DESGN OF WASTEWATER TREATMENT PLANTS

DESIGN PROJECT

GROUP 06| SPRING 2012

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PROJECT SUMMARY

This report contains design of wastewater treatment plant units in accordance with the course material of ENV 5517: Design of Wastewater Treatment Plants. The key directions for the design projects are

Population (2011) :40000

Design Period: 50 years

Polulation Increase : 0.7 %/year

The plant to be constructed in two stages

stage 1: Design period 25 years (1-25 years of operation)

stage 2: Design period 25 years (26-50 years of operation)

Peaking Factor: 2.5

Min Flow: 0.5 of Design flow

The design is undertaken according to the ten state standards following class lectures by Dr. Berrin Tansel. The design team sincerely acknowledges her guidance and thanks her for continuous support and inspiration.

PART A

RESUME OF TEAM MEMBERS

PART B

DESIGN OF TREATMENT PLANT UNITS

1.1 Introduction

Bar screens are typically at the headworks (entrance) of a wastewater treatment plant (WWTP), bar screens are used to remove large objects such as rags, plastics bottles, bricks, solids, and toy action figures from the waste stream entering the treatment plant. Bar screens are vital to the successful operation of a plant, they reduce the damage of valves, pumps, and other appurtenances. Floatables are also removed at the entrance to a treatment plant, these are objects that "float" on the surface of the water and if aren't removed end up in the primaries or aeration tanks.

Fig 1.1 : Bar Screen in operation

There are various types of bar screens available for installation, they include but not limited to chain bar screens, reciprocating rake bar screens, catenary bar screens, and continuous belt bar screens.

1.2 Review of Ten State Standards

1. According to 10state standard, article 61.121, Clear openings between bars should be no less than 1 inch (25 mm) for manually cleaned screens. Clear openings for mechanically cleaned screens may be smaller. Maximum clear openings should be 1¾ inches (45 mm).

2. According to 10state standard, article 61.122, velocities should be no greater than 3.0 fps (0.9 m/s) to prevent forcing material through the openings.

3. According to 10state standard, article 61.122, Manually cleaned screens should be placed on a slope of 30 to 45 degrees from the horizontal.

4. According to 10state standard article 61.124, Where two or more mechanically cleaned screens are used, the design shall provide for taking any unit out of service without sacrificing the capability to handle the design peak instantaneous flows.

1.3 Related Photographs

(i) Chain Bar Screen

(ii) Reciprocating Bar Screen

(iii) Catenary Bar Screen

(iv)Continuous belt Bar Screen

Fig 1.2: Different types of bar screens

1.4 Design of Bar Screen

Arithmetic method has been adopted for population projection as shown below. Typical value has been assumed for per capita consumption. Since the plant is to be constructed in two stages, design flow for 25 years has been used in preliminary design of bar screen and grit removal.

Current population = 40,000
Rate of Increase = 0.7%
Projected population after 25 years= 40,000 + 40000*25*(0.7/100)
= 47,000
= 50,000 (rounding up)

Consumption per capita-day= 100 gallon
Average flow= 50000*100
= 50,0000 gallon/day
= (50000* 0.133680556)/86400 ft^3/s
= 7.74 ft^3/s
Peak factor= 2.5

Peak flow = 7.74*2.5
= 19.35 or 20 ft^3/s

Let's Assume,

3 bar screens will be used with each having a peak flow = 20/3 = 7 ft^3/s
Approach velocity = 2 fps
and width = 3.5 ft

So, depth= Peak flow/(width*velocity)
= 7/(2*3.5)=1 ft

Assume , 1.5 inch bar spacing and 0.5 in bar width
N_{bars}= (width of channel-bar spacing)/(bar width + bar spacing)
= (3.5*12-1.5)/(1.5+0.5)
= 20
Number of bar spaces= 20+1
= 21
Total area of opening= 21*bar spacing*depth
= 21*(1.5/12)*1
= 2.625 sq ft
Velocity through the opening= Q/ area of opening
= 7/2.625
= 2.66 fps <3 fps

Assume, slope= 45˚
Height of the rack= 1 ft/Sin 45
= 1.4 ft
Allowing 0.6 ft freeboard, height of the rack= 2.0 ft
Head loss of the rack = $(V_{opening}^2 - V_{approach}^2)/C*2g$
= $(2.66^2-2^2 0/(0.7*2*32.14)$
= 0.07 ft

Maximum allowable head loss is 0.6m-0.7m. So head loss is acceptable.

If one unit is taken out peak flow will be =	20/2 = 10 cu ft/s
Total area of opening (including freeboard) =	21*2*sin45*(1.5/12)= 3.71
Velocity through opening=	10/3.71=2.7 fps < 3 fps
	So design is acceptable.

1.5 Design Summary

Table 1.1 Bar Screen Design Summary

Design summary
3 manually cleaned bar screens.
3.5 ft width and 2 ft depth channel for each
21 bars with 1.5 inch spacing and 0.5 inch bar width for each

2.1 Introduction

An Aerated Grit Chamber removes grit from wastewater streams. Airflow is generated by a blower and is introduced into the Aerated Grit Chamber via a tube which is located near the bottom of the chamber, thereby creating a circular or toroidal flow pattern in the wastewater. The continuous rising flow deflects off an energy-recovery baffle at the liquid surface. This flow pattern causes the grit to settle to the bottom of the chamber while keeping lighter organic material in suspension to be processed further downstream. Once the grit has settled, either a recessed-impeller grit pump or, more commonly, an air-lift pump is used to remove the grit slurry and send it on for dewatering.

2.2 Review of Ten State Standards

1. According to 10state standard article 63.3, Detention time in the tank should be in the range of 3 to 5 minutes at design peak hourly flows.

2. According to typical design information for aerated grit chambers provided in the class lecture, Range for depth is 7-16 ft, width 8-23 ft and length 25-65 ft and width :depth= 1:1-5:1, length : width ratio 3;1-5:1.

3. According to 10state standard article 63.43, Channel-type chambers shall be designed to control velocities during normal variations in flow as close as possible to 1 foot per second (0.3 m/s).

2.3 Related Photographs

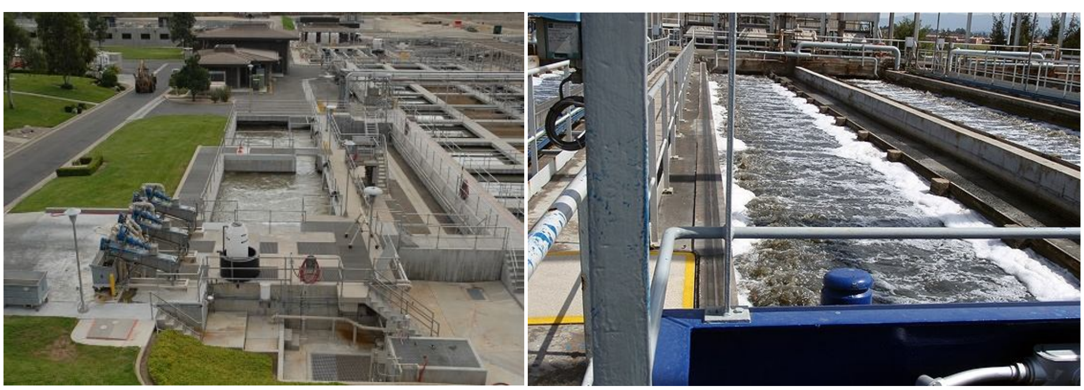


Fig 2.1: Aerated grit chamber in operation

2.4 Design of Aerated Grit Chamber

Assuming three aerated grit chambers, so

Design, Q= (20/3)= 7ft^3/s

Assume 4 minutes for peak hourly flow

Detention time= 240 s

Volume= Q*t=7*240

= 1680 cu ft

Assume, Depth= 84 in Or 7 ft

width to depth ratio= 1.2:1

Width= 7*1.2= 8.4 ft or 8.5 ft

Length= Volume/(Depth*Width)

= 1680/(7*8.5)

= 28 ft

length to width ratio= 28/8.5=3.3:1

(within typical limit)

Checking horizontal velocity

Velocity= Q/A= 7/(7*8.5)

= 0.12 fps

On the low side but acceptable.

Air supply, Using 5 ft^3/m of length

=28*5

=140 ft^3/min

2.5 Design Summary

Table 2.1 Grit Chamber Design Summary

Design Summary
Three horizontal aerated grit chamber in use.
Dimension= 7 feet * 8.5 feet * 28 feet
Air supply 140 ft^3/min

3.1 Introduction

Parshall Flumes are used for metering liquids in open channels, the unbroken flow lines do not obstruct flow which can cause buildup of debris. The unbroken lines also result in low head loss characteristics which make the flume useful for measuring under gravity head situations and measuring liquids.

The Parshall flume consists of a metal or concrete channel structure with three main sections: (1) a converging section at the upstream end, leading to (2) a constricted or throat section and (3) a diverging section at the downstream end.

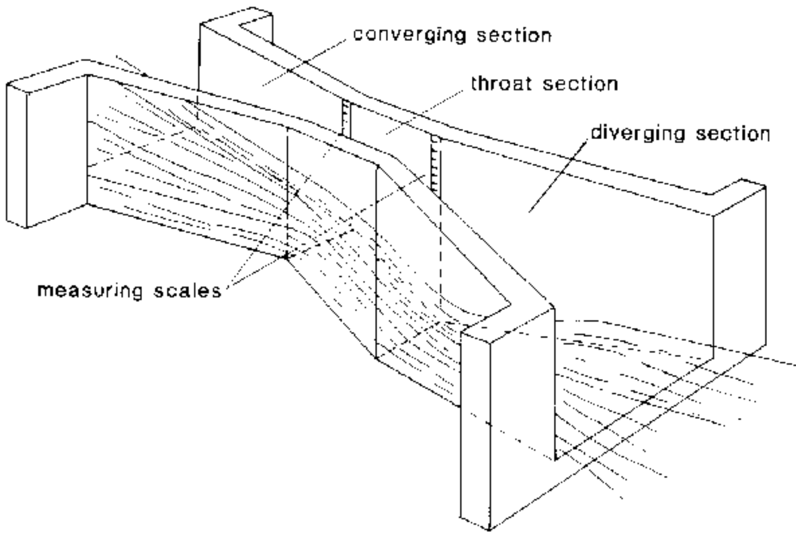


Fig 3.1: schematic of Parshall flume

3.2 Related Photographs

Fig 3.2 Parshall flume in operation

3.3 Design of Parshall Flume

Peak flow= 20 ft^3/s
= 20*0.0283168466*60*60
= 2039 m^3/hr
= 0.57 m^3/s

Using table 20.2 (Mackenzie & Devis, 2008) for 50-2450 m^3/hr

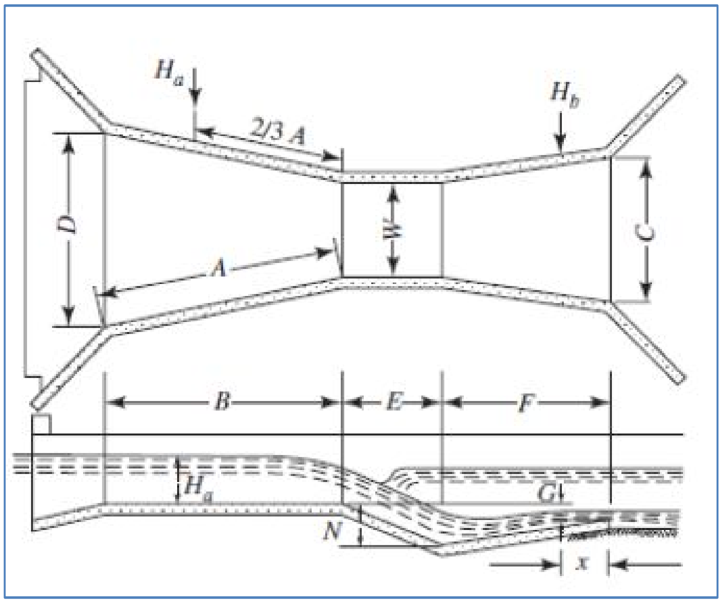
Throat width,w= 0.46

Using figure 20.3A and 20.3B ((Mackenzie & Devis, 2008)

C= 2
N= 1.54

Depth of water H_a= $(Q/C)^{1/n}$
= $(0.57/2)^{1/1.54}$
= 0.45 m or 1.45 ft

Depth of flume, H= 2.5 ft (including freeboard)



Using table 20.2 (Mackenzie & Devis, 2008), other dimensions

	A	B	C	D	E	F	G	N	X
m	1.45	1.42	0.76	1.03	0.61	0.91	76	229	51

4.1 Introduction

The Primary Clarifier is a basin where water has a certain retention time where the heavy organic solids can sediment (suspended solids). Efficiently designed and operated primary sedimentation tanks should remove from 50 to 70 percent of the suspended solids and 25 to 40 percent of the BOD.

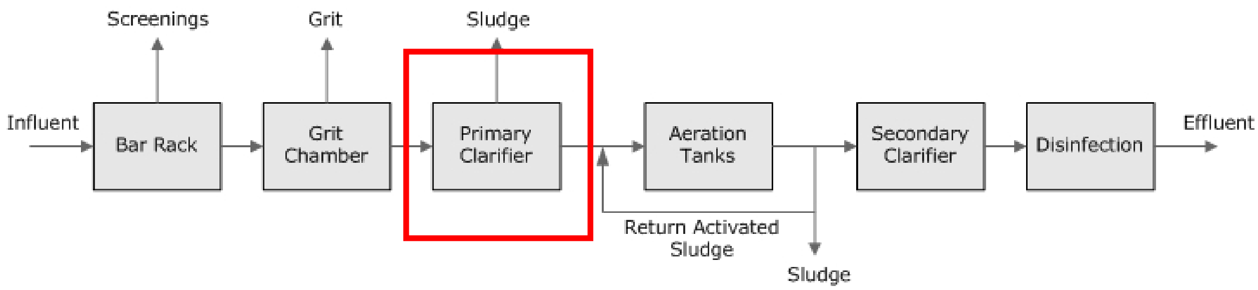


Fig 4.1 Position of Primary Clarifier in process diagram

Primary clarifier consists of sedimentation unit, inlet and outlets. Based on experience and technical knowledge several dimensions and design criteria of primary clarifier have been suggested. 10 states standards have been considered as the basis for design of parameter in Unites states although in academic books one can find some other typical design parameters.

4.2 Review of Ten State Standards

Table 1.1: Comparison of standards for Primary Clarifier

Design Parameter	10 States Standards	According to (Davis, 2011)
Number of Units	Multiple units capable of independent operation are desirable and shall be provided in all plants where design average flows exceed 100,000 gallons/day (380 m^3/d)	Multiple Units
Depth of Settling Tank	Minimum 10 ft	10-15 ft
Length : Width	4-5:1	4:1-6:1
Surface Overflow Rate	61-81 m^3/day.m^2	40-70 m^3/day.m^2
Velocity		0.005-0.018 m/s
Weir Loading	Maximum 250 m^3/day.m	140-320 m^3/day.m
Free board of settling tank	6-12 inch	

4.3 Related Photographs


Primary Clarifier Weir


Primary clarifier scum trough


Typical View of Primary Clarifier


Primary Clarifier Weir

Fig 4.2: Primary Clarifier in operation

4.4 Design of Primary Clarifier

Design Flow Rate (Q) $= 20\ ft^3/s$
Detention Time (T) = 45 min

Assume,

Depth (D) = 10 ft
Width (W) = 20 ft
Length (L) = 5 x width
= 5 x 20 ft
= 100 ft

Volume of the one tank, (V_{ind}) = Length × Width × Depth
$= (100 \times 20 \times 10)\ ft^3$
$= 20000\ ft^3$

Required Volume, (V) $= 20\ ft^3/s \times (45 \times 60)\ s$
$= 54000\ ft^3$

Number of tanks required, (n) = Required Volume/ Volume of the one tank

= V/ V_{ind}

= 54000 ft^3/20000 ft^3

= 2.7

So, 3 tanks will be provided.

Check for Overflow Rate

For each tank,

design flow rate, (Q_{ind}) = 20/3 = 6.67 ft^3/s

Overflow rate $= \frac{Q_{ind}}{W \times L}$

$$= \frac{6.67\, ft^3}{1s} \times \frac{1}{100\, ft \times 20\, ft} \times \frac{0.3048m}{1\, ft} \times \frac{60s \times 60m \times 2}{1day}$$

= 87 m^3/ *m^2-day for peak flow.*

The overflow rate is slight higher but satisfactory.

Check for Flow Through Velocity

Flow through velocity $= \frac{Q_{ind}}{D \times W}$

$$= \frac{6.67\, ft^3}{1s} \times \frac{1}{10\, ft \times 20\, ft} \times \frac{0.3048m}{1\, ft}$$

= 0.0102 m/s

The flow through velocity is within the typical range recommended by reference and hence satisfactory.

Design of Weir

Length of the weir = L/3

=100/3 = 33.3 ft

Provide spacing of weir = 5 ft

Number of weir = 4

Check for Weir Loading

weir overflow rate $= \frac{20\, ft^3}{1s} \times \frac{1}{3} \times \frac{1}{4 \times 33.33\, ft \times 2} \times \left(\frac{0.3048m}{1\, ft}\right)^2 \times \frac{(60 \times 6}{1}$

= 200 m^3/day-m

The weir overflow rate is within recommendations of 10 state standard and so design of weir is satisfactory.

4.5 Design Summary

Table 4.1 Primary Clarifier Design Summary

Design Summary	
Number of Tanks	3
Length of each Tank	100 ft
Width of each Tank	20 ft
Depth of flow	10 ft
Total Depth of each Tank	12 ft
Number of Weir in each tank	4
Weir spacing C/C	5ft
Length of each Weir	33.3ft

5.1 Definition

An aeration tank is a place where a liquid is held in order to increase the amount of air within it. The most common uses of aeration tanks are in wastewater recovery, as the high oxygen levels will increase the speed at which the water is cleaned.

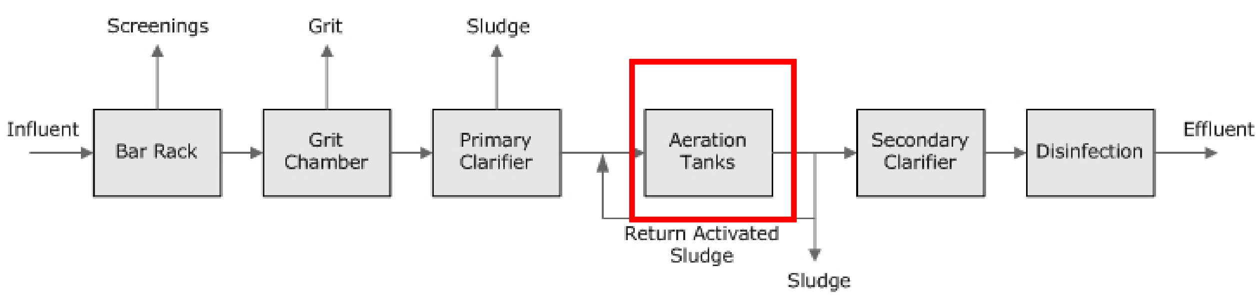


Fig 5.1 Position of Aeration Tank in process diagram

The primary purpose of the aeration tank is to convert BOD to SS. In the Aeration Tank the mixed liquor is aerated. By aerating the mixed liquor the aerobic processes will be stimulated, the growth rate of bacteria will be must faster.

5.2 Review of Ten state standards

1. Maintain a minimum of 2.0 mg/L of dissolved oxygen in the mixed liquor at all times throughout the tank or basin;

2. Maintain all biological solids in suspension (for a horizontally mixed aeration tank system an average velocity of 1 foot per second (0.3 m/s) must be maintained);

3. Meet maximum oxygen demand and maintain process performance with the largest unit out of service;

4. Provide for varying the amount of oxygen transferred in proportion to the load demand on the plant; and

5.3 Related Photographs

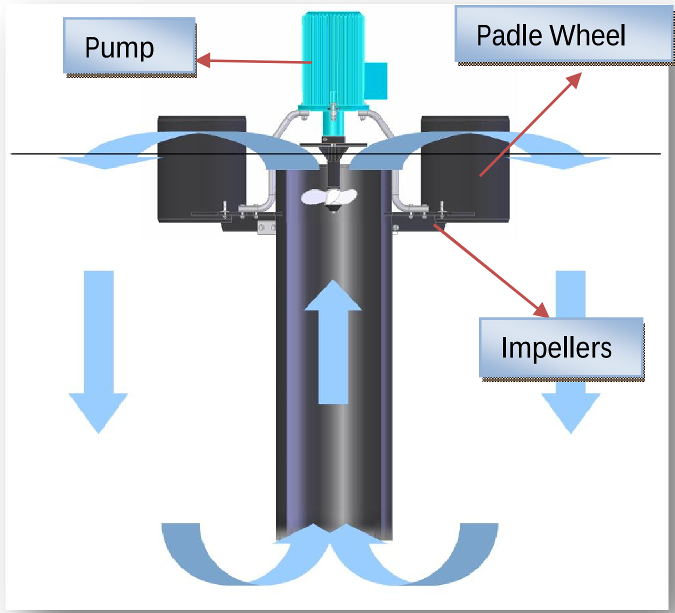


Fig 5.2 : Surface Aerator

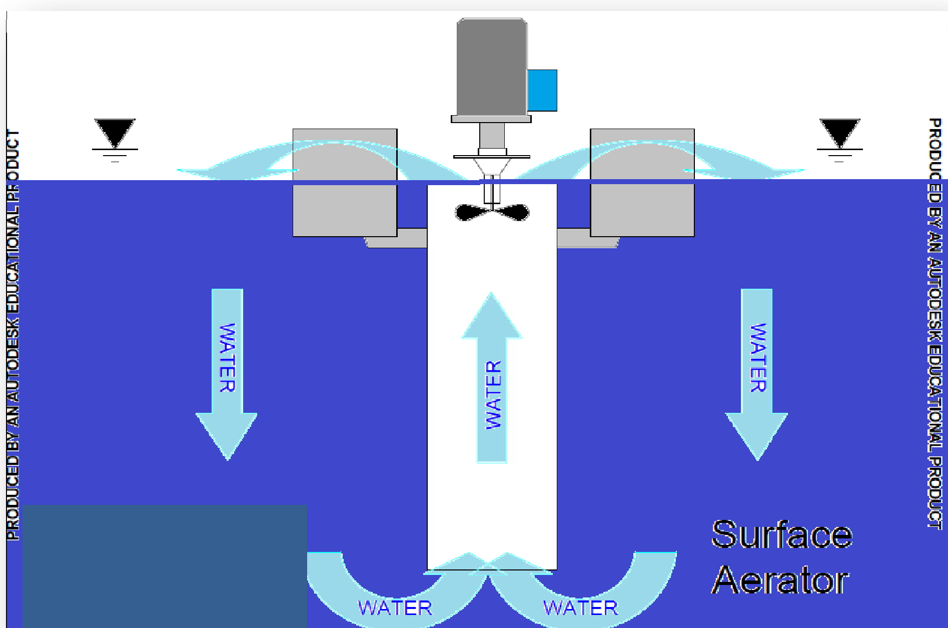
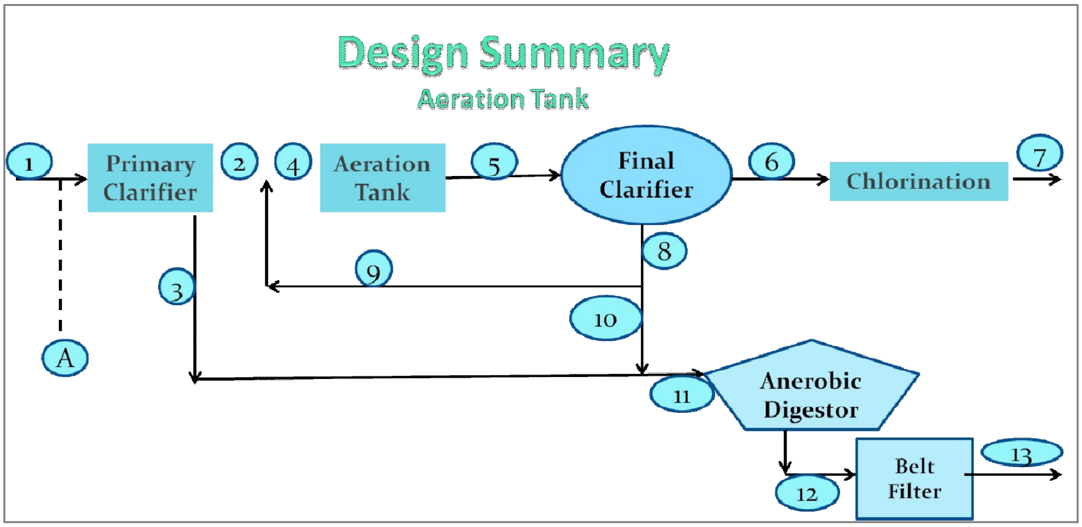


Fig 5.3: Surface aerator typical configuration



5.4 Design of Aeration Tank

Design flow =Population* Per capita consumption
= 50000*100 gpd = 5 MGD

SS loading =Population* Per capita loading
= 50000*0.2 lb/capita-day
= 10000 lb/day

BOD loading =Population* Per capita loading
= 50000*0.17 lb/capita-day = 8500 lb/day

Since, Primary Clarifier removes 50% BOD and 30% Suspended Solids

Solids in primary effluent =10000*(1-0.5)
= 10000*0.5
= 5000 lb/day

BOD in primary effluent =10000*(1-0.3)
= 8500*0.7 = 5950 lb/day

BOD concentration $= \frac{5950 \text{ lb/day}}{5 \text{ MGD} * 8.34}$
= 142.7 mg/L

SS concentration $= \frac{5000 \text{ lb/day}}{5 \text{ MGD} * 8.34}$
= 119.9 mg/L

Volume of Aeration Tank

(a) Based on detention time, V_{at} $= Q_2 * t$

$= 5\text{ MGD} * 6\text{hr} * \frac{1\text{ day}}{24\text{hrs}}$

$= 1.25$ MG

(b) Based on BOD loading, V_{at} $= \frac{\text{lbs of BOD}}{\text{BOD loading}}$

$= \frac{5950 \text{ lb/day}}{\frac{40 \text{ lbs}}{1000 \text{ ft}^3}}$ (assuming BOD loading 40 lbs/1000 ft^3

$= 148750\ ft^3$

$= 1.11$ MG

Area of Aeration Tank

Assume, Depth $= 12$ ft

Area required, A_{req} $= \frac{1.25 \text{ MGD} * 10^6 \text{ gal} * 1 \text{ ft}^3}{12 \text{ ft} * 1 \text{ MG} * 7.48 \text{ gal}}$

$= 13927\ ft^2$

Assuming folding in segments of 20 ft* 100 ft

Each segment = 20 ft *100 ft = 2000 ft^2

Number of folds = 13927/2000 = 6.96 = 7

Total area provided = 20 ft *100 ft* 7 = 14000 ft^2

Total volume = 14000 ft^2*12 ft

= 168000 ft^3

= 1.253 MG

F/M ratio calculation

Assume, MLVSS/MLSS =0.8

M = V_{at} * MLSS*0.8*8. 34

=1.253*3500*0.8*8.34

= 29260 lb

F/M ratio $= \frac{\text{lbs of BOD}}{\text{lb of MLVSS in AT}}$

= 5950/29260

= 0.2day^{-1}

Solid Produced = BOD loaded* Y

= 5950 * 0.4

=2380 lb/day (to be wasted)

Sludge Age Calculation

Solid Lost = Q*C*8.24
= 5*20*8.34
=834 lb/day

Sludge age = MCRT

$$= \frac{\text{lbs of solids in AT}}{\text{lbs of solid lost}}$$

$$= \frac{\text{Vat} * \text{C}_{\text{MLSS}} * 8.34}{\text{lbs of solid}}$$

$$= \frac{1.253 * 3500 * 8.34}{2380 + 834}$$

= 12 days

Air Requirement Calculation

Air Requirement = 1.1 lbs of Oxygen*BOD applied
=1.1*5950 lb/day
= 6545 lbs Oxygen /day

Air = 6545/0.22
= 29750 lbs air/day

Mechanical Aerator Design

Where, N =oxygen transfer rate under filed condition, Kg/KW.h

N_o =Oxygen transfer rate provided by manufacturer, kg/KW.h

β =Salinity Surface Tension correction factor = 1

Cw =Osygen saturation concentration at given altitude and temperature, mg/L

C_L =Operating DO concentration

C_{s20} =Oxygen saturation concentration at 20^0C, mg/L

T = temperature

α =Oxygen transfer correction factor for wastewater= 0.8 to 0.9

Design of Mechanical Aerator

$$N = N_0 (\frac{\beta C_w - C_L}{C_{s20}}) 1.024^{T-20} \alpha$$

Where, N = Oxygen transfer rate under filed condition, Kg/KW.h
N_o= Oxygen transfer rate provided by manufacturer, kg/KW.h
β=Salinity Surface Tension correction factor = 1
C_w=Osygen saturation concentration at given altitude and temperature, mg/L
C_L=Operating DO concentration
C_{s20}=Oxygen saturation concentration at 20^0C, mg/L
T= temperature
α=Oxygen transfer correction factor for wastewater= 0.8 to 0.9
<u>Design of Mechanical aerator</u>
N_0=2 kg Oxygen/KW-H (assuming)
β=1
α=0.9
C_L=2.0 mg/L
C_w=8.38 mg/L (25 ^{0}C)
C_{s20}=9.02
$N = 2(\frac{1*8.38-2}{9.02})1.024^5 * 0.9$ =1.433 kg/KW-h=34.40 kg/Kw-day
<u>Power Requirement</u>
Oxygen Requirement = 6545 lb/day
Power Required = N/Oxygen Requirement= $\frac{6545lb}{day} * \frac{1KW - day}{34.40kg} * 0.4535\frac{kg}{lb}$ =86.2 KW5.5

5.5 Design Summary

Table 5.1 Aeration Tank Design Summary

Design Summary	
Volume of AT	1.25 MG
Sludge age	12 days
Air required	29750 lbs/ day
Power required	86 KW

6.1 Introduction

From its name alone, many would believe that secondary treatment is simply a second step of primary treatment. However, it differs significantly in the type of treatment provided. Through physical processes driven by gravity, primary treatment consists of a separation of solids and liquids. Secondary treatment is related to a biological processing of the organic waste products contained within the waste stream. One can argue that the final or secondary clarifier is one of the most important unit processes of the treatment process. It often determines the capacity of the plant.

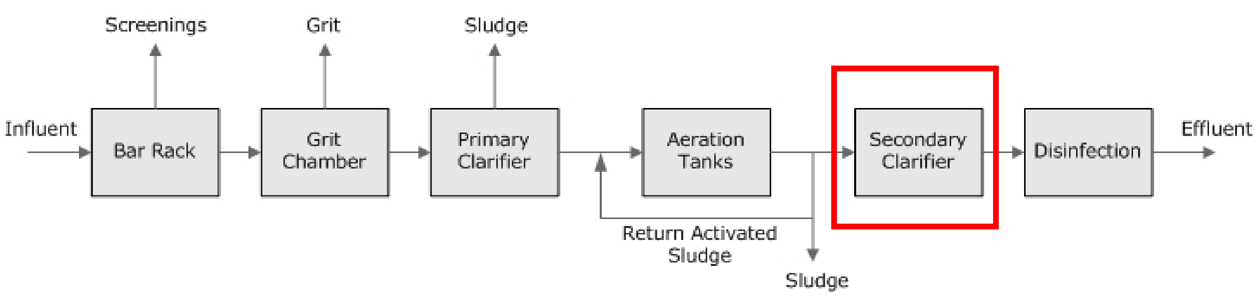


Fig 6.1 Position of Secondary Clarifier in process diagram

6.2 Review of Ten state standards

Settling tests shall be conducted wherever a pilot study of biological treatment is warranted by unusual waste characteristics, treatment requirements, or where proposed loadings go beyond the limits set forth in this Section.

Final Clarifier - Attached Growth Biological Reactors

Surface overflow rates for settling tanks following trickling filters or rotating biological contactors shall not exceed 1,200 gallons per day per square foot [49 $m^3/(m^2 \cdot d)$] based on design peak hourly flow.

Final Clarifier - Activated Sludge

To perform properly while producing a concentrated return flow, activated sludge settling tanks must be designed to meet thickening as well as solids separation requirements. Since the rate of recirculation of return sludge from the final settling tanks to the aeration or re-aeration tanks is quite high in activated sludge processes, surface overflow rate and weir overflow rate should be adjusted for the various processes to minimize the problems with sludge loadings, density currents, inlet hydraulic turbulence, and occasional poor sludge settleability. The size of the settling tank must be based on the larger surface area determined for surface overflow rate and solids loading rate. The following design criteria shall not be exceeded:

Table 6.1 : 10 state standards for Secondary Clarifier

Treatment Process	Surface Overflow Rate at Design Peak Hourly Flow (gpd/ft^2)	Peak Solids Loading Rate (lb/day/ft^2)
Conventional, Step Aeration, Complete Mix, Contact Stabilization, Carbonaceous Stage of Separate Stage Nitrification	1,200	50
Extended Aeration Single Stage Nitrification	1,000	35
2 Stage Nitrification	800	35
Activated Sludge with Chemical addition to Mixed Liquor for Phosphorus Removal	900	As Above

6.3 Related Photographs

Clarifier peak solids loading rate shall be computed based on the design maximum day flow rate plus the design maximum return sludge rate requirement and the design MLSS under aeration.

Fig 6.2: Secondary clarifier in operation.

6.4 Design of Secondary Clarifier

Gravity solids flux calculation is shown below,

V (ft/hr)	C (mg/L)	C (lb/cu ft)	G (lb/ft^2-hr)
7	2000	0.124856	0.873991
4	3000	0.187284	0.749136
2	3700	0.230983	0.461967
1	5800	0.362082	0.362082
0.5	7900	0.493181	0.24659

Design flow =Population* Per capita consumption
= 50000*100 gpd = 5 MGD
=668402.78 cu ft/day

Assume,

Clarifier surface diameter, D =125 ft

Clarifier surface area =(pi/4)* D^2
=12271 sq ft

Overflow surface flux rate, SF = (668402.78/12271)*X

Determining the plotting point for 3250 lb/cu ft (from graph 1)

SF =(668402.78/12271)*3250*6.24279*10^{-6} *(1/24)
= 0.46 lb/ft^2-hr
= 5000 lb/day

Drawing limiting solids flux by drawing a line from 10000 g/m^3 or 0.62 lb/ft^3 tangent to the gravity flux curve shows,

limiting solids flux = 0.77 lb/ft^2-hr

drawing a line from 10000 g/m^3 or 0.62 lb/ft^3 to the state point shows,

Flux rate= = 0.7 lb/ft^2-hr

So, the clarifier is underloaded.

Underflow velocity =(0.7/0.6248)
=1.12 ft/hr

Clarifier Overflow Rate = 688402.78/(12271.85*24)
=2.27 ft/hr

Recycle Ratio = 1.12/2.27
=0.49

6.5 Design Summary

Table 6.2 : Design summary of Secondary Clarifier

Design Summary	
Surface Diameter	125 ft
Flux Rate	0.7 lb/ft^2-hr
limiting solids flux	0.7 lb/ft^2-hr
Underflow velocity	1.12 ft/hr
Overflow rate	2.27 ft/hr
Recycle Ratio	0.49

7.1 Introduction

Chlorination is the process of adding the element chlorine to water as a method of water purification. From a chemical standpoint, the reaction is as follows:

When chlorine is added to water, it reacts to form a pH dependent equilibrium mixture of chlorine, hypochlorous acid and hydrochloric acid

$$Cl_2 + H_2O \rightarrow HOCl + HCl$$

Hypochlorous acid partly dissociates to hydrogen and hypochlorite ions, depending on the pH.

$$HOCl \rightarrow H^+ + ClO^-$$

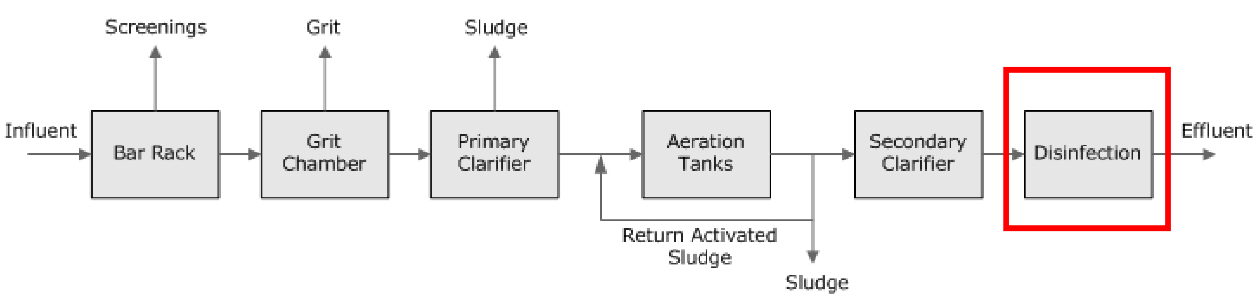


Fig 7.1 Position of Chlorination in process diagram

7.2 Review of ten state standards

Chlorine is available for disinfection in gas, liquid (hypochlorite solution), and pellet (hypochlorite tablet) form. The type of chlorine should be carefully evaluated during the facility planning process. The use of chlorine gas or liquid will be most dependent on the size of the facility and the chlorine dose required.
For disinfection, the capacity shall be adequate to produce an effluent that will meet the applicable bacterial limits specified by the regulatory agency for that installation. Required disinfection capacity will vary, depending on the uses and points of application of the disinfection chemical. The chlorination system shall be designed on a rational basis and calculations justifying the equipment sizing and number of units shall be submitted for the whole operating range of flow rates for the type of control to be used.

Table 7.1 : 10 state standards for Chlorination Tank

Type of Treatment	Dosage
Trickling Filter Plant Effluent	10 mg/L
Activated sludge plant effluent	8 mg/L
Tertiary filtration effluent	6 mg/L
Nitrified effluent	6 mg/L

7.3 Related Photographs

Chlorine Contact Basin

Effluent from chlorination

Fig 7.2: Chlorination in process

7.4 Design Calculation

Assume, Detention time, t= 15 min

Required volume, V= Avg Flow * Peak Factor * Detention Time

$$= 5MGD * 2..5 * \frac{15\min}{60\min} * \frac{1day}{24hr}$$

= 0.1302 MG

Assume, Depth= 10 ft

Required Area, A= $0.1302MG * 10^6 * \frac{1\,ft^3}{7.48gal} * \frac{1}{10\,ft}$

= 1740 Sq ft

Assume 2 chlorination tanks.

Area of Each Tank= (1740 sq ft/2)= 870 sq ft

7.5 Design Summary

Table 7.2: Design Summary for Chlorination Tank

Design Summary	
Detention Time=	15 min
No of Tanks=	02
Volume of Each Tank=	0.065 MG
Depth of Each Tank=	10 ft
Area of Each Tank=	870 sq ft

8.1 Introduction

Anaerobic digestion is the most common process for dealing with wastewater sludge containing primary sludge. Anaerobic digestion is preferred to reduce the high organic loading of primary sludge because of the rapid growth of the biomass that would ensue if the sludge were treated aerobically. Anaerobic decomposition creates considerably less biomass than the aerobic process. Anaerobic digestion converts as much of the sludge as possible to end products, such as, liquid and gases while producing as little residual biomass as possible. The liquids, in the form of supernatant, from the digester are sent back through the plant for further treatment. An anaerobic sludge digester is designed to encourage the growth of anaerobic bacteria, particularly the methane producing bacteria that decreases organic solids by reducing them to soluble substances and gases, mostly carbon dioxide and methane.

8.2 Review of ten state standards

- Multiple units or alternate methods of sludge processing shall be provided. Facilities for sludge storage and supernatant separation in an additional unit may be required, depending on raw sludge concentration and disposal methods for sludge and supernatant.
- If process design provides for supernatant withdrawal, the proportion of depth to diameter should be such as to allow for the formation of a reasonable depth of supernatant liquor. A minimum side water depth of 20 feet (6.1 m) is recommended.
- To facilitate emptying, cleaning, and maintenance, the following features are desirable:
- The tank bottom shall slope to drain toward the withdrawal pipe. For tanks equipped with a suction mechanism for sludge withdrawal, a bottom slope not less than 1 to 12 is recommended. Where the sludge is to be removed by gravity alone, 1 to 4 slope is recommended.
- At least 2 access manholes not less than 30 inches (760 mm) in diameter should be provided in the top of the tank in addition to the gas dome. There should be stairways to reach the access manholes.
- A separate side wall manhole shall be provided that is large enough to permit the use of mechanical equipment to remove grit and sand. The side wall access manhole should be low enough to facilitate heavy equipment handling and may be buried in the earthen bank insulation.

- Non-sparking tools, rubber-soled shoes, safety harness, gas detectors for flammable and toxic gases, and at least two self-contained breathing units shall be provided for emergency use. Refer to other safety items as appropriate in Section 57.
- If the anaerobic digestion process is proposed, the basis of design shall be supported by wastewater analyses to determine the presence of undesirable materials, such as high concentrations of sulfates and inhibitory concentrations of heavy metals.

8.3 Related Photographs

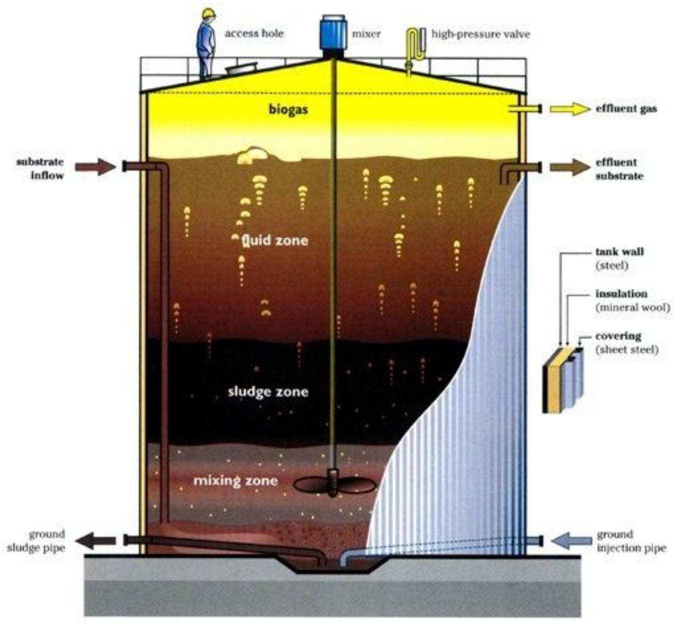


Fig: 8.1 Anaerobic Digester schematic

8.4 Design Calculation

Solids in Primary effluent= 5000 lbs/day
Solids in final effluent= 2380 lbs/day

Total solids in effluent= (5000+2380) lbs/day
= 7380 lbs/day

Assuming sludge solids are 80% volatile and 60% volatile solids destruction.
Volatile solids to digester= 7380 * 0.8
= 5904 lbs/day

Volatile solids destroyed= 5904*0.6

= 3543 lbs/day

Assuming, 8 cubic feet of gas is produced per pound of volatile solids to digester.

Gas production= 7380*8

= =59040 cubic ft

Assuming primary sludge contains 5% solids.

Primary effluent flow*50000 mg/L* 8.34= 5000

So, Primary effluent flow= $12 *10^{-3}$ MGD

Assuming , concentration of solids in secondary sludge 1 %

Flow rate of WAS* 10000 mg/L*8.34*1.02= 2380 [assuming sludge density 1.02]

So, Flow rate of WAS= $28 *10^{-3}$ MGD

Assuming 30 days of detention time

Volume needed= $(28+12)*10^{-3}*10^{6}*30$ days

= 1200000 gal

Assuming 3 digester of 20 ft depth

Area of each digester= $\frac{1200000}{7.48gal} * 1 ft^3 * \frac{1}{20 ft} * \frac{1}{3}$

2675 ft^3

Diameter= $(\frac{area * 4}{\pi})^{1/2}$

= 60 ft

Heat Calculation

Surface area of digester= (Roof + floor + side)area

= 6031 sq ft/digester

Total area= 18093 sq ft

Assuming ambient summer temperature 7º c and winter temperature 19º c.

Heat transfer loss in Summer= 72950 BTU/hr

Heat transfer loss in Winter= 127664 BTU/hr

Volume of Sludge Cake

Assuming,

Final sludge cake is 30% solids.

Density of sludge cake= 100 lbs/ft^3

Solids in digested sludge= Solids in effluent-Volatile solids destroyed

= 3837 lbs/day

Volume of Sludge cake= 3837/(8.34*30000*(100/62.4))

=9570 gpd

8.5 Design Summary

Table 8.1 : 10 state standards for Anaerobic Digester

Design Summary	
Number of digesters	3
Diameter	60 ft
Depth	20 ft
Detention time	30 days

9.1 Introduction

Drying beds are either planted or unplanted sealed shallow ponds filled with several drainage layers and designed for the separation of the solid from the liquid fraction of sludge. Unplanted drying beds are simple sand and gravel filters on which batch loads of sludge are dewatered. As for unplanted beds, planted beds consist of an impermeable shallow pit filled with different layers of coarse to fine sand. Normal operation of the system involved sludge being placed on a bed layer and then allowed for drying to take place by either water draining through the mass and supporting sand bed or evaporation from the surface. Since water drains through, having an advanced drainage system is a must.

Advantages

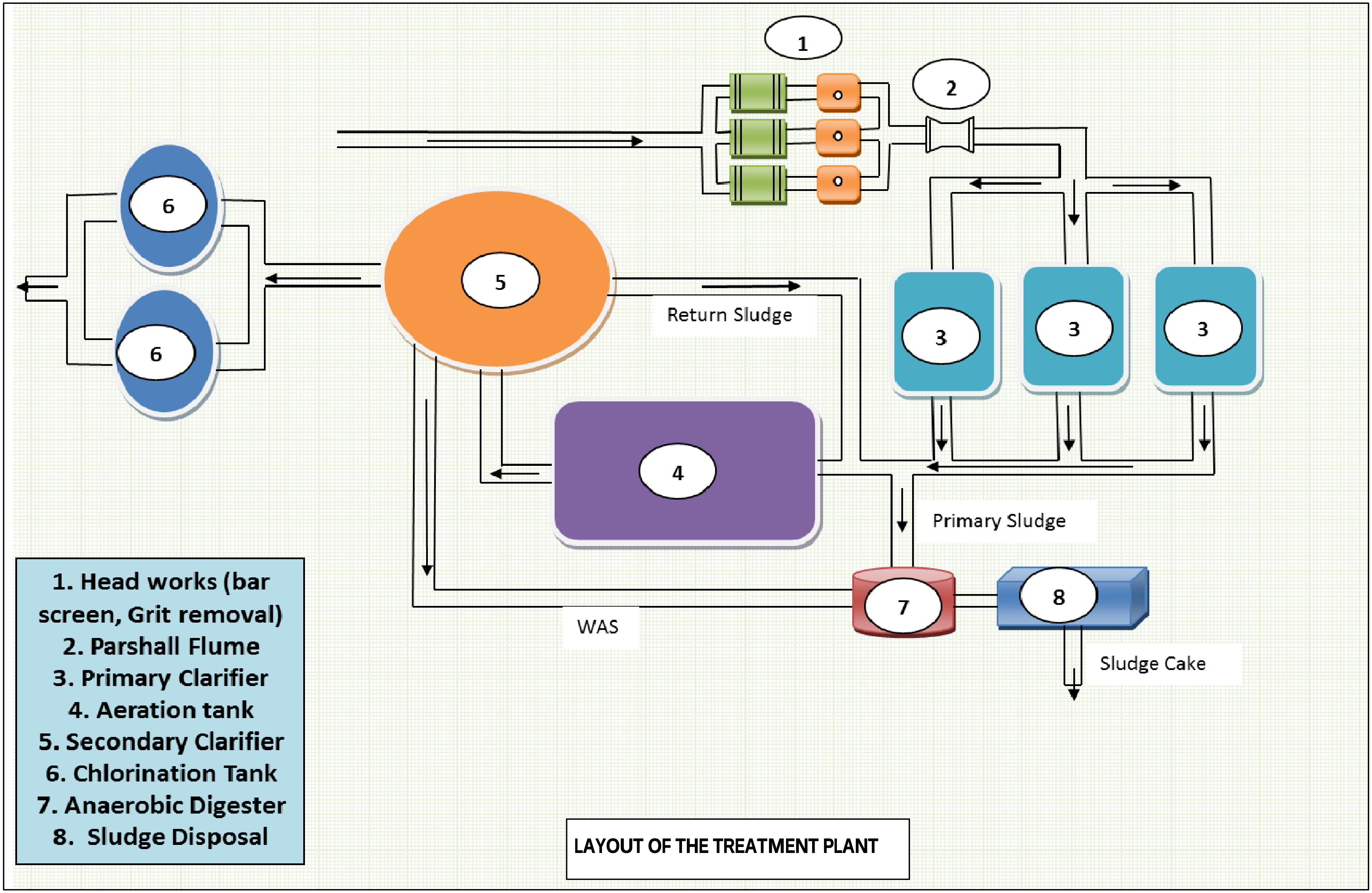
- Dried sludge can be used as fertiliser (either directly in the case of planted beds or after composting in the case of unplanted beds)
- Easy to operate (no experts, but trained community required)
- High reduction of sludge volume
- Can achieve pathogen removal
- Can be built with locally available materials

Disadvantages

- Requires large land area
- Requires treatment of percolate
- Only applicable during dry seasons or needs a roof and contour bund
- Manual labour or specialised equipment is required to remove dried sludge from beds
- Can cause odour problems.

9.2 Related Photographs

Fig. 9.1 Examples of drying beds



LAYOUT OF THE TREATMENT PLANT

The costs include civil construction, equipment supply and installation, auxillary buildings, contractors' overhead and profit. When primary treatment is needed, the cost is close to the upper limit.

Installation Cost		
No	Item	Cost
1	rectangular clarifiers (1000 gpd/ft2; 40,000 ft3),	$2,247330 (Construction cost)
2	mechanical aeration equipment	$1900/hp for 25 hp aerator, $750/hp for 100 hp aerator
3	diffused aeration systems	blower $250-$550 / hp for 500-1000 hp aerator
4	mixers in anoxic or anaerobic zones	$2300 / hp for 5 hp mixer to $1000 / hp for 40 hp mixer
5	recycle pumping for WAS or RAS	$6250 - $7000 / MGD capacity

Table 10.1 Installation and Maintenance Cost

Running cost of wastewater treatment plant mainly includes wastewater discharge fee, electricity cost, chemical cost, staff cost, maintenance and replacement cost, sludge disposal, administration cost .

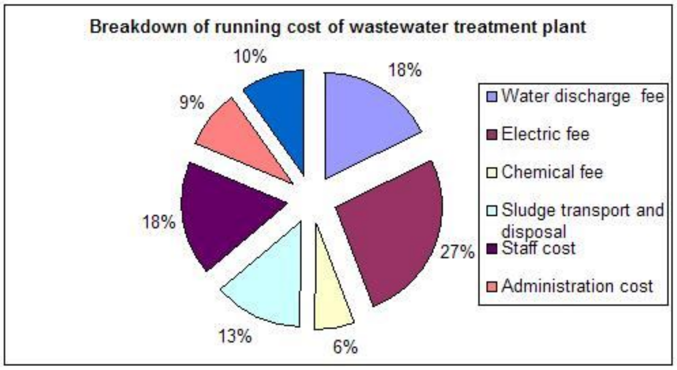


Fig 10.1 Breakdown of running costs of a treatment plant

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