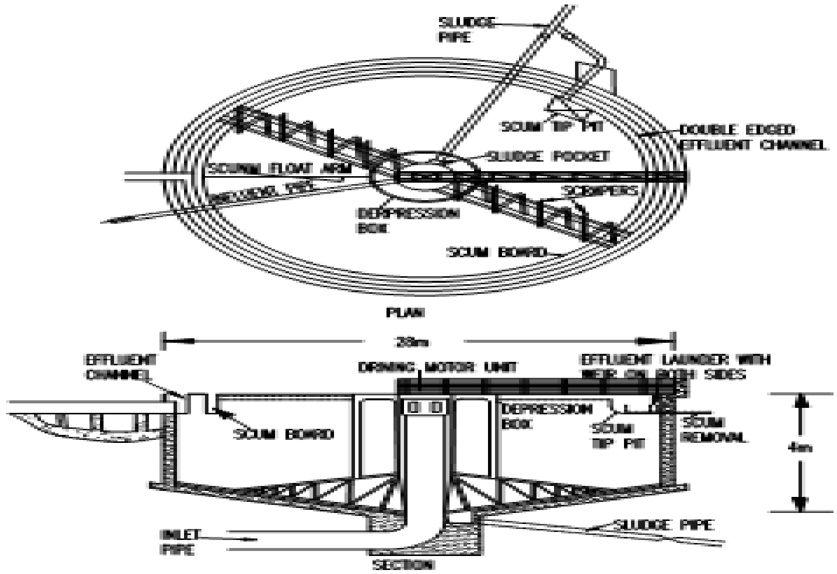

Fig. 13. Secondary sedimentation tank for activated sludge process

Hence, adopting surface area of 1000 m^2 which is highest of the three values; Adopting a circular tank Diameter; d = $\sqrt{\frac{592 \, x \, 4}{\pi}}$ = 27.5 m, Adopt a diameter of 28m; Actual area = $\frac{\pi}{4}(28)^2$ = 615.8 m^2, Actual solid loading at average = $\frac{(11840000) \, x \, (3000 \, x \, 10^{-6})}{615.8}$ = 58 kg/day/m^2, Length of weir = π x 28 = 88 m, Weir loading at average flow $\frac{11840}{88}$ = 135 m^3/d/m.

Note: This is more than recommended value 125 m^3/m/d for small tank. Hence provide a trough instead of a single weir, at the outer periphery thus getting double edge effluent channel, for which available over flow length will be 2 x 88 and weir loading will reduce to $\frac{135}{2}$ = 67.5 m^3/d/m length. Keep the depth of tank equal to 4 m.

2.6.12 Sludge digester

Here the digester is the Mesophilic type. In the moderate temperature, digestion is brought about by common mesophilic organism. The temperature in this zone ranges between 20 to 45°C. The optimum mesophilic temperature is about 37°C; and at this temperature, the digestion period can be brought down to about 30 days.

Hence, it can be concluded that the sludge can be quickly digested, if the temperature in the digestion tank is kept high but best results are obtained at about 37°C. However it may be difficult to control temperature in practice, as it mainly depends upon the prevailing local climatic conditions. In this regard, external heating devices may sometimes be employed to control temperature in the digestion tanks, especially in cold countries.

Construction Details: A typical sludge digestion tank consists of a circular tank with hoppered bottom and having a fixed or a floating type of roof over its top. The raw sludge is pumped into the tank, and when the tank is first put into operation, it is seeded with the digested sludge from another tank, as pointed out earlier. A screw pump with an arrangement for circulating the sludge from bottom to top of the tank or vice versa (by reversing the direction of rotation of the screw) is commonly used, for stirring the sludge. Sometimes, power driven mechanical devices may be used for stirring the sludge, although these are not very popular at present.

The tank is provided with heating coils through which hot water is circulated in order that the temperature inside the tank is maintained at optimum digestion temperature level.

The gases of decomposition (chiefly methane and carbon dioxide) are collected in a gas dome (in smaller tanks) or collected separately in gas holders (in larger tanks) for subsequent use. The digested sludge which settles down to the hoppered bottom of the tank is removed under hydrostatic pressure, periodically, once a week or so. The supernatant liquor lying between the sludge and the scum is removed at suitable elevations, through a number of withdrawal pipes. The supernatant liquor, being higher in BOD and suspended solids contents, is sent back for treatment along with the raw sewage in the treatment plant. The scum formed at the top surface of the supernatant liquor is broken by the recirculating flow or through the mechanical rakers called scum breakers.

Population of CBN Estate = 2500 persons

Designing digester for digesting mixed raw and activated sludge, referring to table of solids in sludge's (per capita per day) for volumes of mixed raw primary, activated and digested sludge.

Volume of mixed raw (undigested) primary plus activated sludge % of the volatile matter in raw max sludge = $\frac{58}{85}$ x 100 = 68.24%, % of non – volatile (or fixed) matter in sludge = $\frac{27}{85}$ x 100 = 31.76%.

Hence specific gravity of dry solids in mixed raw sludge is given by the equation

$$\frac{100}{S_s} = \frac{\text{\% of mineral/matter}}{\text{(SP gravity of mineral matter)}} + \frac{\text{\% of volatile/matter}}{\text{(SP gravity of organic matter)}} = \frac{68.24}{1} + \frac{31.76}{2.5} = S_d = 1.235$$

Taking percentage solids as 4%, specific gravity of wet – sludge (mixed primary plus activated sludge) is given the equation $\frac{100}{S_{sl}} = \frac{\text{\% moisture}}{\text{SP.gravity of water}} + \frac{\text{\% of solids}}{S_s} = \frac{96}{1} + \frac{4}{1.235} = S_{Sl} =$ 1.0077.

Hence, volume of mixed sludge is given by the formula,

$$V_{Sl} = \frac{Ws}{\rho w\, Ssl\, Ps} \tag{2.23}$$

Where; Vsl = Volume of sludge, Ws = Weight of dry soils (kg), Ssl = specific gravity of sludge, Ps = Percent solids expressed as a decimal, ρ_w = Density of water (10^3 kglm^3 at 5°C).

$$Vsl = \frac{\text{Ws}}{\text{Sw Ssl Ps}} = \frac{85 \times 2500 \times 10^{-3}}{1000 \times 1.0077 \times 0.04} = 5.27 \text{ m}^3\text{/day}$$

Volume of mixed digested primary plus activated sludge. % of volatile matter in digest sludge from the Table of solid in sludge (per capita per day) = $\frac{20}{57}$ x 100 = 3509%, % of non – volatile (or fixed) matter in digested sludge = $\frac{37}{57}$ x 100 = 64.91%

Hence, Sd^1 = 1.638

Taking percentage solids digested sludge as 7%, specific gravity of wet digested sludge is given by $\frac{100}{S_{sl}^1} = \frac{93}{1} = \frac{7}{1.638} = Ssl^1 = 1.028$, Hence volume of digested mixed sludge is $V_{Sl}{}^1 = \frac{57 \, x \, 2500 \, x \, 10^{-3}}{1000 \, x \, 1.028 \, x \, 0.07} =$ 2 m^3/day

Volume of digester, assuming average working temperature = 27.8°C.

From the table of variation of digestion with temperature, the digestion period = 30 days. Also assume 60 days storage in monsoon. Assuming a parabolic reduction of volume, the capacity (or volume) of the digester is given by the equation.

Where; V = Volume of digester, V_f = Volume of fresh sludge added per day, V_d = Volume of digested sludge with drawn per day, T_1 = Digestion time in days, and T_2 = Monsoon storage in days

$$V = \left[5.27 - \frac{2}{3}(5.27 - 2)\right] x\, 30 + 2\, x\, 60 = 92.7 + 120 = 212.7 \text{ m}^3$$

(Note: This is within 0.08 to 0.15m^3/capita of combined sludge)

Loading factor: Total loading of volatile solid = 58 x 2500 x 10^{-3} = 145 kg/day

Volatile solid loading factor = $\frac{145}{212.7}$ = 0.682 kg/day/m^3

(Note: This is within the prescribed range of 0.3 to 0.75 kg/day/m^3)

Dimensions of digester: Let's assume that for a cylindrical digester, the average gas production is at 0.9 m^3/kg of volatile matter destroyed. From table of solids in sludge (per capita per day) volatile matter destroyed during digestion of combined sludge = 38 gm/capita

Total volatile matter destroy = 38 x 10^{-3} x 2500 = 95 kg, Gas produced = 0.9 x 95 = 85.5 cm^3

Note: It is recommended that in order to avoid foaming, the optimum diameter or depth is calculated such that twice the average rate of gas production, the value of 9 m^3/m^2 of tank area is not exceeded.

Hence minimum area of digester required (to avoid foaming) = $\frac{2 x 85.5}{9}$ = 19 m^2, depth of digester = $\frac{212.7}{19}$ = 11.19 m, the depth should not exceed 9 m.

Hence 2 (two) digesters are proposed; Volume of each tank = ½ V = ½ x 212.7 = 106.35 m^3, Adopting depth of 8 m in each tank; Diameter of each tank = $\sqrt{\frac{106.35 x 4}{\pi x 8}}$ = 4.11 m.

Provide a free board of 0.6 (for floating cover). Hence adopt 2Nos of digestion tank, each of 5m diameter and 8.6 m height.

2.6.13 Sludge drying beds

This method of dewatering or drying the sludge is especially for those locations where temperature is higher. The method consists of applying the sludge on specially prepared open beds of land. A sludge drying bed usually consists of a bottom layer of underground of uniform size over which is laid a bed of clean sand. Open jointed tile under drains are laid in the ground layer to provide positive drainage as the liquid passes through the sand and gravel. See the design calculations below.

Designing a sludge drying bed for digested sludge from sludge digester plant for 2500 persons in CBN Housing Estate, from table of solids in sludge (per capita per day), total solids remaining in digested sludge (combined primary and activated) = 57 gm/capita/day, Daily solids = 2500 x 57 x10^{-3} = 142.5 kg/day.

Adopting a dry solid loading of 100 kg/m^2/year; Area of bed needed = $\frac{142.5 x 365}{100}$ = 520.125 m^2.

Check for per capita area = $\frac{520.125}{2500}$ = 0.2081 m^2 (Note: This is within the recommended range of 0.175 to 0.25).

Adopt 8 m wide x 30 m long Beds with single point discharge and a bed slope of 0.5%; Number of Beds = $\frac{520.125}{8 x 30}$ = 2Nos

Assuming 2 months of rainy season in a year and 3 weeks of drying and one week for preparation and repair of bed, number of cycle per year = $\left(\frac{12-2}{4}\right) x\ 4$ = 10.

Let's assume 7% solid and a specific gravity of 1.025, the volume of digested sludge is given by the equation

$$V_{sl} = \frac{\mathrm{Ws}}{\rho\mathrm{w.Ssl.Ps}} \qquad (2.24)$$

V_{sl} = $\frac{142.5}{1000 x 1.025 x 0.07}$ = 2 m^3/day, Depth of application of sludge = $\frac{2 \text{ x } 365}{2 \text{ x } 8\ 30 \text{ x} 10}$ = 0.152 m = 15 cm.

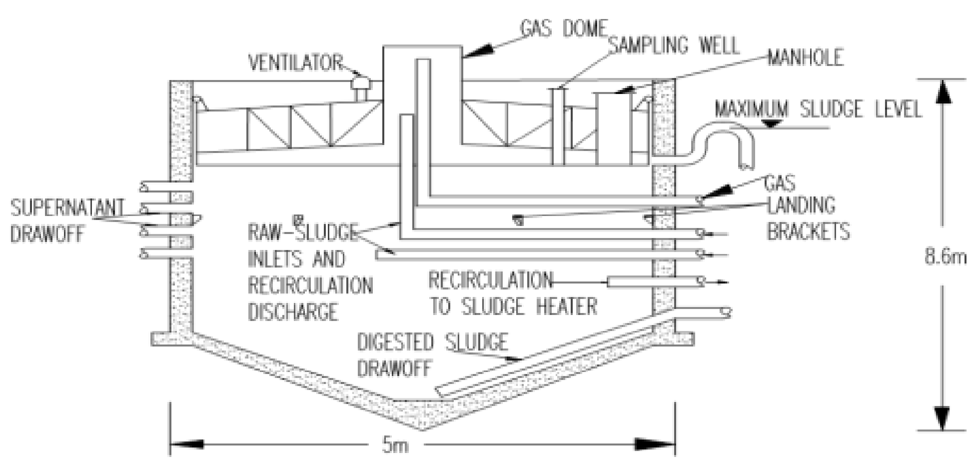

Fig. 14. Cross section of typical anaerobic sludge digester

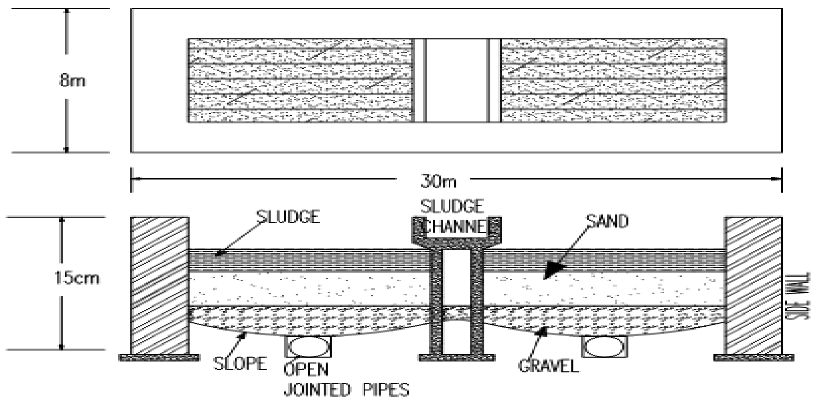

Fig. 15. Sludge drying bed

3. CONCLUSION

A successful technical project involves the integration of various fields. This is an attempt to combine several aspects of environmental, biological, chemical, civil and mechanical engineering. Since in CBN Housing Estate there is no proper treatment plant for sewage, it is necessary to construct a sewage treatment plant for a housing estate of such magnitude. The plant is designed perfectly to meet the future expansion for the next 30 years in accordance with Federal Government of Nigeria Codal provisions. This project consists the design of the complete components of sewage Treatment Plant from Receiving Chamber, Grit Chamber, Comminutor, Primary Settling Tank, Trickling Filter, Secondary Settling Tank, Sludge Digester and sludge Drying Beds for sewage.

The basic data were first of all worked out and stipulated for the proposed sewage treatment plant on the basis of per capita sewage produced, quality of sewage produced and the standards of effluent specified. The STP was designed using trickling filter instead of activated sludge process due to the population of the occupants and the availability of land area for the construction of the plant.

Proper use and maintenance of the sewage treatment system will ensure effective sewage management in the estate.

COMPETING INTERESTS

Authors have declared that no competing interests exist.

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