UNIVERSITY OF NAIROBI

Design of Sewerage and Wastewater Treatment Plant for Thika Greens Phase II Housing Development

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Abstract

This project is a design of a wastewater treatment plant (WWTP) for a housing development. The housing development is Thika Greens Phase II which is located 40km from Nairobi. The treatment plant will treat domestic and commercial wastewater from the housing development. The WWTP will cater for a 3 % projected population growth rate over a design period of 20 years from 2014 to 2034. The sewerage system consisted of 4.3 km of concrete pipes ranging from 150 – 1000mm. The quantity of flow ranged from 0.01 – 7 m^3/min. The velocities ranged from 0.1 – 1 m/s. The components of the WWTP were sized in order to cater for the quantity of wastewater entering the different treatment processes. The treatment plant consisted of one bar screen chamber, two aerated grit chambers, four sedimentation tanks, three secondary clarifiers and two sludge drying beds.

Dedication

This project is dedicated to God without whom I am truly nothing.

Psalms 139:5 “You hem me in - behind and before; you have laid your hand upon me”

Acknowledgements

I would wish to acknowledge my family who have given me invaluable support, my supervisor Dr. P.K. Ndiba for his advice, understanding and guidance, Eng. Moses. O. Otieno of Finix Consultants who was very generous with the data I needed and my friends who have given me advice, encouraged and prayed for me.

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CHAPTER ONE

1 INTRODUCTION

1.1 BACKGROUND

Any new and existing housing development requires supporting infrastructure such as roads, electrical and water supply system, and a means of collecting and treating wastewater. The necessary utilities factor in population growth over a certain design period.

This project focuses on providing a sewerage system and a wastewater treatment plant for a new housing development -Thika Greens Phase II. Thika Greens lies along Thika super highway. It is the pioneer golf estate concept in Kenya. Phase II includes sewer lines, water and roads infrastructure. The housing development covers an area of 113 ha and has 295 housing units each designed to hold 6 people and each plot covering an approximate area of 0.1 ha.

A sewerage system transports wastewater to a treatment facility. It should convey the wastewater in such a manner to avoid clogging and silting of sewer pipes while considering contributing factors such as the slope of the land, the maximum flow and velocity.

A Wastewater Treatment Plant receives the wastewater from domestic, commercial and industrial sources and removes materials that damage water quality and compromise public health and safety when discharged into water receiving systems. It includes physical, chemical, and biological processes to remove various contaminants depending on its constituents.

1.2 OBJECTIVES

The overall objective of the project is to design a wastewater treatment facility that will produce an environmentally safe wastewater stream and a solid waste suitable for disposal or reuse.

The specific objectives were to:

1. To project present and future effluent emissions from the housing development for a period of 20 years.
2. Collect wastewater from the houses with sewer pipes and to transport it to the wastewater treatment plant.
3. Size the various components of the treatment works.

CHAPTER TWO

2 DESIGN CRITERIA

2.1 INTRODUCTION

In the design of the sewerage system and the treatment plant, various constraints are put in place to aid in the design. These constraints are standards that have been tried and tested in the design of many other facilities and work. The constraints also determine the final recommendations for action and the project's potential for success.

The design is also based on the following approaches:

1. Where does the wastewater come from?
2. How much wastewater flow is there going to be?
3. How is the wastewater going to be removed and treated?

Municipal waste water is the term applied to the liquid wastes collected from residential, commercial, and industrial areas and conveyed by means of a sewerage system to a central location for treatment.

Treatment of sewage is essential to ensure that the receiving water into which the effluent is ultimately discharged is not significantly polluted. The degree of treatment required however depends upon the type of receiving water. This is referred to as the receiving water standards.

The current treatment methods or technologies for sewage/domestic wastewater treatment are either oxidation ditches, aerated lagoons, trickling filters, stabilization ponds, wetlands and trickling filters combined with stabilization ponds.

2.2 SEWERAGE SYSTEM

Designing a sanitary sewer involves estimation of waste flow rates for the design data and evaluation of any local conditions, which may affect the hydraulic operation of the system; the selection of the hydraulic-design equation, alternative sewer pipe materials and minimum and maximum sizes, minimum and maximum velocities and slopes; the evaluation of alternative alignments or designs.

Public sanitary sewers perform two primary functions:

1. Safely carry the design peak discharge,
2. Transport suspended materials to prevent deposition in the sewer.

In designing a sewer system, the designer must

i. conduct preliminary investigations,
ii. review design considerations and select basic design data and criteria,
iii. design the sewers which include preparation of a preliminary sewer system and design of individual sewers

The average daily domestic wastewater generation was estimated on a water consumption basis. The design criteria is:

1. Domestic wastewater is approximately 80% of water consumption = 300 l/capita/day
2. Water consumption is based on the land carrying capacity(per ha)

   LCC= $\frac{\text{population}}{\text{Land area}}$
3. Peak factor = 2.5
4. The sewer is designed to run half full. Generally, sewers less than 0.4m in diameter are designed as running half full at maximum discharge and those larger than 0.4m are designed as running $^2/_3$ or $^3/_4$ full. [5]
5. Velocity: 0.75 – 3 m/s
6. Minimum size of sewer: 100 - 150mm
7. Minimum slope: for full pipe velocity 0.6 m/s
8. Minimum cover = 2m

2.3 TREATMENT WORKS

The design criteria includes that for the treatment facilities such as the collection basin, screens, aerated grit chamber, primary sedimentation tank, secondary treatment tank and sludge drying beds.

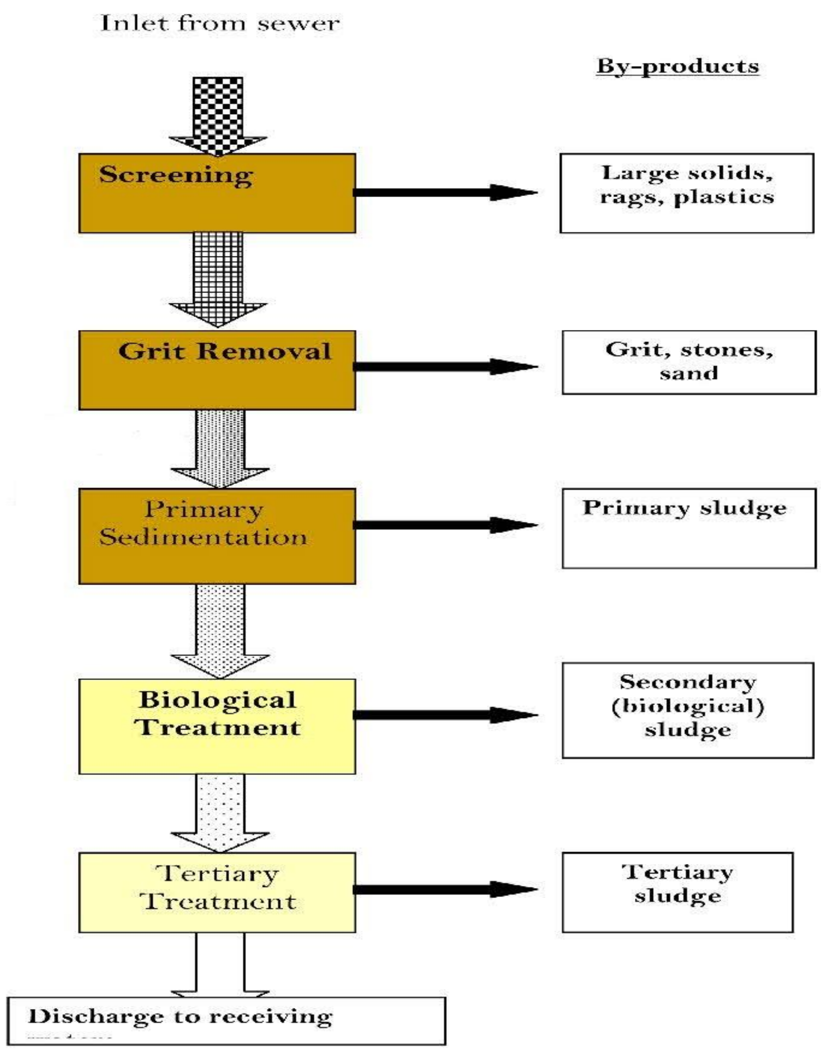

Fig. 1: Flow chart for wastewater treatment process

2.3.1 Preliminary Treatment

2.3.1.1 Screens

The general purpose of screens is to remove large objects such as rags, paper, plastics, metals, and the like. These objects, if not removed may damage the pumping and sludge removal equipment, hangover wires, and block valves, thus creating serious plant operation and maintenance problems. Screening is normally the first unit operation used at wastewater treatment plant, used remove large objects from wastewater.

Removal of screening:

Manually cleaned bar rakes have sloping bars that facilitate hand raking. The screening is placed on a perforated plate for drainage and storage.

The mechanically cleaned bar rakes are front-cleaned or back –cleaned, in both cases the traveling rake moves the screenings upward and drops them into a collection bin or conveyor.

Bar rack screens will be used for the screening process. The design criteria is:

1. Slope from horizontal = 45° - 70°
2. Clear spacing between bars = 10 - 40 mm
3. Velocity through rack = 0.3 - 1 m/s

2.3.2.1Aerated Grit Chamber

The Grit chamber follows the screening process. Flow from screen channels shall be taken into grit chambers. It is necessary to remove the grits and other materials that are heavier than organic matter, in order to:

1. Protect moving mechanical equipment and pumps from unnecessary wear and abrasion.
2. Prevent clogging in pipes heavy deposits in channels.
3. Prevent cementing effects on the bottom of sludge digesters and primary sedimentation tanks.
4. Reduce accumulation of inert material in aeration basins and sludge digesters which would result in loss of usable volume.

Type of grit removal :

1. Velocity – controlled grit channel : It is a long narrow sedimentation basin with better control of flow through velocity.
2. An aerated grit chamber : A Spiral current within the basin is created by the use of diffused compressed air- the air rate is adjusted to create a velocity near the bottom , low enough to allow the girt to settle, whereas the lighter organic particles are carried with the roll and eventually out basin.

Advantages of aerated grit chamber

i. An aerated grit chamber can also be used for chemical addition, mixing, and flocculation a head of primary treatment.
ii. Wastewater is freshened by the air, thus reduction in odours and additional BOD removal may be achieved.
iii. Grease removal may be achieved if skimming is provided.

Provide two identical grit chambers for independent operation and to allow ease of maintenance. The design criteria is:

1. Detention time = 4 min
2. Freeboard = 0.8 m
3. Air supply = 7.8 l/s per metre length of the chamber

2.3.2 Primary Treatment

2.3.2.1 Primary Sedimentation Tank

The purpose of sedimentation of sewage is to separate the settle-able solids to that the settled sewage if discharged into water does not affirm sludge banks. Sedimentation of solids also reduces the organic load on the secondary treatment units. Normally, a primary sedimentation will remove 50 - 70 % of the total suspended solids and 30 - 40 % BOD5.

Primary sedimentation (or clarification) is achieved in large basins under relatively calm conditions. The settled solids are collected by mechanical scrapers into hopper, from which they are pumped to sludge – processing area oil, grease , and other floating materials are skimmed from the surface. The effluent is discharged over weirs into a collection trough.

Types of clarifiers:

In general, the design of most of the clarifiers falls into three categories:

1. horizontal flow,
2. solids contact, and

3. inclined surface.

The common types of horizontal flow clarifiers are rectangular,square, or circular . On the other hand the types of include surface are tube settler and parallel plate settler. In this project a rectangular clarifier will be used.

Sludge collection :

Collection is from the bottom slope. The floor of the rectangular and circular tanks are sloped toward the hopper. The slope made to facilitate draining of the tank and to move the sludge the hopper. Rectangular tanks have a slope of 1-2 percent. In circular tanks, the slope is approximately 40-100 mm in diameter.

The type of equipment in mechanized sedimentation tanks, for sludge collection varies with size and shape of the tank.

In rectangular tanks the sludge collection equipment may consist of

1. a pair of endless conveyor chains running over sprockets attached to the shafts or
2. moving bridge sludge collectors having a scraper to push the sludge into the hopper.

The design criteria is:

1. Overflow rate and detention time shall be based on the average design flow.
2. Detention time is between 2 - 2.5 hours.
3. Overflow rates = max. of 15 m^3/m^2.day for average flow

   = max. of 40 m^3/m^2.day for peak flow
4. Depth = 3 - 4.5 m

2.3.3 Secondary Treatment

Secondary waste treatment involves bringing the active microbial growth in contact with wastewater so that they can consume the impurities as food. The purpose of secondary treatment is to remove the soluble organisms that escape the primary treatment and to provide further removal of suspended solids.

A great variety of microorganisms come into play that include bacteria, protozoa, rotifers, fungi, algae, and so forth . these organisms in the presence of oxygen convert the biodegradable organics into carbon dioxide, water, more cell material, and other inert products.

Biological Treatment Process:

Biological treatment process can be achieved by two types of growth:

1. **Suspended Growth Biological Treatment :**

Suspended growth treatment systems are those in which the microorganisms remain in suspension, also known as mixed liquor volatile suspended solids (MLVSS). Common suspended growth processes used for secondary treatment include:

i. Activated sludge and other modifications.

ii. Aerated lagoons.

iii. High –rate stabilization ponds.

2. **Attached Growth Biological Treatment**

In attached growth biological treatment process the population of active microorganisms is developed over a solid media (rock or plastic).the attached growth of microorganisms stabilize the organic matter as the wastewater passes over them. There are two major types of attached growth process:

i. Trickling filter.

ii. Rotating Biological contractor.

Although secondary treatment may remove than 85 percent of the BOD and suspended solids, it does not remove significant amount of nitrogen, phosphor heavy metals, non-degradable organics, bacteria and viruses. These pollutants may require a further removal through an advanced treatment.

The design criteria is:

1. Organic loading = 0.1 kg/m^3/day
2. Hydraulic loading = 0.5 m^3/m^2/day
3. MLVSS = 3000 mg/l

2.3.4 Sludge dewatering

The principal sources of sludge at municipal wastewater treatment plants are the primary sedimentation basin and the secondary clarifiers.

Additional sludge may also come from chemical precipitation, nitrification de-nitrification facilities, screening and grinder, and filtration devices if the plant has these processes. Many times the sludge is produced in these processes treatment systems so that the sludge is removal as either primary or secondary sludge. In some cases, secondary sludge is returned to the primary setting tank, ultimately giving a single stream consisting of combined sludge.

Sludge contains large volumes of water. The small fraction solids in the sludge is highly offensive. Thus, the problem involved with handling and disposal of sludge are complex. Common sludge management processes include thickening, stabilization, dewatering, and disposal.

Sludge dewatering is necessary to remove moisture so that the sludge cake can be transported by truck and can be composted or disposed of by land filling or incineration. The solid particles in municipal sludge are extremely fine, are hydrated, and carry electrostatic charges.

These properties of sludge solids make dewatering quite difficult. Sludge conditioning is necessary to destabilize the suspension so that proper sludge- dewatering devices can be effectively used.

Sludge dewatering systems range from very simple devices to extremely complex mechanical processes. Simple process involves natural evaporation, and percolation from sludge lagoons or drying beds.

2.3.4.1 Sludge drying beds

Sludge drying beds are the oldest method of sludge dewatering. These are still used extensively in small- to medium – size plants to dewater digested sludge. Typical sand beds consist of a layer of coarse sand 15 - 25 cm in depth and supported on a graded gravel bed that incorporates selected files or perforated pipe under rains. Paved drying beds are also used. Each section of the bed contains water-tight walls, under drain system, and vehicle tracks for removal of sludge cake.

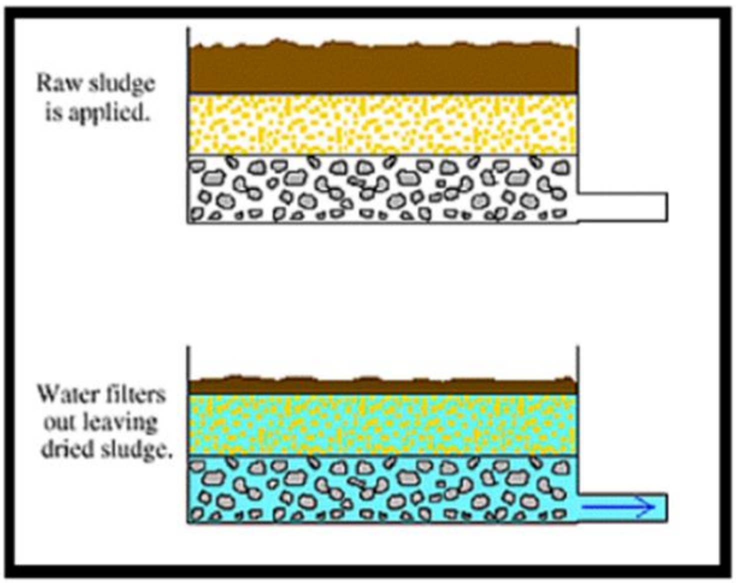

Fig 2: Sludge drying bed

The design criteria is:

1. Specific gravity = 1.015 g/cm^3
2. Solids content = 1.5 %
3. MLVSS = 3000 mg/l

CHAPTER THREE

3 CALCULATIONS AND RESULTS

3.1 FLOW PROJECTIONS

i. Land area = 113 ha

ii. Population density = 60 people per ha

iii. Population $P_o = 113 \times 60 = 6780\ people$

iv. Design period t= 20 years

v. Growth projection = 3 %

k= 0.03

vi. $P_t = P_o + ktP_o$

= 6780 + 0.03 *20*6780 = 10,848 people

3.2 SEWERAGE DESIGN

i. Flow rate is estimated = 300 l /person/ day

ii. Land carrying capacity = 60 persons/ ha

iii. Manning's coefficient n = 0.015

iv. Minimum slope = 0.01 %

v. Minimum pipe size = 150 mm

vi. Manning's equation for determination of diameter

$$D = 1.548({}^{n\,Q}/_{S^{0.5}})^{0.375}$$

vii. Velocity of flow $V = {}^{1}/_{n} \,.\, ({}^{D}/_{4})^{2/3} \,.\, S^{0.5}$

3.3 PLANT CAPACITY

i. Population = 10848

ii. Waste water generation = 300 l /capita/ day

iii. Annual daily flow = 300*10848 = 3,254,400l/day

= 388.5 m^3/day

=0.904 m^3/s

iv. Peak factor = 2

v. Design flow capacity (maximum) = 1.808 m^3/s

3.4 BAR SCREEN

i. Assume depth of flow in the rack chamber = 1.2m

ii. Clear area through the rack = $Q_{average} / Velocity\ through\ rack$

$= 0.904/0.3 = 3.01\ m^2$

iii. Clear width of the opening = $3.01/1.2 = 2.5\ m$

iv. Assume the width of each bar = 1cm

v. Assume clear spacing = 2.5 cm

vi. No. of spacing = $2500/25 = 100\ spaces$

vii. Total no. of bars = 100-1 = 99 bars

viii. Width of chamber = 2.5 + $^{(10 \times 99)}/_{1000} = 3.49m$

ix. Efficiency = $^{25 \times 99}/_{3490} = 0.71$

3.5 AERATED GRIT CHAMBER

Provide **two** identical grit chambers for independent operation.

i. Maximum design flow through each chamber

$= {}^{1.808}/_{2}$

$= 0.904\ m^3/s$

ii. Volume of each chamber for 4-min detention period

$= 0.904 m^3/s \times 4$ min $\times$ 60 sec/min

$= 216.96\ m^3$

iii. Provide average water depth at mid width

= 3.5 m

iv. provide freeboard

= 0.8 m

v. Total depth of grit chamber

= 4.3 m

vi. Surface area of chamber

$= {}^{217\ m^3}/_{3.5\ m}$

$= 61.99\ m^2$

vii. provide length to width ratio

= 4:1

⇒area = $4w^2$

viii. Width of the chamber

= 3.94 m

= 4.0 m

ix. Length of the chamber

= 15.6 m

= 16 m

Air Supply System

i. Provide air supply at a rate of 7.8 l /s per meter length of the chamber.

ii. Theoretical air required per chamber.

= 7.8 l /s.m × 16 m

= 124.96 l /s

iii. Provide 150 percent capacity for peaking purpose.

iv. Total capacity of the diffusers

= 1.5× 124.96

= 187.44 l /s per chamber

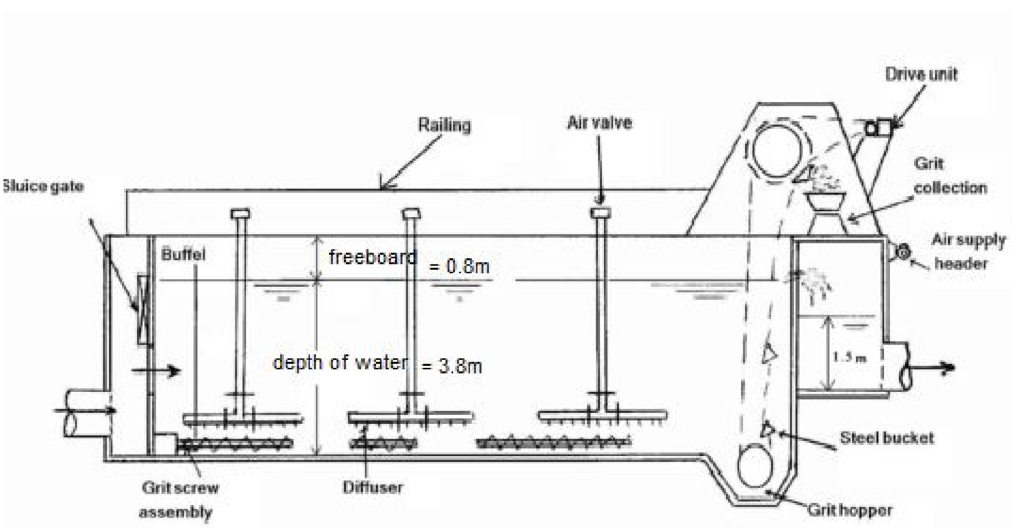

Fig. 3: Aerated Grit Chamber

3.6 PRIMARY SEDIMENTATION TANK

Four rectangular units were designed for independent operation. A bypass to the aeration basin shall be provided for emergency conditions when one unit is out of service.

i. Average design flow through each basin

$= {}^{0.904}/_{4}$

$= 0.226\ m^3/s$

ii. Overflow rate at average flow

$= 24\ m^3/m^2 day$

iii. Surface area $= {}^{0.226\ m^3 \times 86400\ s/day}\Big/_{24 m^3/m^2 day}$

$= 813\ m^2$

iv. Use length to width ratio (4:1)

$\rightarrow$ A = $4w^2$

v. Wide of each basin

= 14.3 m

vi. Length of each basin

= 4× 14.3

= 57.2 m

vii. Provide average water depth at mid. Length of the tank.

= 3.1 m

viii. Provide Freeboard

= 0.6 m

ix. Average depth of the basin

= 3.1+0.6

= 3.7 m

Check Overflow rate

i. Overflow rate at average design flow

$$= \frac{0.226\, m^3 \times 86400\, s/day}{14.3 \times 57.2}$$

$$= 23.87\, m^3/m^2 day$$

ii. Average rate at max. design flow

$$= \frac{0.452\, m^3 \times 86400\, s/day}{14.3 \times 57.2}$$

$$= 47.74\, m^3/m^2 day$$

Detention Time

i. Average volume of the basin

$$= 3.1 \times 14.3 \times 57.2$$

$$= 2535.67\, m^3$$

ii. Detention time of average design flow

$$= \frac{2535}{0.226 \times 3600}$$

$$= 3.1\ h$$

iii. Detention time at max design flow

$$= \frac{2535}{0.452 \times 3600}$$

$$= 1.5h$$

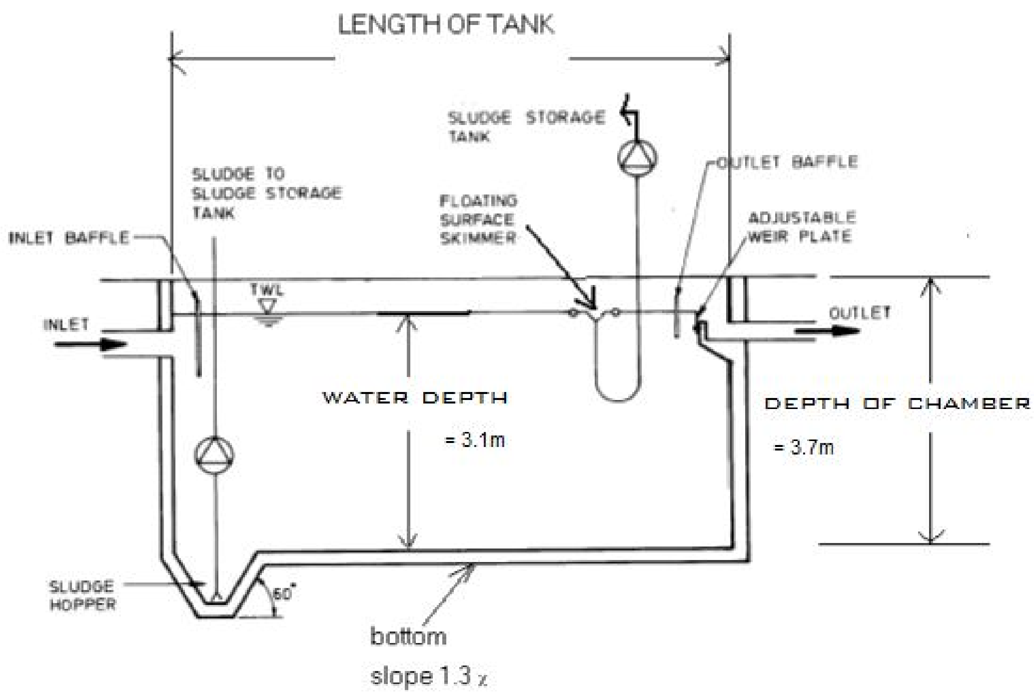

Fig. 4: Primary Sedimentation Tank

3.7 AERATION BASINS AND SLUDGE GROWTH

Provide two tanks for aeration with a common wall.

i. Average flow Q = 3254 m^3/day

ii. MLVSS/MLSS (F/M) = 0.8

iii. Assume influent BOD_5 = 200 mg /l

iv. MLVSS = 3000 mg /l

v. $F/_M = {}^{Q}/_{V} . {}^{Y_O}/_{X_t}$

$$0.8 = {}^{3254}/_{V} . {}^{200}/_{3000}$$

V = 271.17 m^3

vi. Take depth of flow = 3.0 m

vii. Volume for each basin = $^{271.2}/_{2}$

= 136 m^3

viii. Surface area = $^{Volume}/_{depth\ of\ basin}$

= $^{136}/_{3}$

= 45.3 m^2

ix. Length to width ratio = 2:1

Area = $2W^2$

Width = 5.0 m

Length = 9.6 m

x. Volume provided = 9.6 × 4.8 × 3

= 138.24 m^3

3.8.1. Detention time

i. $t_D = {}^{V}/_{Q} . 24h$

= $^{138.24 \times 24}/_{3245}$

= 1.02 h

3.8.2 Volumetric loading

i. Loading = $^{Q.Y_o}/_{V}$

= $^{3254 \times 200}/_{138.24}$

= 4707 g/m^3

= 4.7 kg/m^3

3.8.3 Check for Sludge ratio

i. $Q_r/_Q = X_t/(10^6/_{SVI} - X_t)$

ii. X_t= 3000 mg /l

iii. SVI = (50-150)mg/l

Take 100 mg /l

iv. Ratio = $3000/(10^6/_{100} - 3000)$

= 0.43

3.8 SECONDARY CLARIFIER

Three clarifiers were provided and was designed for the average design flow and recirculated flow. Each of the clarifiers will have an independent operation with respect to the aeration basins.

i. Average flow = 3254 m^3/day

ii. BOD = 200 mg /l

iii. Take **recirculated flow** = 50%

= 1627 m^3/day

iv. Total inflow = 4881 m^3/day

v. Detention time = 2 h

vi. Volume = $total\ inflow\ \times\ detention\ time$

= $4881 \times 2/_{24}$

= 406.75 m^3

Exclusive of hopper portion

vii. Assume liquid depth = 3.5 m

viii. Surface area = $^{406.75}/_{3.5}$

= 116.21 m^2

ix. Given 3 clarifiers in use, actual surface area = 116.21 × 3

= 348.63 m^3

x. Surface loading rate = $^{Average\ flow}/_{Surface\ area}$

= $^{3254}/_{348.63}$

= 9.3 $\therefore OK$

3.9.1 Geometry of Clarifier

i. Diameter = $\sqrt{^{Surface\ area\ \times 4}/_{\pi}}$

= $\sqrt{(^{116.21\ \times 4}/_{\pi})}$

= 12.2 m

ii. Depth = $^{406.75}/_{116.21}$

= 3.5 m

iii. For additional safety provide a freeboard = 0.5 m

iv. Total depth of clarifier = 4 m

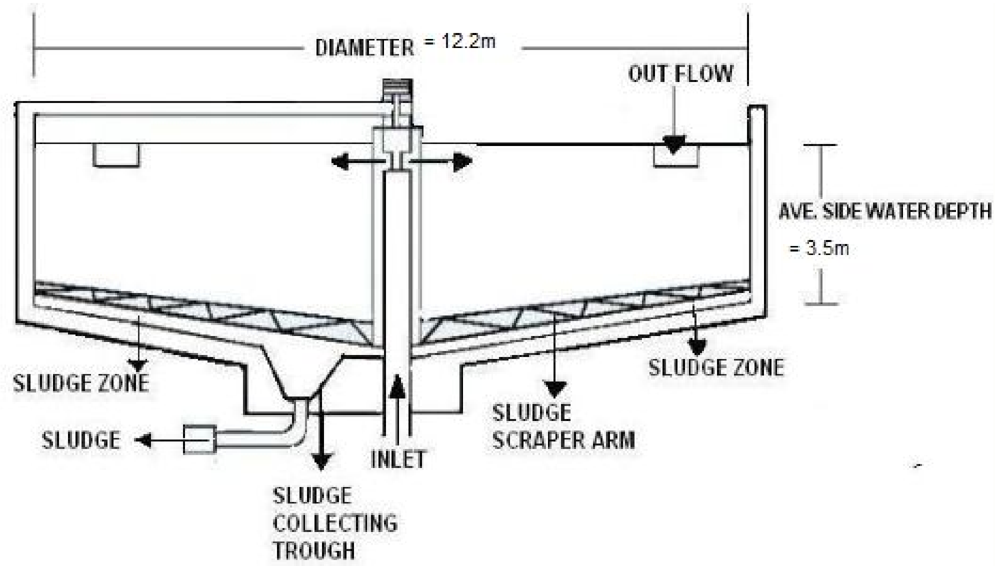

Fig. 5: Secondary Clarifier

3.9 SLUDGE DRYING BEDS

i. Recirculated flow = 1627 m^3/day

ii. Average flow = 3254m^3/day

iii. MLVSS in the tank = 3000 mg /l

iv. Total solids inflow = (1627+3254)×3
= 14,643 kg /day

v. Solids loading = $^{14643}/_{348.63}$
= 42 kg / day.m^2

vi. Average quantity of sludge produced from the 3 clarifiers
= 42×3
126 kg / day.m^2

vii. Assume solid content in the feed= 1.5%

viii. Specific gravity of the sludge= 1.015

ix. Volume of sludge= $^{126}/_{1.015 \times 1.5\% \times 1000}$

= 8.27 m^3/day

v. Assume the depth of sludge over drying bed = 0.25m

vi. Surface area = $^{8.27}/_{0.25}$

=33.08 m^2/day

vii. Provide 2 beds

Surface area for each = 16.54 m^2/day

x. Assume length of each bed = 7m

∴width= 2.4 m

Thus providing 16.8 m^2

CHAPTER FOUR

4 DISCUSSIONS

4.1 SEWER DESIGN

The pipe material was chosen as concrete because it allows for larger pipe sizes. The problem of corrosion will also not be a big issue because there is no industrial wastewater being collected from the housing development.

In the analysis of the sewer pipe design, the slope was too small to achieve the minimum required velocity which is 0.6 m/s. The slope is therefore be increased in order to achieve the minimum flow velocity.

4.2 TREATMENT PLANT DESIGN

The treatment plant was placed in the lowest lying area where all trunk sewers could converge while transporting water by gravity flow. This is to minimize on pumping costs.

CHAPTER FIVE

5 CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSION

The sizes of sewers is highly affected by the population density, population growth and design period and the design parameters should be carefully considered. The sizes of the parts of a wastewater treatment plant is affected by the amount of wastewater entering the treatment plant.

5.2 RECOMMENDATIONS

1. In addition to physical, chemical and biological treatment processes, there is are advanced treatment methods used to remove those constituents which are not adequately removed, which can be used for cooling in industries.
2. A disinfection unit could be added to this design project to get a higher quality of water.

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APPENDICES