Waste-water treatment plant: Design

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Abstract:

Wastewater treatment is a process which is being done on the wastewater to change it's quality for drinking or other suitable porpuses. Wastewater treatment takes palce in wastewater treatment plants which should be designed under different circumstances. These criterias will be considered in this literature while designign the wastewater treatment plant.

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1. Introduction

Wastewater is the water which has been released to the environment that is defined as a combination of the water plus wastes that have been added to the water from a variety of uses, such as industrial, commercial, residences and there are two sources which release the wastewater into the environment.

First, sewage/community wastewater is the kind which has been expelled from domestic premises such as institutions, residence etc. and commercial establishments which are organic because of the consistency of carbon composites alike vegetables, human waste, paper etc. Second, is the wastewater that has been produced by industrial procedures which is also organic in composition (*Zhou, H. 2002).* These pollutions can be dangerous for human body and environment so wastewater should be treated in order to prevent these damages to take place, the process which purifies the wastewater in order to discharge it back into a watercourse is known as wastewater treatment. Wastewater treatment uses chemical, physical, and biological processes to cleanse wastewater in order to protect the environment and public health.

Wastewater treatment happens in some infra structures which are called wastewater treatment plant (*Hammer, 1986*). Generally a wastewater treatment plant consists of Mechanical treatment, Biological treatment and Sludge treatment sections. There are different kinds of pollutants and wastes in the wastewater such as, nutrients, inorganic salts, pathogens, coarse solids etc., which are very dangerous for ecology and human. In order to remove these pollutants different processes have been exposed. There are specific processes and unit operations in wastewater treatment which are chemical, physical or biological. All these processes should be considered before deigning a proper wastewater treatment plant which depends on the characteristics of the wastewater. In this text a wastewater treatment plant will be designed related to the characteristics of the wastewater.

2. Technical Report

2.1 Question:

A wastewater treatment plant is to be designed for a wastewater with the following information. N is the average of last digits of two team members. Design questions are inserted in between the information for easiness. Assume other values as per your educated guess with the help of the text book or other literature, but don't forget to state where that information came from.

1. Some bigger size impurities such as bottles and hair are present.
2. Silt particles of 0.0170*(1+N*0.1) cm diameter and 3*(1-N*0.1) g/cm^3 are present. Design a grit chamber. What are the advantages of an aerated grit chamber?
3. Obtain inflow rate and BOD_5 results in page 529 for question 6-7 by multiplying the flow rate by (1+N*0.1) and B OD_5 by (1+N*0.05)
4. Design an equalization basin.

Time	Flow (m3/s)	BOD5 (mg/L)
0:00	0.0875	110
1:00	0.07	81
2:00	0.0525	53
3:00	0.0414	35
4:00	0.0334	32
5:00	0.0318	42
6:00	0.0382	66
7:00	0.0653	92
8:00	0.113	125
9:00	0.131	140
10:00	0.135	150
11:00	0.137	155
12:00	0.135	160
13:00	0.129	150
14:00	0.123	140
15:00	0.111	135
16:00	0.103	130
17:00	0.104	120
18:00	0.105	125
19:00	0.116	150
20:00	0.127	200
21:00	0.128	215
22:00	0.121	170
23:00	0.11	130

5. 60% removal efficiency of suspended solids should be achieved in primary sedimentation tank. Use Figure P6.12 on page 530 of the book for settlement characteristics. Design the primary sedimentation tank: calculate the necessary overflow rate and thus estimate the surface area required then calculate the dimensions. Design the overflow weir using assumed (from the book) weir loading rates.
6. Microbes growing in the aeration tank are assumed to have the characteristics. K_s , 40*(1+0.1*N) mg BOD_5/L; k_d = 0.1*(1+0.1*N) d^{-1}; μ_m = 2*(1+0.1*N) d^{-1}; Y=0.4*(1+0.05*N) mg VSS/mg BOD_5.
7. 35+20*(1+0.02*N)% of the BOD5 in the wastewater treatment plant effluent can be assumed to be in ss form.
8. In the activated sludge about 40% of the mixed liquor suspended solids is non volatile.
9. Calculate the mean cell residence time
10. If MLVSS in the aeration tank is expected to be 2000*(1+N*0.05) mg/L, calculate the volume of aeration tank, hydraulic retention time, F/M ratio (check if is feasible), return sludge flow rate, sludge production and oxygen requirement.
11. From your own reading, design the secondary settling tank.
12. Form your own reading of the book, what are possible means of treating the waste sludge?
13. What are the advantages and pitfalls of biological phosphorus removal process?

2.2 Designing

2.2.1 Grit chamber:

N = 5

Silt particles diameter = 0.017*(1+N*0.1) = 0.0255 cm

ρ_s = 3*(1-N*0.1) = 1500 kg/m^3

Note that the product of the specific gravity of the particle (1.5) and the density of water is the density of the particles (ρ_s)

Before we can calculate the terminal settling velocity of the particle, we should gather some information from Table A-1 in Appendix A from book (introduction to **environmental engineering** fourth edition) at wastewater temperature of 22^{o}C, we find the water density to be 997.774 kg/m^3. We will use 1000 kg/m^3 as a sufficiently close approximation. Form the same table, we find the viscosity to be 0.995 mPa. Silt particles diameter is 0.000255 m.

$$Vs = \frac{g(\rho s - \rho)d2}{18\mu}$$

$$Vs = \frac{9.8\,(1500-1000)0.000255}{18(0.000995)} = 0.0177 \text{ m/s} = \mathbf{17.7\ mm/s}$$

The Reynolds number for this settling velocity (0.0177 m/s) and particle size is =

$$Re = (0.0177)*(0.000255)/(0.000995/1000) = \mathbf{4.54}$$

From the iterative solution Vo = 0.028 m/s. with a flow of 0.15 m^3/s and a horizontal velocity of 0.25 m/s the cross sectional area of flow may be estimated to be.

$$Ac = (0.15 \text{ m}^3/\text{s})/(0.25\text{m/s}) = \mathbf{0.60\ m^2}$$

The depth of flow is then estimated by diving the cross-sectional area by the width of the channel.

$$h = 0.60 \text{ m}^2/0.56\text{m} = \mathbf{1.07m}$$

If the grit particle in question enters the grit chamber at the liquid surface, it will take h/Vs seconds to reach the bottom.

$$t = 1.07\text{m}/0.028\text{m/s} = \mathbf{38.2\ s}$$

Since the chamber is 13.5 m in length and the horizontal velocity is 0.25 m/s the liquid remains in the chamber.

$$t = 13.5\text{m}/0.025\text{m/s} = \mathbf{54s}$$

thus, the particle will be captured in the grit chamber.

Overflow velocity = 0.15/(13.5*0.56)=**19.8 mm/s**

Vs/Vo = 17.7 / 19.8 = 0.893 < 1 hence some sand particles with this diameter and density would settle down.

The assumption is that the horizontal grit chamber that is 13.5 m in length if the average grit chamber flow is 0.15 m3/s, the width of the chamber is 0.56 m and the horizontal velocity is 0.25 m/s.

Advantages:

Girt removal from wastewater happens through Aerated Grit Chamber removes grit. Grit chamber has different advantages (*Umesh, 2011*);

1. Possible septic conditions of the plant influent may be alleviated through pre-preparation in the grit chamber.

2. Consistent removal efficiency over a wide flow range.

3. Performance of downstream units may be improved by using pre-aeration to reduce septic conditions in incoming wastewater.

4. Aerated grit chambers are versatile, allowing for chemical addition, mixing, pre-aeration and flocculation.

5. Maintenance is significantly reduced.

6. The mechanical design is very simple.

7. Tin this process there is no moving parts under the water surface.

8. Blower air can be used in this method to air-lift pumping.

9. A relatively low decayable organic content may be removed with a well-controlled rate of aeration. (*Umesh, 2011*)

2.2.2 Equalization basin

Flow equalization is not a treatment process in itself, but a technique that can be used to improve the effectiveness of both secondary and advanced wastewater treatment processes.

The first step then is to calculate the average flow. In this case it is 0.1404 m^3/s. Next the flows are arranged in order beginning with the time and flow that first exceeds the average. In this case it is at 0900h with a flow of 0.1965m^3/s. the tabular arrangement is show on table below.

Explanation of the calculations for each column follows.

$$\text{Volume inflow} = (0.1965\ m^3/s)(1h)(3600\ s/h) = 707.4\ m^3$$

Same step for all value.

$$\text{Volume outflow} = (0.1404\ m^3/s)\ (1h)\ (3600s/h)= 505.44\ m^3$$

Difference between the inflow volume and outflow volume.

$$dS = Vin – Vout$$

The cumulative sum of the difference between the inflow volume and outflow volume.

$$\text{Storage} = \sum dS.$$

The required volume for the equalization basin is the maximum cumulative storage. With the requirement for 25 percent excess, the volume would then be.

Storage volume = 2219.4m^3*1.25= 27774.25 m^3.

The average concentration is determine as.

$$Savg= \frac{(Vi)(So)+(Vs)(Sprev)}{Vi+Vs}$$

Where. Vi =volume of inflow during time interval Δt, m^3.

So = average BOD5 concentration during time interval Δt, g/m^3

Vs = volume of wastewater in the basin at the end of the previous time interval m^3.

Sprev = concentration of BOD5 in the basin at the end of the previous time interval g/m^3.

Time	Flow	BOD5	Inflow	BOD5
0:00	0.0875	110	0.13125	137.5
1:00	0.07	81	0.105	101.25
2:00	0.0525	53	0.07875	66.25
3:00	0.0414	35	0.0621	43.75
4:00	0.0334	32	0.0501	40
5:00	0.0318	42	0.0477	52.5
6:00	0.0382	66	0.0573	82.5
7:00	0.0653	92	0.09795	115
8:00	0.0113	125	0.01695	156.25
9:00	0.131	140	0.1965	175
10:00	0.135	150	0.2025	187.5
11:00	0.137	155	0.2055	193.75
12:00	0.135	160	0.2025	200
13:00	0.129	150	0.1935	187.5
14:00	0.123	140	0.1845	175
15:00	0.111	135	0.1665	168.75
16:00	0.103	130	0.1545	162.5
17:00	0.104	120	0.156	150
18:00	0.105	125	0.1575	156.25
19:00	0.116	150	0.174	187.5
20:00	0.127	200	0.1905	250
21:00	0.128	215	0.192	268.75
22:00	0.121	170	0.1815	212.5
23:00	0.11	130	0.165	162.5
			0.1404	151.3542
			Inflow Ave	**BOD5 Ave**

Table 1. *Shows the primary data*

Time	inflow	outflow	BOD5	dS	Storage	BOD5, ET	Volume of Inflow	Volume outflow
9:00	0.1965	0.1404	175	201.96	201.96	175.00	707.4	505.44
10:00	0.2025	0.1404	187.5	223.56	425.52	184.79	729	505.44
11:00	0.2055	0.1404	193.75	234.36	659.88	190.48	739.8	505.44
12:00	0.2025	0.1404	200	223.56	883.44	195.48	729	505.44
13:00	0.1935	0.1404	187.5	191.16	1074.6	191.96	696.6	505.44
14:00	0.1845	0.1404	175	158.76	1233.36	185.48	664.2	505.44
15:00	0.1665	0.1404	168.75	93.96	1327.32	180.01	599.4	505.44
16:00	0.1545	0.1404	162.5	50.76	1378.08	174.84	556.2	505.44
17:00	0.156	0.1404	150	56.16	1434.24	167.65	561.6	505.44
18:00	0.1575	0.1404	156.25	61.56	1495.8	164.42	567	505.44
19:00	0.174	0.1404	187.5	120.96	1616.76	171.23	626.4	505.44
20:00	0.1905	0.1404	250	180.36	1797.12	194.69	685.8	505.44
21:00	0.192	0.1404	268.75	185.76	1982.88	215.26	691.2	505.44
22:00	0.1815	0.1404	212.5	147.96	2130.84	214.58	653.4	505.44
23:00	0.165	0.1404	162.5	88.56	2219.4	203.23	594	505.44
0:00	0.13125	0.1404	137.5	-32.94	2186.46	191.69	472.5	505.44
1:00	0.105	0.1404	101.25	-127.44	2059.02	178.36	378	505.44
2:00	0.07875	0.1404	66.25	-221.94	1837.08	164.79	283.5	505.44
3:00	0.0621	0.1404	43.75	-281.88	1555.2	151.66	223.56	505.44
4:00	0.0501	0.1404	40	-325.08	1230.12	140.06	180.36	505.44
5:00	0.0477	0.1404	52.5	-333.72	896.4	129.33	171.72	505.44
6:00	0.0573	0.1404	82.5	-299.16	597.24	120.57	206.28	505.44
7:00	0.09795	0.1404	115	-152.82	444.42	118.50	352.62	505.44
8:00	0.01695	0.1404	156.25	-444.42	0.00	123.06	61.02	505.44
					2219.4	171.96		
					Storage Ave	BOD5,ET Ave		

Table 2. *Shows the results*

2.2.3 Primary sedimentation

Evaluate the following primary tank design with respect to detention time, overflow rate, and weir loading.

Design data.

Flow = 0.1965 m^3/s.

Influent ss = 286 mg/l.

Sludge concentration = 6 %

Efficiency = 60%

Length = 40 m (effective)

Width = 10 m liquid depth = 2 m

Weir length = 75m

The detention time is simply the volume of the tank divided by the flow:

$$\theta = V/Q = (40*10*2)/\ 0.1965 = 4071.24 \text{ s} = 1.13 \text{ h}$$

this is a reasonable detention time.

The overflow rate is the floe divided by the surface area.

Vo = 0.1965 / (40*10) = 0.000491 m/s x 86400 s/d = 42.4 m/d

This is an acceptable overflow rate.

The weir loading is calculated in the same fashion:

WL = 0.1965/75 = 0.00262 m^3/s. m x 86400 s/d = 226.368 m^3/d. m

This is an acceptable weir loading.

Ks = half velocity constant, mg/l = 60 mg BOD5 /L.

Kd = decay rate of microorganisms, d^{-1}= 0.15 d^{-1}.

μm = maximum growth rate constant, d^{-1}= 3 d^{-1}.

Y = decimal fraction of food mass converted to biomass, mg VSS/mg BOD5. = 0.5 mg VSS/mg BOD5.

BOD5 = 84 mg/L assumed from the book example 6-5.

Flow = 0.150 m^3/s assumed from the book example 6-5.

57% of the BOD5 in the wastewater treatment plant effluent can be assumed to be in SS form = (84*57)/ 100 = 47.88 mg/l = 48 mg/L

In the activated sludge about 40% of the mixed liquor suspended solids is non-volatile.

S = BOD5 allowed – BOD5 in suspended solids

S= 48 – (0.4)(48) = 29 mg/L

The mean cell-residence time can be estimated with equation: S= $\frac{Ks(1+KdOc)}{Oc(\mu m-Kd)-1}$ and the assumed values for the growth constants. 29 = $\frac{60(1+0.15Oc)}{Oc(3-0.15)-1}$

Solving for θc.

(29)(2.85θc – 1) = 60 + 9θc

82.65 θc – 29 = 60 + 9 θc

θc = $\frac{89}{73.65}$ = 1.2 d

if we assume a value of 2000*(1+N*0.05) mg/L for the MLVSS, we can solve equation X = $\frac{\theta c(Y)(So-S)}{\theta(1+Kd\theta c)}$ for the hydraulic detention time.

2500 = $\frac{1.2(0.5)(84-29)}{O(1+0.15*1.2)}$

θ = $\frac{33}{2950}$ = 0.01 d or 0.26 h

The volume of the aeration tank is then estimated using equation: $\frac{V}{Q}$ = θ

0.26 = $\frac{V}{0.150\ X\ 3600}$

V = 140.4 m^3 or 140 m^3

2.2.4 F/M ratio

By using the data from the equation before we can calculate the F/M.

F/M = $\frac{QSo}{VX}$

F/M = $\frac{(0.150)(84)(86400)}{(140)(2500)}$ = 3.11 mg/mg.d

This is not well within the typical range of F/M ratios.

2.2.5 Return sludge flow rate

Qr = $\frac{QX' - QwXr'}{Xr' - X'}$

Where . Q = wastewater flow rate m^3/d.

Qr = return sludge flow rate m^3/d.

X' = mixed liquor suspended solids (MLSS), g/m^3.

Xr' = maximum return sludge concentration, g/m^3.

Q_w = sludge wasting flow rate m^3/d.

Calculate the return sludge concentration.

Xr' = 10^6 / SVI

Assumption SVI to be 175

Xr' = 5714.29 mg/L

Xr = Xr'/1.43 = 3986 mg/L

Calculate the Q_w .

$Q_w = \frac{VX}{OcXr} = \frac{140*2500}{1.2*3986}$ = 73.17 m^3/d.

Converting Q_w to m^3/s.

Q_w = 0.00085 m^3/s.

Noting that 1 mg/L = 1 g/ m^3. If we ignor the effluent suspended solids the estimated return sludge flow rate is

Qr = $\frac{(0.150)(2500)-(0.00085)(5714)}{5714-2500}$ = 0.115 m^3/s.

We can check this result by using the figure of design MLSS versus SVI and return sludge ratio. (Source: WEF, 1992)

Sludge production.

The net activated sludge produced each day is determined by.

Yobs = $\frac{Y}{1+KdOc}$ = $\frac{0.5}{1+(0.15*1.2)}$ = 0.49 kg VSS/Kg BOD5 removed .

The net waste activated sludge produced each day is.

Px = YobsQ(So – S)(10^{-3}kg/g)

Px = (0.49)(0.150)(84 – 29)(86400)(10^{-3}) = 349.272 kg/d of VSS.

The total mass produced includes inert material. Using the relationship between MLSS and MLVSS.

Increase in MLSS = 1.25(349.272) = 436.59 kg/d

The oxygen demand of the waste activated sludge may be estimated as 1.42(Px).

The mass of oxygen required may be estimated as:

$Mo_2 = \frac{Q(So-S)(\frac{0.001Kg}{g})}{f} - 1.42(Px)$

Where. f = conversion factor for converting BOD5 to ultimate BOD_L and it assumed to be 68 percent.

$Mo_2 = \frac{(0.150)(84-29)(86400)(0.001)}{0.68}$ – 1.42(349.272) = 552.57 kg/d of oxygen.

2.2.6 Secondary settling tank

Utilizing an average overflow rate of 42.4 m/d, we determine the diameter of the secondary tank as follows: first, compute the surface area required:

As = $\frac{0.150\ X\ 86400}{42.4}$ = 305.66 m^2

Only one-half of the hydraulic load is used to compute the surface area.

The diameter of the tank is then

305.66 = $\pi D^2/4$

D = 19.7 m or 20 m

From the table of final settling basin side water depth we select an SWD of 3.7 m

Now we must check the solids loading. Using the equality 1mg/L = 1g/m^3

SL = $\frac{2500 \, X \, 0.3}{\pi(20)2/4}$ = 750/314 *10^{-3}*86400 = 206.36 kg/d .m^2.

Checking this rate with the maxima shown in figure of design solids loading versus SVI. We find that for an SVI of 175 from assumption, we have the maximum allowable loading of 200 kg/d .m^2.

The weir loading for a single weir located at the periphery is.

WL = $\frac{0.15 \, X \, 86400}{\pi(20)}$ = 206.36 m^3/d.m.

This is less than the prescribed loading given by GLUMRB and, therefore, is acceptable.

2.2.7 Waste sludge

Theory:

Bio-solids or Sludge is the residue which stores in sewage treatment plants. Sludge treatment means reducing the volume of the sludge, stabilizing the organic materials and ultimately disposing the sludge. Sludge treatment can be done by the combination of thickening, stabilisation, drying, incineration, digestion, and dewatering processes. Parameters in relation to the disposal/use operations of the sludge are:

1. Agricultural Use
2. Composting
3. Incineration
4. Landfilling (*E. E. A.)*

2.2.8 Biological phosphorus removal

High amount of phosphorus can be harmful for human body so phosphorus removal is an essential effort in wastewater treatment process which has it's own advantages and disadvantages:

Advantages:

1. "There is no deterioration of dewater ability
2. There is lower salinity in the effluent
3. There is no chemical sludge production
4. There is decreased inhabitation of nitrification process
5. There are fewer negative consequences for total N removal
6. Sludge is of a better quality" (*Janssen et al,. 2002*).

Pitfalls:

1. "Dependence on wastewater composition
2. Lower stability and flexibility
3. Influence on the sludge volume index
4. Phosphorus release in the sludge treatment" *(Janssen et al,. 2002)* .

3. Conclusion

This project was undertaken to design a wastewater treatment plant with some particular data. The grit chamber, equalisation basin, primary sedimentation tank and secondary settling tank have been designed, then the values for mean cell residence time, volume of aeration tank, hydraulic retention time, f/m ratio, return sludge flow rate, sludge production and oxygen requirement have been calculated, ultimately the theoretical aspects of grit chamber, waste sludge and biological phosphorus removal have been covered. Some assumptions have been made during designing the plant, the recommendation is to reduce these assumptions as many as possible to achieve the more accurate and reliable results. Some designing calculations have been done by using Excel software so it is advised to use such softwares for the ease of calculation. In addition, this designing process is suitable for this particular situation and it cannot be followed for every situations. Designing a wastewater treatment plant depends on the characteristics of the wastewater so the designing process should be analysed carefully because even a small mistake can be fatal.

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