



FACULTY OF BIOSCIENCE ENGINEERING

Waste Water Treatment Plant Case Study

Biotechnological Processes

Faculty of Bioscience Engineering

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I. INTRODUCTION

Until 1990, only around 30 % of sewage water was treated out of 70 % collected wastewater in Belgium. After the European Communion gave the deadline to treat more sewage water, then Flemish Government decided to establish [redacted] to handle the issue, starting from collecting the wastewater until treating it. [redacted] is responsible to collect and treat wastewater from households/domestic source, and other several small industries that cannot build their own wastewater treatment plant (WWTP). At the moment, there are 250 WWTP operated by [redacted] throughout Flanders ranging from small to big sizes. The WWTP in [redacted] is the biggest. The idea is to have desentralized WWTP in as many places as possible, and thus the size of WWTP will be small. The WWTP in [redacted] handles waste water from 270,000 inhabitants. From the beginning of construction, the WWTP was designed to be able to treat rainwater. When it is raining, the water flow will be 10 times than the dry weather flow (DWF). Dry weather flow is characterized by a water flow without rainwater. The waste water treatment plant is designed to accomodate the water flow which is 6 times the dry weather flow. Water flow which is more than 6 times of the dry weather flow is discharged to the river.

When the operation of the WWTP was started, the established standard was only for BOD and COD, but along with the development of sanitation knowledge, the standard is improved to also include other parameters such as nutrients concentration level. The company operates all the water treatment cycle except for the treatment of the sludge which is treated through incineration. Other company operates the incineration plant.

During the visit it is aimed to describe and assess all the processes that take place during waste water treatment and the calculation of all the parameters necessary for the design of a WWTP.

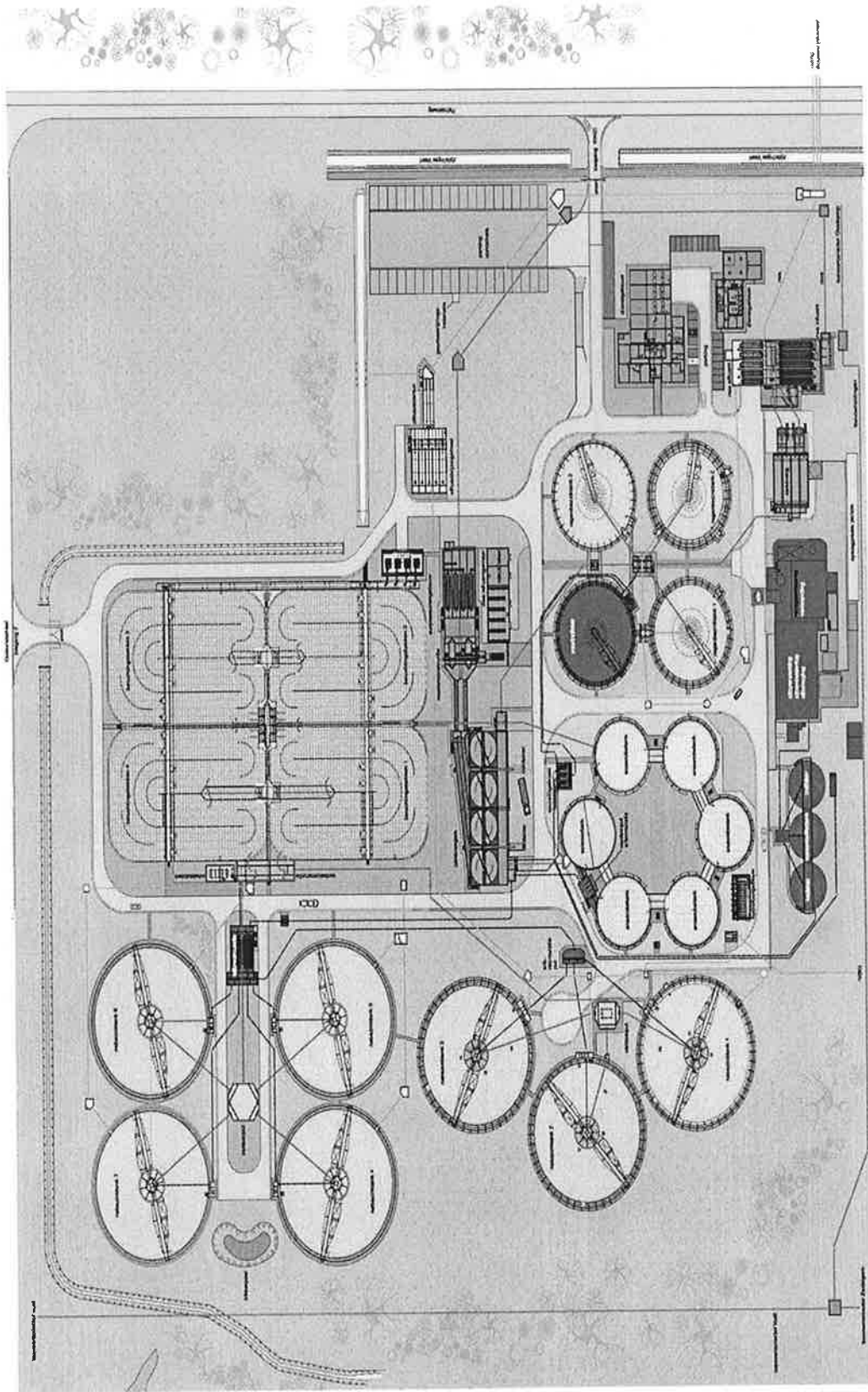


Figure 1. General scheme of the WWTP in

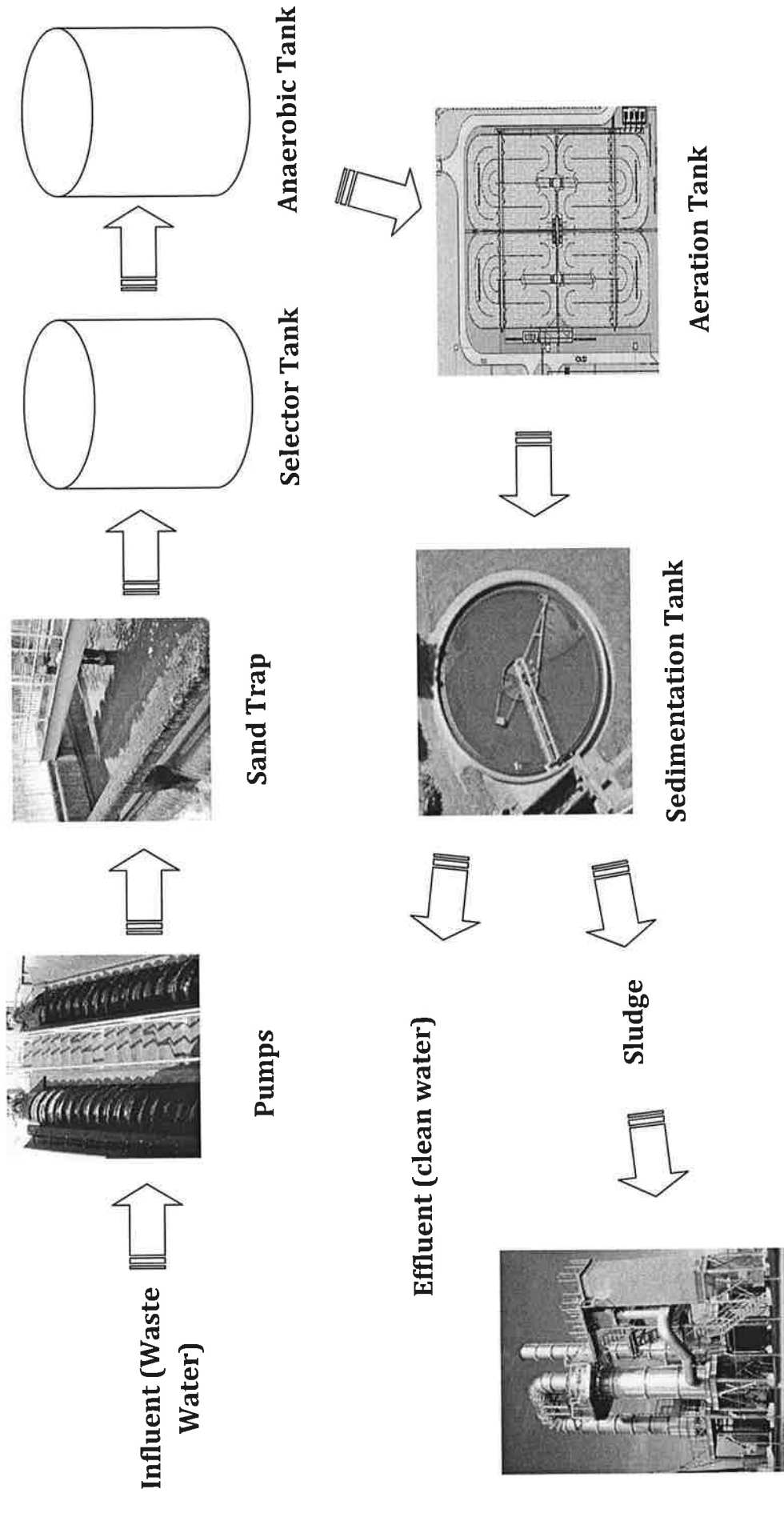


Figure 2. Waste water scheme for dry weather flow (DWF).

II. GENERAL SCHEME OF THE WWTP

The general scheme of the WWTP can be seen in Figure 1. The whole process for the wastewater treatment is explained with the red and blue lines. The red lines indicates flow of the influent while the blue lines indicates flow of the effluent. Figure 2 shows the treatment process for dry weather condition, when the amount of wastewater to be treated is not including the rain water.

A. Collectors and screw system

Water comes from the sewage network is brought to big collectors underground for treatment process. From collectors it is pumped into the sand trap by using screw systems which works based on the principle of gravity. The flow of waste water starting from the pump screws to the sand traps until the sedimentation tanks works by gravity and thus minimizes energy consumption. The highest point in the treatment plant is therefore the pump screws. Each screw works at a volumetric rate of 982 L/s. Above the pump screws, screens are found to trap large waste water debris and thus preventing them from entering into the sand traps. This whole process can be seen on Figure 3 to 5.

In cases of heavy rain, water passes through 3 buffer tanks and subsequently through sand and fat traps before it is discharge directly into the river. Whenever possible, the water from the buffer tanks is added to the influent to be treated in the WWTP.

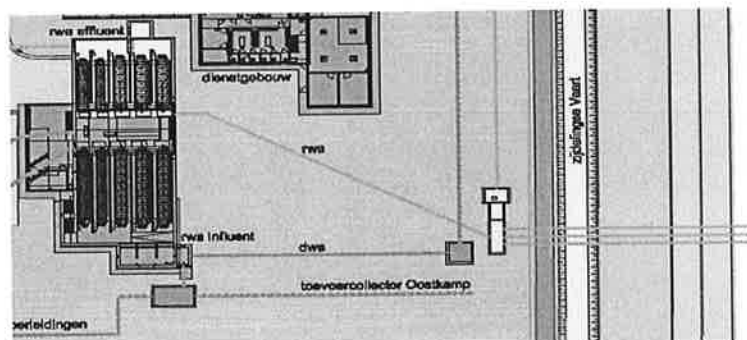


Figure 3. Scheme of water entering WWTP.

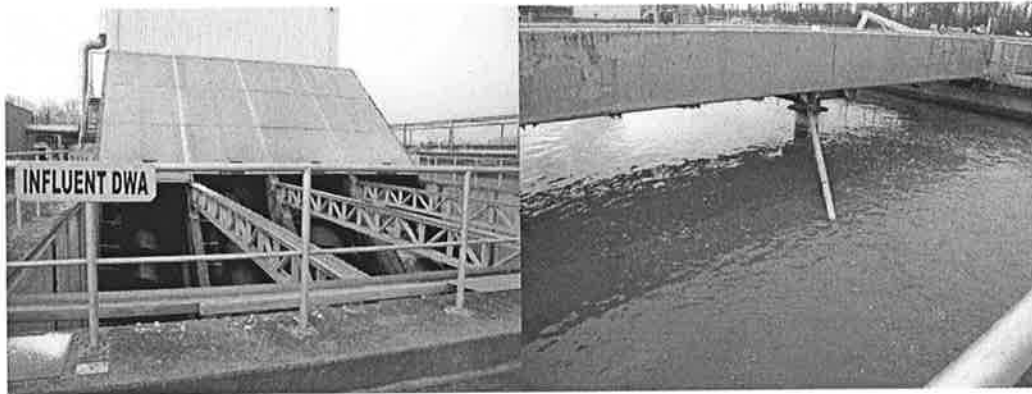


Figure 4. Domestic wastewater entering the plant and is pumped up into the sand traps.

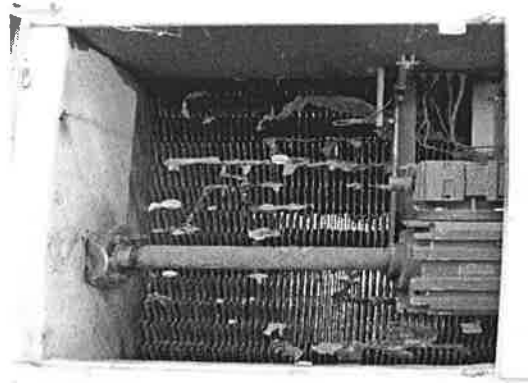


Figure 5. Screens above the screw pumps.

B. Sand traps

Water passes at low velocity through sand traps where big particles and sands are removed. The sand trap is installed before the selector tanks. After sand is removed, the collected sand is transported to a collector truck, as can be seen from Figure 6.

The sand trap is Dorr type. Retention time in sand traps is approximately 8.5 minutes.

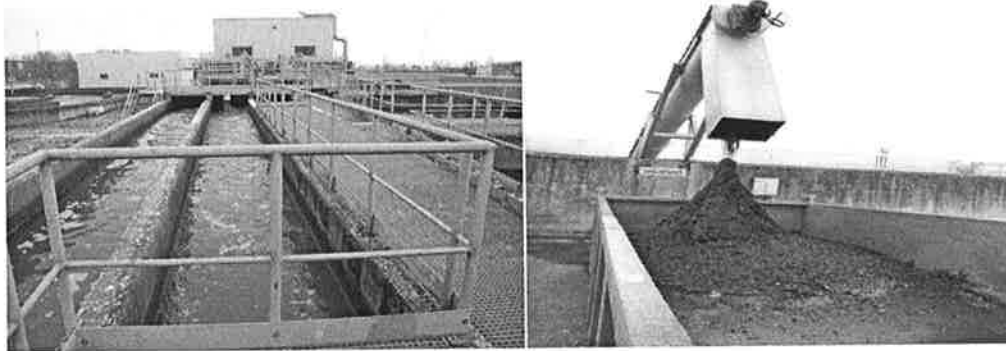


Figure 6. Sand treatment.

C. Selector tank and anaerobic tank

The installation consists of 4 selector tanks with a volume of 2.350 m³ each. In the selector tank, the sludge is allowed to feed at high substrate concentrations. This allows the sludge to improve the formation of floc-forming microorganisms in order to assure good sedimentation process. Water in selector tanks has high content of BOD and COD which provides the necessary conditions to develop the floc-forming microorganisms. The selector tank also provides the P-accumulating microorganisms an opportunity to store the readily biodegradable COD (RbCOD).

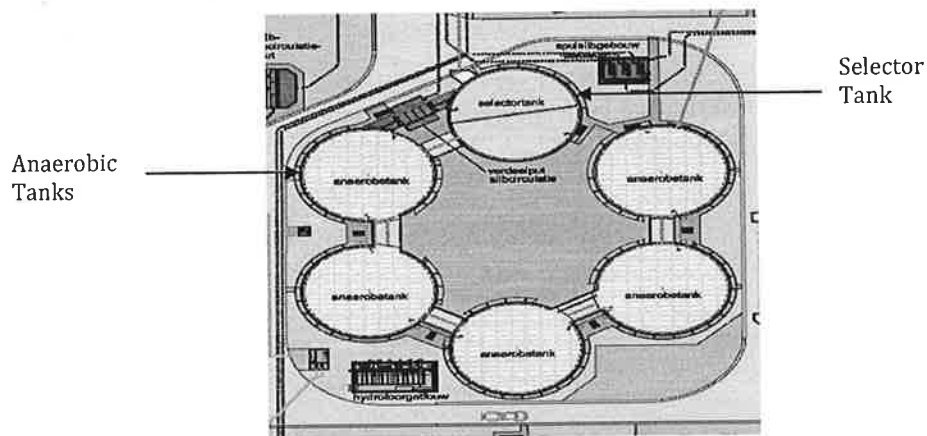


Figure 7. Scheme of selector and anaerobic tanks in the WWTP.



Figure 8. Selector tank.

In the anaerobic tanks, the so-called phosphate accumulating organisms (PAOs) take up fatty acids and store them as polyhydroxybutyrate (PHB). This process will at the same time hydrolyse internal poly-P reserves, thus releasing phosphorus. Figure 7 shows the scheme for selector tanks and anaerobic tank, Figure 8 shows the selector tanks, while Figure 9 shows the anaerobic tank.



Figure 9. Anaerobic tank.

D. Aeration

The installation consists of 4 aeration tanks with a total volume of 70.000 m³. On dry weather flow, however, usually only 2 aeration tanks are used. Aeration is provided in order to remove COD and BOD. In the aeration tanks two different processes take place i.e nitrification and denitrification.

During nitrification, ammonia (NH₄⁺) is oxidized to nitrate (NO₃⁻) and later in the anaerobic phase the nitrate is reduced to dinitrogen (N₂). The nitrification-denitrification processes take place in a so-called carousel configuration in which there are anoxic and oxic zones. Air is provided by air diffusers placed at the bottom of the aeration tanks. The whole process can be seen on Figure 10, while Figure 11 shows the aerations tanks.

During aeration, Phosphate Accumulating Organisms (PAO's) take up the phosphates released from the anaerobic digestion as well as the phosphates released during degradation of the substrates.

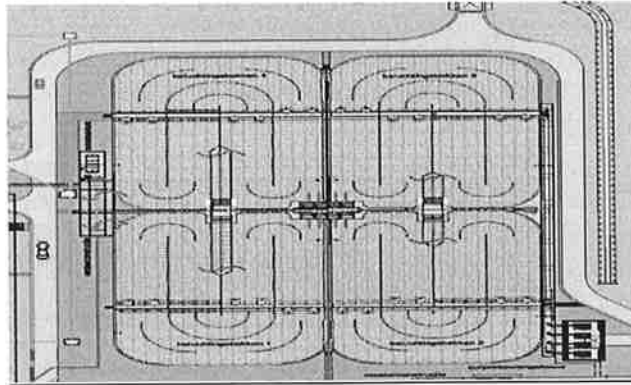


Figure 10. Scheme of aeration tanks.

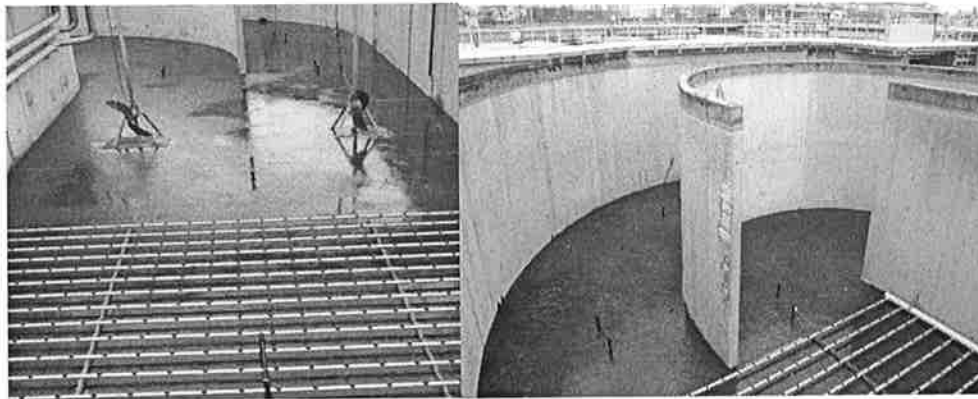


Figure 11. Aeration tanks.

The aeration uses fine bubbles air diffusers of plastic type. Clogging of the air diffusers can occur due to the attached growth of biomass or sedimentation of solids or to chemical precipitation. Clogging can be noticed because there will be change of energy when it happens. When the tank is clogged, it is emptied and replaced or fixed. This replacement or fixing of the air diffusers cause additional expenses to the maintenance cost.

E. FeCl₃ storage tank

When the phosphorus removal in the aeration tank is not sufficient enough, phosphorus is removed through the chemical process. Chemical phosphorus removal is performed by using iron chloride (FeCl₃), which is stored in the tank. The installation removes phosphorus chemically by precipitation of phosphorus with FeCl₃. This treatment is used when the biotical removal achieved is not enough to comply with the standards in legislation. In this way, sensors for phosphorus detection are installed in order to detect when the concentrations after biological removal still

exceed the limits allowed. If so, chemical treatment is used. Figure 12 shows the FeCl_3 storage tank for extra phosphate removal.

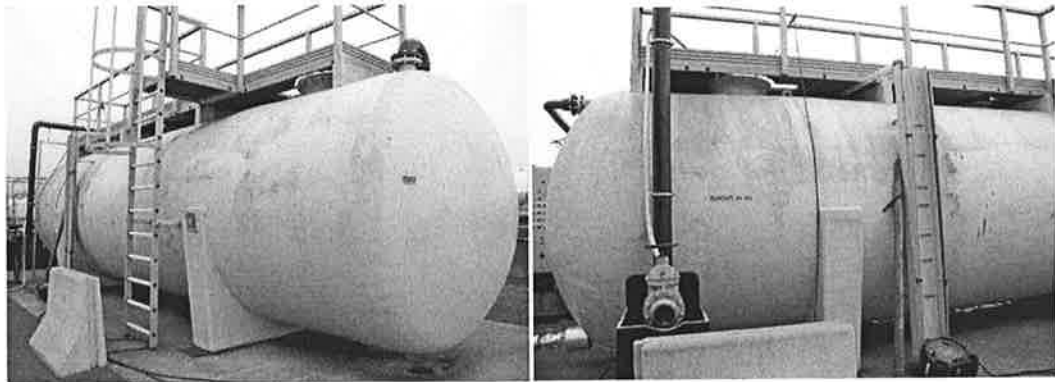


Figure 12. FeCl_3 storage tank for extra phosphate removal.

F. Sedimentation tanks and sludge thickener tank

After water passes through the phosphate removal tanks, it is transported to the sedimentation tanks where it is equally distributed into the 4 tanks in the WWTP. Each sedimentation tanks has a diameter of 50 m and a volume of 5000 m³. During sedimentation activated sludge is collected in the center. Part of the sludge is recirculated back into the selector tanks and the remaining is collected for disposal or further treatments. Recirculation is done by using screw system which operates at a rate of 700L/s, as can be seen from the scheme on Figure 13 to 16.

In the WWTP in there is an incineration facility operated by a third party. In this installation sludge is firstly thickened by gravity and subsequently it is centrifuged and dried to remove excess of water. After water is removed sludge goes to incineration, where heat is recovered to dry the sludge. At the incineration process, the dried sludge is converted into ash.

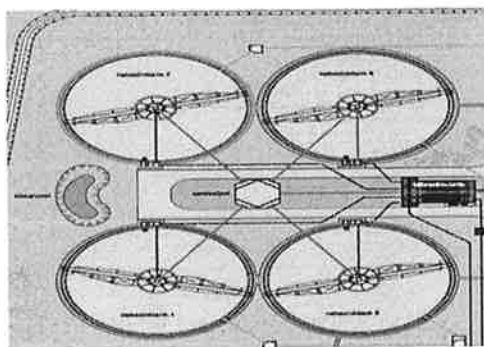


Figure 13. Scheme of sedimentation tanks.



Figure 14. Sedimentation tanks.



Figure 15. Sludge collector and recirculation.

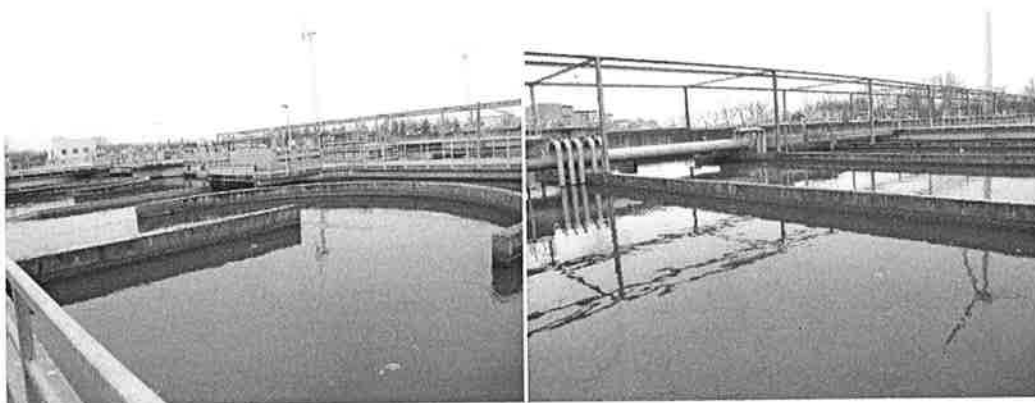


Figure 16. Sludge thickener tank.

G. Discharge of treated water

After all processes have taken place in the WWTP, water passes through quality control, where all parameters established in legislation are measured to ensure that the water complies and is safe for consumers. In events of heavy rain, when not all the water can be treated, water is discharged into the river after removal of fat and sand. Figure 17 shows where the treated water passes the pump to discharge the water into the canal.

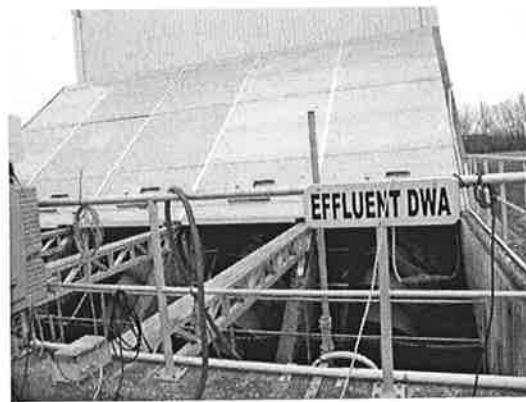


Figure 17. Domestic effluent to discharge into the canal.

III. CHARACTERISTIC OF THE TREATMENT PLANT

A. Wastewater Characteristics

The wastewater characteristics of influent and effluent for the WWTP can be seen on Table 1.

Table 1. Characteristics (mean values) of the wastewater on the influent and effluent.

	BOD (mg/L)	COD (mg/L)	Suspended solids (mg/L)	NH ₄ ⁺ (mg/L)	KJN (mg/L)	NO ₃ ⁻ (mg/L)	NO ₂ ⁻ (mg/L)	Total N (mg/L)	PO ₄ ³⁻ (mg/L)	Total P (mg/L)
influent	76.9	259.5	148.5	15.69	26.82	1.27	0.251	28.34	2.27	5.68
effluent	2	30.9	6.5	0.18	1.5	3.88	0.049	5.43	0.58	0.89
Percentage treated (%)	97.40	88.09	95.62	98.85	94.41			80.84	74.45	84.33

The mean values of the effluent does not exceed the typical criteria for discharge on inland surface waters which are [1]:

COD: 125 mg/L
 BOD₅: 25 mg/L
 SS: 35 mg/L
 Total N: 15 mg/L
 Total P: 2 mg/L

The waste water composition corresponds to low – medium strength as described by Metcalf and Eddy, 2004) [1]. The composition of the wastewater however is highly variable on season, as well as on a specific hour of the day. According to Rabaey et al (2012), for optimal aerobic metabolism, a bCOD/N/P ratio of 100/5/0.5 is necessary [1]. For the mean wastewater characteristic, the bCOD/N/P ratio is 100/12/2. Although, the waste water characteristic is very variable, the mean ratio shows that N and P ratio is a bit higher than the bCOD/N/P ratio ideal optimal aerobic metabolism.

B. Basic Information of the Installation

1. Rainwater treatment tanks

The dimensions of rainwater treatment tanks can be seen on Table 2.

Table 2. Rainwater treatment tanks dimensions

	Water Depth (m)		Diameter (m)	Tank Volume (m ³)
	Min	Max		
Rainwater Treatment 1	1.6	2.6	35	1803
Rainwater Treatment 2	1.6	2.6	35	1746
Rainwater Treatment 3	1.6	2.6	35	1803

2. Domestic sand trap

The dimensions of sand trap tanks can be seen on Table 3.

Table 3. Domestic sand trap tanks dimensions

	Water Depth (m)		Diameter (m)	Tank Volume (m ³)
	Min	Max		
Sand Trap Domestic 1	0.63	0.73	12	102
Sand Trap Domestic 2	0.63	0.73	12	102
Sand Trap Domestic 3	0.63	0.73	12	102
Sand Trap Domestic 4	0.63	0.73	12	102

3. Rainwater sand trap

Table 4 shows the dimensions of rainwater sand trap tanks.

Table 4. Rainwater sand trap tanks dimensions

	Width (m)	Length (m)	Water Depth (m)	Tank Volume (m ³)
Sand Trap Rainwater 1	3.5	18	2.9	187
Sand Trap Rainwater 2	3.5	18	2.9	187
Sand Trap Rainwater 3	3.5	18	2.9	187
Sand Trap Rainwater 4	3.5	18	2.9	187

4. Selector tank

Table 5 shows the dimension of selector tank.

Table 5. Selector tank dimensions

	Water Depth (m)	Diameter (m)	Tank Volume (m ³)
Selector Tank	5.77	22.8	2327

5. Anaerobic tanks

The dimensions of 5 anaerobic tanks can be seen on Table 6.

Table 6. Anaerobic tank dimensions

	Water Depth (m)	Diameter (m)	Tank Volume (m ³)
Anaerobic Tank 1	5.62	22.8	2295
Anaerobic Tank 2	5.62	22.8	2295
Anaerobic Tank 3	5.62	22.8	2295
Anaerobic Tank 4	5.62	22.8	2295
Anaerobic Tank 5	5.62	22.8	2295

6. Aeration tanks

Table 7 shows the dimensions of 4 aeration tanks.

Table 7. Aeration tank dimensions

	Width (m)	Length (m)	Water Depth (m)	Tank Volume (m ³)
Aeration Tank 1	43.8	70	5.93	17075
Aeration Tank 2	43.8	70	5.93	17075
Aeration Tank 3	43.8	70	5.93	17075
Aeration Tank 4	43.8	70	5.93	17075

7. FeCl₃ storage tank (cylindrical)

The dimension of the FeCl₃ storage tank can be seen on Table 8.

Table 8. FeCl₃ storage tank dimensions

	Length (m)	Diameter (m)	Tank Volume (m ³)
Phosphate Removal tank (FeCl ₃)	7.76	2.5	40000

8. Sedimentation tanks

The dimensions of 7 sedimentation tanks can be seen from Table 9.

Table 9. Sedimentation tank dimensions

	Water Depth (m)		Diameter (m)	Tank Volume (m ³)
	Min	Max		
Sedimentation tank 1	2.02	3.77	50	5312
Sedimentation tank 2	2.02	3.77	50	5312
Sedimentation tank 3	2.02	3.77	50	5312
Sedimentation tank 4	2.05	3.86	50	5334
Sedimentation tank 5	2.05	3.86	50	5334
Sedimentation tank 6	2.05	3.86	50	5334
Sedimentation tank 7	2.05	3.86	50	5334

9. Sludge thickening tank

Table 10 shows the sludge thickening tanks dimensions.

Table 10. Sludge thickening tank dimensions

	Water Depth (m)		Diameter (m)	Tank Volume (m ³)
	Min	Max		
Sludge Thickening Tank 1	3.5	4.19	15.6	717.57
Sludge Thickening Tank 2	3.5	4.19	15.6	717.57
Sludge Thickening Tank 3 (Front)	1.6	2.6	35	1956.88

10. Sludge storage tank

Table 11 shows the sludge storage tank dimension.

Table 11. Sludge storage tank dimensions

	Water Depth (m)		Diameter (m)	Tank Volume (m ³)
	Min	Max		
Sludge Storage Tank 1	3.5	4.19	15.6	717

11. The biodegradable fraction (F_b)

The biodegradable fraction is the fraction of the COD that can be biodegraded.

$$COD_{influent} = 259.5 \frac{mg}{L} \quad COD_{effluent} = 30.9 \frac{mg}{L}$$

$$F_b = \frac{COD_{influent} - COD_{effluent}}{COD_{influent}}$$

$$F_b = \frac{259.5 \frac{mg}{L} - 30.9 \frac{mg}{L}}{259.5 \frac{mg}{L}} = 0.88$$

In this waste water, 88% of the COD can be biodegraded.

12. Biodegradable COD (bCOD)

The bCOD is the substrate that is biodegradable.

$$bCOD = COD * F_b$$

$$bCOD = 259.5 \frac{mg}{L} * 0.88 = 228.6 \frac{mg}{L}$$

Conversion of bCOD into kg/day

$$bCOD = \frac{Q \left(\frac{m^3}{day} \right) * COD \left(\frac{mg}{L} \right)}{\frac{1000 \frac{mg}{L}}{\frac{kg}{m^3}}}$$

$$Q_w \text{ (mean water flow)} = 140363 \text{ m}^3/\text{day}$$

$$bCOD = \frac{140363 \frac{m^3}{day} * 228.6 \frac{mg}{L}}{\frac{1000 \frac{mg}{L}}{\frac{kg}{m^3}}} = \frac{32086.98 kg}{day}$$

13. The sludge flow ($Q_{w,s}$)

To compute for the biomass formed, it is assumed that the yield (Y) is equal to 0.4 g biomass formed/g COD metabolised as a rule of thumb and the sludge concentration (X) is 4 kg/m³.

$$Biomass\ formed = Y * bCOD$$

$$Q_{w,s} = bCOD \left(\frac{mg}{L} \right) * \frac{Y \left(\frac{g}{g} \right)}{\frac{1000 \frac{mg}{L}}{\frac{kg}{m^3}}}$$

$$Q_{w,s} = \frac{228.6 \frac{mg}{L} * 0.4 \left(\frac{g}{g} \right)}{\frac{1000 \frac{mg}{L}}{\frac{kg}{m^3}}} = 0.091 kg/m^3$$

$Q_{w,s}$ in kg/day

$$Q_{w,s} = Q_{w,s} \left(\frac{kg}{m^3} \right) * Q_w \left(\frac{m^3}{day} \right)$$

$$Q_{w,s} = \frac{0.091 kg}{m^3} * \frac{140363 m^3}{day} = \frac{12834.79 kg}{day}$$

The amount of biomass (kg) formed per day is quite high. One-third of the operating cost goes to the treatment of the sludge.

$Q_{w,s}$ in m³/day

$$Q_{w,s} = \frac{Q_{w,s} \left(\frac{kg}{day} \right)}{X \left(\frac{kg}{m^3} \right)}$$

$$Q_{w,s} = \frac{\frac{12834.79 \text{ kg}}{day}}{\frac{4 \text{ kg}}{m^3}} = 3208.70 \frac{m^3}{day}$$

14. Volatile suspended solids (VSS)

Volatile suspended solids (VSS) is obtained by filtering the sample wastewater [1]. The filtered solids is dried and weighed to obtain the suspended solids [1]. The suspended solids (SS) is heated at an elevated temperature causing it to be ashed [1]. It is weighed again and the % loss of the mass is the volatile suspended solids [1]. The volatile suspended solids represent the amount of organic matter that is present in particulate form in the water [1]. As a rule of thumb, 65-75% of the suspended solids are organic [1]. Taking 70% of the suspended solids are organic; the mean VSS in the influent and effluent is therefore,

$$VSS = 0.70 * SS$$

$$VSS_{influent} = 0.70 * 148.5 \frac{mg}{L} = 103.95 \frac{mg}{L}$$

$$VSS_{effluent} = 0.70 * 6.5 \frac{mg}{L} = 4.55 \frac{mg}{L}$$

15. Volumetric loading rate (B_v)

Volumetric loading rate is the amount of substrate introduced per m³ reactor per day [1]. In practice B_v is equal to 1.0 kg bCOD/m³.day [1]. In the waste water treatment plant, two aeration tanks were used so the total volume of the tank is (2*17075 m³).

$$B_v = \frac{bCOD * Q_w}{\text{volume of 2 aeration tanks}}$$

$$B_v = \frac{\left(228.6 \frac{mg}{l} * 140363 \frac{m^3}{day} \right)}{\frac{1000 \frac{mg}{L}}{\frac{kg}{m^3}} * (2 * 17075 m^3)} = 0.94 \frac{kg \text{ bCOD}}{m^3 \text{ day}}$$

The computed B_v is near to 1.0 kg bCOD/m³.day.

16. Sludge loading rate (B_x)

Sludge loading rate is the amount of substrate given each day to the activated sludge to metabolize
[1]. In practice $B_x = 0.25$ kg bCOD/kg VSS.day [1]

$$B_x = \frac{B_v}{X}$$

$$B_x = \frac{0.94 \frac{\text{kg bCOD}}{\text{m}^3 \text{day}}}{4 \frac{\text{kg biomass}}{\text{m}^3}} = \frac{0.23 \text{ kg bCOD}}{\text{kg biomass.day}}$$

17. Hydraulic residence time (HRT)

Hydraulic residence time is the time that water resides in the aeration system [1]. There are 4 aeration tanks in the treatment plant but only 2 tanks were used on dry weather flow.

$$HRT = \frac{2 * \text{aeration tank volume}}{Q_w}$$

$$HRT = \frac{2 * 17075 \text{ m}^3}{140363 \frac{\text{m}^3}{\text{day}}} = 0.24 \text{ day}$$

Conversion of day to hours:

$$HRT = 0.24 \text{ day} * \frac{24 \text{ hours}}{\text{day}} = 5.84 \text{ hours}$$

18. Sludge Retention Time (SRT/ θ)

The sludge residence time is the average time of residence of the sludge in the aerated system and is controlled by the rate of withdrawal from the reactor [1].

$$SRT = \frac{2 * \text{aeration tank volume}}{Q_{w,s}}$$

$$SRT = \frac{2 * 17075 \text{ m}^3}{3208.70 \frac{\text{m}^3}{\text{day}}} = 10.64 \text{ day}$$

19. Removal or treatment efficiency (%)

The removal or treatment efficiency is the percent that is removed (COD/N/P/Suspended solids) as a result from the treatment [1]. The mean values of COD, total N and total P were used for the computation for efficiency.

$$\text{Efficiency (\%)} = \frac{S_{\text{influent}} - S_{\text{effluent}}}{S_{\text{influent}}} * 100$$

$$\% E_{\text{COD}} = \frac{259.5 \frac{\text{mg}}{\text{l}} - 30.9 \frac{\text{mg}}{\text{l}}}{259.5 \frac{\text{mg}}{\text{l}}} * 100 = 88.09 \%$$

$$\% E_{\text{Ntot}} = \frac{28.34 \frac{\text{mg}}{\text{l}} - 5.43 \frac{\text{mg}}{\text{l}}}{8.34 \frac{\text{mg}}{\text{l}}} * 100 = 80.84 \%$$

Total N is used in the calculation of the treatment efficiency since:

$$\text{Total Nitrogen} = \text{Total Kjeldahl N} + \text{Total oxidized nitrogen}$$

$$\text{Total Kjeldahl N} = \text{Organic N} + \text{Total ammoniacal nitrogen}$$

$$\text{Total oxidized N} = \text{NO}_3^- + \text{NO}_2^-$$

This is to take into account all forms of nitrogen coming in and out of the treatment plant.

$$\% E_{\text{Ptot}} = \frac{5.68 \frac{\text{mg}}{\text{l}} - 0.89 \frac{\text{mg}}{\text{l}}}{5.68 \frac{\text{mg}}{\text{l}}} * 100 = 84.33 \%$$

$$\% E_{\text{suspended solids}} = \frac{148.5 \frac{\text{mg}}{\text{l}} - 6.5 \frac{\text{mg}}{\text{l}}}{148.5 \frac{\text{mg}}{\text{l}}} * 100 = 95.62 \%$$

20. Specific growth rate (μ)

The specific growth rate equals to^[1]

$$\mu = \frac{1}{X} \frac{dX}{dt} = \frac{1}{X} Y_{\text{COD}}^{\text{max}} \frac{dS}{dt} - b$$

$$\mu = \frac{1}{\theta_x} = Y_{\text{COD}}^{\text{max}} q - b$$

where:

$Y_{\text{COD}}^{\text{max}}$ = maximum cell yield coefficient (maximum g biomass formed/g substrate COD removed)

X = biomass concentration

S = total substrate (g COD/L)

θ_x = sludge residence time

q = specific substrate removal rate

b = biomass decay coefficient (g biomass which dies per g biomass present per unit time)

$$\mu = \frac{1}{10.64} = \frac{0.094}{d}$$

21. Cell Yield Coefficient (Y_{COD})

The amount of biomass formed per kg substrate removed^[1].

$$Y_{\text{COD}} = \frac{Q_{w,s} \left(\frac{\text{kg}}{\text{day}} \right) * \frac{1000 \frac{\text{mg}}{\text{L}}}{\frac{\text{kg}}{\text{m}^3}}}{Q \left(\frac{\text{m}^3}{\text{d}} \right) * \left(S_{\text{influent}} \left(\frac{\text{mg}}{\text{l}} \right) - S_{\text{effluent}} \left(\frac{\text{mg}}{\text{l}} \right) \right)}$$

$$Y_{\text{COD}} = \frac{\frac{12834.79 \text{ kg}}{\text{day}} * \frac{1000 \frac{\text{mg}}{\text{L}}}{\frac{\text{kg}}{\text{m}^3}}}{140363 \frac{\text{m}^3}{\text{d}} * \left(259 \frac{\text{mg}}{\text{l}} - 30.9 \frac{\text{mg}}{\text{l}} \right)} = \frac{0.4 \text{ kg MLVSS}}{\text{kg COD removed}}$$

22. Specific substrate removal rate (q)

$$q = \frac{dS}{X} = \frac{\mu + b}{Y_{COD}^{max}} = (S_{inf} - S_{eff}) * \frac{Q}{(X * V)}$$

$$q = \frac{\left(259 \frac{mg}{l} - 30.9 \frac{mg}{l}\right) * 140363 \frac{m^3}{d}}{\left(\frac{1000 \frac{mg}{L}}{\frac{kg}{m^3}} * \frac{4 kg}{m^3} * 2 * 17075 m^3\right)} = \frac{0.23 kg COD removed}{kg MLVSS \cdot day}$$

23. Biomass decay coefficient (b)

The biomass decay coefficient is the amount (g) of biomass that dies away per g biomass present per unit time [1]. The typical values of Y_{COD}^{max} for domestic sewage is in the range of 0.4 – 0.6. It is assumed that the Y_{COD}^{max} is 0.5 maximum g biomass formed/g substrate COD removed for this waste water treatment plant.

$$b = Y_{COD}^{max} * q - \mu$$

$$b = \left(\frac{0.5 kg biomass}{kg substrate} * \frac{0.23 kg COD removed}{kg MLVSS \cdot day}\right) - \frac{0.094}{d} = \frac{0.023 kg biomass which dies away}{kg biomass present \cdot day}$$

The typical values for the decay coefficient for a domestic sewage is in the range of 0.02 – 0.20. The computed value falls in the range for the domestic sewage decay coefficient.

24. Excess biomass production rate (ΔX)

$$\Delta X = Y_{COD}^{max} * Q * \frac{(S_{inf} - S_{eff})}{1 + b\theta x}$$

$$\Delta X = \frac{0.5 kg biomass}{kg substrate} * \frac{140363 m^3}{d} * \frac{\left(259.5 \frac{mg}{l} - 30.9 \frac{mg}{l}\right)}{\frac{1000 \frac{mg}{L}}{\frac{kg}{m^3}} \left(1 + \frac{0.023}{d} * 10.64 d\right)} = \frac{12834.79 kg biomass}{day}$$

25. Oxygen demand from substrate respiration (mean values)

$$OD = Q * (S_{inf} - S_{eff}) - 1.33 * \Delta x$$

$$OD = \frac{140363 \text{ m}^3}{d} * \frac{\left(259.5 \frac{\text{mg}}{\text{l}} - 30.9 \frac{\text{mg}}{\text{l}}\right)}{\frac{1000 \frac{\text{mg}}{\text{L}}}{\frac{\text{kg}}{\text{m}^3}}} - \left(1.33 * \frac{12834.79 \text{ kg}}{\text{day}}\right) = 15016.71 \frac{\text{kg O}_2}{d}$$

26. Oxygen demand from endogenous respiration

$$OD = \frac{0.1 \text{ kg O}_2}{\text{kg MLVSS} \cdot \text{day}} * X * V$$

$$OD = \frac{0.1 \text{ kg O}_2}{\text{kg MLVSS} \cdot \text{day}} * \frac{4 \text{ kg}}{\text{m}^3} * (2 * 17075 \text{ m}^3) = \frac{13660 \text{ kg O}_2}{\text{day}}$$

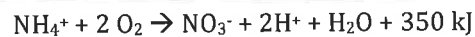
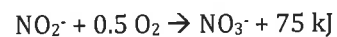
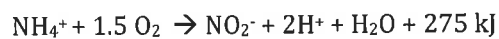
27. Nitrogen Oxygen Demand

$$NOD = (N_{av} - N_{immob}) * 4.33$$

$$N_{av} = \text{NH}_4^+ - N + \text{Organic N} * F_b$$

$$N_{immob} = COD * F_b * Y_{COD} * 0.05$$

In the nitrification process, oxygen is required to convert ammonia into nitrate as shown in the oxidation reactions below [1].



NOD for the sample collected at 2-1-2010

The given data was mg/l NH₄⁺; so it must be converted first into NH₄⁺-N which is :

$$NH_4^+ - N_{influent} = 19.50 \frac{mg}{l} * \frac{14mgN/mmol}{17 mgNH_4^+/mmol} = 16.05 \frac{mg}{l} NH_4^+ - N_{influent}$$

$$NH_4^+ - N_{effluent} = 0.20 \frac{mg}{l} * \frac{14mgN/mmol}{17 mgNH_4^+/mmol} = 0.16 \frac{mg}{l} NH_4^+ - N_{effluent}$$

$$Nav = NH_4^+ - N + Organic N * Fb$$

Since:

$$KJN = Organic N + total ammonia N$$

Therefore:

$$Organic N = KJN - NH_4^+ - N_{influent}$$

$$Nav = (NH_4^+ - N - NH_4^+ - N_{effluent}) + (KJN - NH_4^+ - N_{influent}) * Fb$$

$$NH_4^+ - N_{influent} = 16.05 \frac{mg}{l}$$

$$NH_4^+ - N_{effluent} = 0.16 \frac{mg}{l}$$

$$KJN = 27.70 \frac{mg}{l}$$

Therefore:

$$Nav = (16.05 mg/l - 0.16 mg/l) + (27.70 mg/l - 16.05 mg/l) * 0.8 = 26.15 mg/l$$

$$N_{immob} = COD * Fb * Y_{COD} * 0.05$$

$$N_{immob} = \frac{217 mg}{l} * 0.88 * 0.4 * 0.05 = 3.82 \frac{mg}{l}$$

$$NOD = \left(26.15 \frac{mg}{l} - 3.82 \frac{mg}{l} \right) * 4.33 = 98.67 \frac{mg O_2}{l}$$

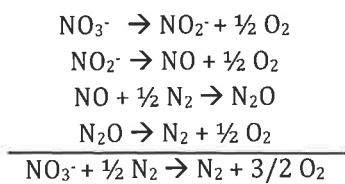
The same calculations were done on the samples obtained on the other dates and an average NOD is obtained (Table 12).

Table 12. Calculation of nitrogen oxygen demand (NOD) of samples collected on each date.

Date	NH ₄ ⁺ inf (mg/l)	NH ₄ ⁺ eff (mg/L)	KJN (mg/l)	Organic N (mg/L)	NH ₄ ⁺ net mg/l	Nav (mg/L)	COD (mg/L)	Nimmob (mg/l)	NOD (mg/l)
2-1-2010	19.50	0.20	27.70	11.64	19.30	26.15	217.00	3.82	96.67
12-1-2010	14.60	0.50	22.20	10.18	14.10	20.58	169.00	2.98	76.20
24-1-2010	16.30	0.50	25.40	11.98	15.80	23.56	243.00	4.28	83.49
2-2-2010	15.60	0.50	19.80	6.95	15.10	18.56	168.00	2.96	67.55
12-2-2010	14.00	0.20	15.00	3.47	13.80	14.42	173.00	3.05	49.25
19-2-2010	20.50	0.50	28.30	11.42	20.00	26.53	249.00	4.39	95.87
26-2-2010	19.80	0.20	42.40	26.09	19.60	39.13	526.00	9.27	129.30
2-3-2010	10.10	0.50	21.00	12.68	9.60	19.08	214.00	3.77	66.28
10-3-2010	10.20	0.60	18.90	10.50	9.60	17.16	162.00	2.85	61.93
17-3-2010	19.00	0.20	29.20	13.55	18.80	27.42	191.00	3.37	104.16
23-3-2010	21.40	0.20	32.00	14.38	21.20	30.12	268.00	4.72	109.99
28-3-2010	18.80	0.20	30.10	14.62	18.60	28.19	376.00	6.62	93.40
12-4-2010	11.70	1.10	25.40	15.76	10.60	22.62	278.00	4.90	76.72
28-4-2010	14.30	0.50	28.00	16.22	13.80	25.66	313.00	5.51	87.21
2-5-2010	20.40	0.50	33.40	16.60	19.90	31.01	302.00	5.32	111.24
10-5-2010	26.50	0.20	38.00	16.18	26.30	35.91	274.00	4.83	134.58
17-5-2010	14.40	0.50	26.10	14.24	13.90	23.99	228.00	4.02	86.49
23-5-2010	23.20	0.50	33.70	14.59	22.70	31.55	264.00	4.65	116.47
28-5-2010	21.50	0.20	34.00	16.29	21.30	31.90	295.00	5.20	115.60
8-6-2010	27.20	0.50	38.80	16.40	26.70	36.44	314.00	5.53	133.81
15-6-2010	25.60	0.50	40.80	19.72	25.10	38.04	456.00	8.03	129.93
20-6-2010	28.20	0.20	48.50	25.28	28.00	45.33	519.00	9.14	156.67
26-6-2010	22.40	0.20	38.90	20.45	22.20	36.30	479.00	8.44	120.64
8-7-2010	31.30	0.70	54.10	28.32	30.60	50.15	503.00	8.86	178.78
15-7-2010	33.80	0.50	44.40	16.56	33.30	42.02	382.00	6.73	152.79
20-7-2010	34.60	0.80	76.00	47.51	33.80	69.68	825.00	14.54	238.80
26-7-2010	39.70	0.50	54.00	21.31	39.20	51.05	397.00	6.99	190.77
Average	21.28	0.43	34.30	16.77	20.85	31.95	325.37	5.73	113.50

28. Nitrogen Oxygen Equivalent (NOE)

In denitrification, the nitrates are converted into N₂ as shown in the equation below ^[1].



As shown in the equation, conversion of nitrates or nitrates into dinitrogen gas releases oxygen. A mass of nitrogen oxygen equivalents (NOE) is the equivalent mass of oxygen that would accept as many electrons as the amount of nitrate or nitrite during reduction to dinitrogen.

$$NOE = \frac{20}{7} NO_3^- - N + \frac{12}{7} NO_2^- - N$$

Calculation for NOE for the sample collected on 2-1-2010 is shown below. It is assumed that all ammonia (including the biodegraded organic nitrogen subtracted by the immobilized N) in the nitrification process is converted into nitrate. Since the data given was mg/l NO_3^- , it is necessary to convert it into $NO_3^- - N$. 14 g N/ mol is the molar mass of nitrogen while 62 g NO_3^- /mol is the molecular mass of nitrate.

$$NO_3^- - N_{influent} = \frac{0.5 \text{ mg } NO_3^-}{L} * \frac{14 \frac{\text{mg N}}{\text{mmol}}}{62 \frac{\text{mg } NO_3^-}{\text{mmol}}} = \frac{0.11 \text{ mg } NO_3^- - N_{influent}}{L}$$

$$NO_3^- - N_{effluent} = \frac{6.1 \text{ mg } NO_3^-}{L} * \frac{14 \frac{\text{mg N}}{\text{mmol}}}{62 \frac{\text{mg } NO_3^-}{\text{mmol}}} = \frac{1.38 \text{ mg } NO_3^- - N_{effluent}}{L}$$

It can be seen that the nitrate in the effluent is higher than the influent. The reason for this is that the NH_4^+ which is converted into NO_3^- during the nitrification process was not completely reduced to N_2 gas and thus release as NO_3^- in the effluent. This increases the NO_3^- in the effluent from the NO_3^- obtained in the influent.

The data given was mg/l NO_2^- , it is necessary to convert it into $NO_2^- - N$ by multiplying it to 14/46 (14 is the molar mass of nitrogen and 46 g NO_2^- /mol is the molecular weight of nitrite).

$$NOE = \frac{20}{7} * (N_{av} - N_{immob}) + NO_3^- - N_{influent} - NO_3^- - N_{effluent} + \frac{12}{7} (NO_2^- - N_{influent} - NO_2^- - N_{effluent})$$

$$NOE = \frac{20}{7} * \left(\frac{22.33 \text{ mg } NO_3^-}{L} + \frac{0.11 \text{ mg } NO_3^- - N_{influent}}{L} - \frac{1.38 \text{ mg } NO_3^- - N_{effluent}}{L} \right) + \frac{12}{7} \left(\frac{0.33 \text{ mg } NO_2^-}{L} - \frac{0.04 \text{ mg } NO_2^-}{L} \right) * \frac{14 \frac{\text{mg N}}{\text{mmol}}}{46 \frac{\text{mg } NO_2^-}{\text{mmol}}} = \frac{66.93 \text{ mg } O_2}{L}$$

The same calculations were done on the samples collected on the other dates and the average was taken (Table 13).

Table 13. Calculation of NOE of samples collected from different dates.

Date	NO ₃ ⁻ in (mg/l)	NO ₃ -N in (mg/l)	NO ₃ -N eff (mg/l)	NO ₃ -N eff (mg/l)	NO ₂ ⁻ in (mg/L)	NO ₂ ⁻ eff (mg/l)	NO ₃ ⁻ -N (mg/l)	NO ₂ ⁻ -N (mg/l)	NOE (mg/l)
2-1-2010	0.5	0.11	6.10	1.38	0.33	0.04	23.37	0.09	66.93
12-1-2010	1.9	0.43	3.90	0.88	0.35	0.04	18.13	0.09	51.96
24-1-2010	1.3	0.29	3.10	0.70	0.21	0.04	19.77	0.05	56.58
2-2-2010	1.6	0.36	3.80	0.86	0.26	0.04	16.20	0.07	46.40
12-2-2010	2	0.45	2.70	0.61	0.24	0.04	11.74	0.06	33.66
19-2-2010	0.3	0.07	5.00	1.13	0.43	0.07	22.84	0.11	65.45
26-2-2010	0.3	0.07	3.60	0.81	0.04	0.04	30.63	0.00	87.52
2-3-2010	1.7	0.38	4.20	0.95	0.16	0.10	16.10	0.02	46.02
10-3-2010	2.4	0.54	3.40	0.77	0.23	0.04	14.84	0.06	42.50
17-3-2010	0.3	0.07	3.80	0.86	0.02	0.02	24.89	0.00	71.13
23-3-2010	0.6	0.14	3.50	0.79	0.08	0.02	26.11	0.02	74.64
28-3-2010	0.3	0.07	2.50	0.56	0.06	0.02	22.07	0.01	63.09
12-4-2010	1.4	0.32	3.60	0.81	0.13	0.13	18.40	0.00	52.58
28-4-2010	0.9	0.20	2.50	0.56	0.53	0.06	20.18	0.14	57.89
2-5-2010	0.3	0.07	2.30	0.52	0.57	0.04	25.64	0.16	73.53
10-5-2010	0.3	0.07	3.60	0.81	0.12	0.04	31.77	0.02	90.83
17-5-2010	1.4	0.32	2.20	0.50	0.16	0.04	20.31	0.04	58.10
23-5-2010	0.5	0.11	2.60	0.59	0.07	0.04	27.42	0.01	78.35
28-5-2010	0.3	0.07	2.20	0.50	0.16	0.10	27.03	0.02	77.27
8-6-2010	0.5	0.11	3.40	0.77	0.23	0.11	31.44	0.04	89.89
15-6-2010	0.3	0.07	2.80	0.63	0.02	0.02	30.62	0.00	87.48
20-6-2010	0.5	0.11	1.50	0.34	0.02	0.02	36.50	0.00	104.29
26-6-2010	0.3	0.07	1.50	0.34	0.15	0.12	28.05	0.01	80.16
8-7-2010	0.5	0.11	3.50	0.79	0.02	0.02	42.06	0.00	120.17
15-7-2010	0.3	0.07	1.80	0.41	0.02	0.02	35.67	0.00	101.92
20-7-2010	0.3	0.07	3.80	0.86	0.04	0.04	55.97	0.00	159.91
26-7-2010	0.3	0.07	2.20	0.50	0.02	0.02	44.53	0.00	127.24
Average	0.79	0.18	3.15	0.71	0.17	0.05	26.75	0.04	76.50

29. Total Oxygen Demand

The total oxygen demand is the sum of the bCOD and the NOD subtracted by the NOE.

$$OD = bCOD + NOD - NOE$$

The OD for the sample in 2-1-2010 is calculated:

$$bCOD = 191.16 \frac{mg}{L}$$

$$NOD = 96.67 \frac{mg}{L}$$

$$NOE = 66.93 \frac{mg}{L}$$

Therefore:

$$OD \left(\frac{mg}{L} \right) = 191.16 \frac{mg}{L} + 96.67 \frac{mg}{L} - 66.93 \frac{mg}{L} = 220.90 \frac{mg O_2}{L}$$

Conversion of OD into kg/m³

$$OD \left(\frac{kg}{m^3} \right) = \frac{OD \left(\frac{mg}{L} \right)}{\frac{1000 \frac{mg}{L}}{\frac{kg}{m^3}}}$$

$$OD \left(\frac{kg}{m^3} \right) = \frac{220.90 \frac{mg}{L}}{\frac{1000 \frac{mg}{L}}{\frac{kg}{m^3}}} = 0.22 \frac{kg}{m^3}$$

Conversion of OD into kg/day

$$OD \left(\frac{kg}{d} \right) = Q \left(\frac{m^3}{d} \right) * OD \left(\frac{kg}{m^3} \right)$$

$$OD \left(\frac{kg}{day} \right) = 78487.00 \frac{m^3}{d} * 0.22 \frac{kg}{m^3} = \frac{17337.75 kg}{day}$$

The same calculations were done on the samples collected at different dates and an average was obtained (Table 14). It is shown that oxygen demand required per day is high. The aeration costs constitute one-third of the operational cost.

Table 14. Calculation of Oxygen Demand.

Date	bCOD (mg/l)	NOD (mg/l)	NOE (mg/l)	OD (mg/L)	OD (kg/m3)	Q (m3/day)	OD (kg/day)
2-1-2010	191.16	96.67	66.93	220.90	0.22	78487.00	17337.75
12-1-2010	148.88	76.20	51.96	173.12	0.17	104240.00	18045.94
24-1-2010	214.06	83.49	56.58	240.97	0.24	91462.00	22039.96
2-2-2010	148.00	67.55	46.40	169.15	0.17	106741.00	18055.13
12-2-2010	152.40	49.25	33.66	167.99	0.17	101634.00	17073.67
19-2-2010	219.35	95.87	65.45	249.78	0.25	80225.00	20038.34
26-2-2010	463.37	129.30	87.52	505.14	0.51	75481.00	38128.39
2-3-2010	188.52	66.28	46.02	208.78	0.21	179693.00	37516.34
10-3-2010	142.71	61.93	42.50	162.14	0.16	153531.00	24893.24
17-3-2010	168.26	104.16	71.13	201.29	0.20	72523.00	14598.43
23-3-2010	236.09	109.99	74.64	271.44	0.27	70603.00	19164.48
28-3-2010	331.23	93.40	63.09	361.54	0.36	78917.00	28531.27
12-4-2010	244.90	76.72	52.58	269.04	0.27	170053.00	45751.65
28-4-2010	275.73	87.21	57.89	305.05	0.31	128411.00	39171.99
2-5-2010	266.04	111.24	73.53	303.75	0.30	74300.00	22568.35
10-5-2010	241.37	134.58	90.83	285.13	0.29	63088.00	17988.33
17-5-2010	200.85	86.49	58.10	229.25	0.23	115620.00	26505.54
23-5-2010	232.56	116.47	78.35	270.69	0.27	72792.00	19704.04
28-5-2010	259.87	115.60	77.27	298.20	0.30	68358.00	20384.43
8-6-2010	276.61	133.81	89.89	320.53	0.32	64600.00	20706.07
15-6-2010	401.70	129.93	87.48	444.15	0.44	115432.00	51268.83
20-6-2010	457.20	156.67	104.29	509.58	0.51	51701.00	26345.77
26-6-2010	421.96	120.64	80.16	462.44	0.46	117250.00	54221.43
8-7-2010	443.11	178.78	120.17	501.72	0.50	49129.00	24648.84
15-7-2010	336.51	152.79	101.92	387.38	0.39	48683.00	18858.81
20-7-2010	726.76	238.80	159.91	805.65	0.81	45414.00	36587.90
26-7-2010	349.73	190.77	127.24	413.25	0.41	45960.00	18993.16
Average	286.63	113.50	76.50	323.63	0.32	89789.93	26634.37

30. Aeration cost

It is assumed that the OC/load is 2 so the oxygen required have to be multiplied twice than what is actually needed. It would also consume 1 KWh for every 1.5 kg O₂ and the cost for 1 KWh is 0.10 Euro.

The sample collected in 2-1-2010 is

$$\text{Cost} = \text{OD} * \frac{\text{OC}}{\text{load}} * \frac{1 \text{ KWh}}{1.5 \text{ kg O}_2} * \frac{0.10 \text{ Euro}}{\text{KWh}}$$

$$\text{Cost} = \frac{17337.75 \text{ kg}}{\text{day}} * 2 * \frac{1 \text{ KWh}}{1.5 \text{ kg O}_2} * \frac{0.10 \text{ Euro}}{\text{KWh}} = 3467.55 \frac{\text{Euro}}{\text{day}}$$

The same calculations were done on the samples collected at different dates and an average was obtained (Table 15).

Table 15. Calculation of the aeration cost.

Date	OD (kg/day)	OC load (kg O ₂ /day)	Energy (KWh/day)	Cost (Euro/day)
2-1-2010	17337.75	34675.50	23117.00	3467.55
12-1-2010	18045.94	36091.88	24061.26	3609.19
24-1-2010	22039.96	44079.92	29386.61	4407.99
2-2-2010	18055.13	36110.27	24073.51	3611.03
12-2-2010	17073.67	34147.35	22764.90	3414.73
19-2-2010	20038.34	40076.68	26717.79	4007.67
26-2-2010	38128.39	76256.78	50837.85	7625.68
2-3-2010	37516.34	75032.67	50021.78	7503.27
10-3-2010	24893.24	49786.48	33190.99	4978.65
17-3-2010	14598.43	29196.85	19464.57	2919.69
23-3-2010	19164.48	38328.97	25552.64	3832.90
28-3-2010	28531.27	57062.54	38041.69	5706.25
12-4-2010	45751.65	91503.31	61002.21	9150.33
28-4-2010	39171.99	78343.98	52229.32	7834.40
2-5-2010	22568.35	45136.71	30091.14	4513.67
10-5-2010	17988.33	35976.65	23984.44	3597.67
17-5-2010	26505.54	53011.08	35340.72	5301.11
23-5-2010	19704.04	39408.09	26272.06	3940.81
28-5-2010	20384.43	40768.86	27179.24	4076.89
8-6-2010	20706.07	41412.14	27608.09	4141.21
15-6-2010	51268.83	102537.66	68358.44	10253.77
20-6-2010	26345.77	52691.55	35127.70	5269.15
26-6-2010	54221.43	108442.85	72295.23	10844.29
8-7-2010	24648.84	49297.69	32865.12	4929.77
15-7-2010	18858.81	37717.61	25145.07	3771.76
20-7-2010	36587.90	73175.80	48783.87	7317.58
26-7-2010	18993.16	37986.32	25324.22	3798.63
Average	26634.37	53268.75	35512.50	5326.87

31. Phosphate removal

As a rule of thumb, the RbCOD is 1/3 of the bCOD [4]. The RbCOD of the sample collected in 2-1-2010 is computed as:

$$RbCOD = \frac{bCOD}{3} = \frac{191.16 \frac{mg}{L}}{3} = 63.72 \frac{mg}{L}$$

As a rule of thumb, the average biomass contains 7.5 % P. The % P for the biomass formed is for the sample collected in 2-1-2010 is

$$P \text{ in biomass } \left(\frac{kg}{d} \right) = \frac{bCOD \left(\frac{mg}{L} \right)}{\frac{1000 \frac{mg}{L}}{\frac{kg}{m^3}}} * Y * Q * 0.075$$

$$P \text{ in biomass } = \frac{191.16 \frac{mg}{L}}{\frac{1000 \frac{mg}{L}}{\frac{kg}{m^3}}} * \frac{0.4 \text{ kg MLVSS}}{\text{kg COD removed}} * 78487 \frac{m^3}{d} * 0.075 = \frac{450.11 \text{ kg}}{\text{day}}$$

To compute for total P that is treated for the sample collected in 2-1-2010.

$$P_{tot \text{ treated}} \left(\frac{kg}{d} \right) = \frac{P_{tot \text{ inf}} \left(\frac{mg}{L} \right) - P_{tot \text{ eff}} \left(\frac{mg}{L} \right)}{\frac{1000 \frac{mg}{L}}{\frac{kg}{m^3}}} * Q \left(\frac{m^3}{d} \right)$$

$$P_{tot \text{ treated}} \left(\frac{kg}{d} \right) = \frac{4.4 \frac{mg}{L} - 2.2 \frac{mg}{L}}{\frac{1000 \frac{mg}{L}}{\frac{kg}{m^3}}} * 78487 \frac{m^3}{\text{day}} = 172.67 \frac{kg}{\text{day}}$$

The same calculations were done on the samples collected on other dates and an average of P in the biomass (kg/d) and the tot P that is treated (kg/day) was computed (Table 16). It is observed that

the computed P found in the biomass is almost twice higher than the treated P. The reason for this could be that the composition of the sludge is varied and that, it may not necessarily that the biomass contains 7.5% phosphorus; it could be lower than 7.5%. Another reason could be that the uptake of microorganisms for phosphates during the aerobic phase is not so efficient.

Table 16. Calculation for phosphate removal of each samples collected at different dates.

Date	P _{tot} inf (mg/l)	P _{tot} eff (mg/l)	P _{tot} treated (mg/l)	Q (m ³ /d)	P _{tot} treated (kg/d)	bCOD (mg/l)	biomass formed (mg/l)	biomass formed (kg/d)	P in the biomass (kg/d)	RbCOD (mg/l)
2-1-2010	4.40	2.20	2.20	78487.00	172.67	191.16	76.46	6001.45	450.11	63.72
12-1-2010	3.30	1.20	2.10	104240.00	218.90	148.88	59.55	6207.55	465.57	49.63
24-1-2010	4.50	1.20	3.30	91462.00	301.82	214.06	85.63	7831.52	587.36	71.35
2-2-2010	3.00	1.00	2.00	106741.00	213.48	148.00	59.20	6318.87	473.92	49.33
12-2-2010	3.20	1.20	2.00	101634.00	203.27	152.40	60.96	6195.61	464.67	50.80
19-2-2010	5.30	1.90	3.40	80225.00	272.77	219.35	87.74	7038.95	527.92	73.12
26-2-2010	8.70	1.80	6.90	75481.00	520.82	463.37	185.35	13990.15	1049.26	154.46
2-3-2010	3.50	1.10	2.40	179693.00	431.26	188.52	75.41	13550.14	1016.26	62.84
10-3-2010	3.20	0.80	2.40	153531.00	368.47	142.71	57.08	8764.15	657.31	47.57
17-3-2010	4.50	1.30	3.20	72523.00	232.07	168.26	67.30	4880.99	366.07	56.09
23-3-2010	5.10	0.70	4.40	70603.00	310.65	236.09	94.44	6667.40	500.06	78.70
28-3-2010	6.40	0.80	5.60	78917.00	441.94	331.23	132.49	10455.80	784.19	110.41
12-4-2010	5.50	0.80	4.70	170053.00	799.25	244.90	97.96	16658.20	1249.36	81.63
28-4-2010	5.50	0.70	4.80	128411.00	616.37	275.73	110.29	14162.68	1062.20	91.91
2-5-2010	6.10	1.40	4.70	74300.00	349.21	266.04	106.42	7906.69	593.00	88.68
10-5-2010	6.10	0.90	5.20	63088.00	328.06	241.37	96.55	6091.11	456.83	80.46
17-5-2010	4.80	0.50	4.30	115620.00	497.17	200.85	80.34	9288.95	696.67	66.95
23-5-2010	6.70	0.80	5.90	72792.00	429.47	232.56	93.03	6771.52	507.86	77.52
28-5-2010	7.70	0.80	6.90	68358.00	471.67	259.87	103.95	7105.75	532.93	86.62
8-6-2010	8.00	1.30	6.70	64600.00	432.82	276.61	110.64	7147.61	536.07	92.20
15-6-2010	8.00	0.90	7.10	115432.00	819.57	401.70	160.68	18547.69	1391.08	133.90
20-6-2010	11.40	0.70	10.70	51701.00	553.20	457.20	182.88	9455.08	709.13	152.40
26-6-2010	10.60	0.50	10.10	117250.00	1184.23	421.96	168.79	19790.06	1484.25	140.65
8-7-2010	13.80	0.80	13.00	49129.00	638.68	443.11	177.24	8707.73	653.08	147.70
15-7-2010	8.20	0.80	7.40	48683.00	360.25	336.51	134.61	6552.99	491.47	112.17
20-7-2010	25.00	0.70	24.30	45414.00	1103.56	726.76	290.71	13202.09	990.16	242.25
26-7-2010	8.90	0.50	8.40	45960.00	386.06	349.73	139.89	6429.38	482.20	116.58
Average					468.80				710.33	

32. Inhabitant Equivalent (IE)

$$IE = \frac{Q(\text{COD} + 4.57N)}{\left(\frac{180\text{g}}{\text{d}}\right)}$$

where:

Q : flow of waste water ($\frac{m^3}{d}$)

COD : $\frac{mg}{l}$ or $\frac{g}{m^3}$

N : KJN in $\frac{mg}{l}$ or $\frac{g}{m^3}$

$$IE = \frac{140363 \text{ m}^3}{d} * \frac{\left(259.5 \frac{mg}{l} + 4.57 * 26.82 \frac{mg}{l}\right)}{\left(\frac{180g}{d}\right)} = 297934.03$$

It shows that the inhabitant equivalent is higher than the number of inhabitants in which the domestic waste water is collected. There are 170000 inhabitants while the inhabitant equivalent is 297,000. It is about 100,000 higher than the actual inhabitants in which the waste water was collected. The reason for this could be that people might be discharging more waste water and there are some small companies that get their waste water treated by

33. Sedimentation-selection time ($t_{sed-sel}$)

To have a good effluent quality, free from washed out sludge, the parameter sedimentation-selection time is used [1],

$$t_{sed-sel} = \frac{V_{tot} (m^3)}{Q \left(\frac{m^3}{d}\right)}$$

where:

$$V_{tot} = V_{reactor} + V_{sedimentation \ tank}$$

In the wastewater treatment plant, at dry weather flow (DWF), 2 aeration tanks and 4 sedimentation tank operate. Therefore:

$$V_{tot} = (2 * 17075m^3 + 4 * 5334 m^3) = 55486m^3$$

$$t_{sed-sel} = \frac{55486m^3}{\frac{140363m^3}{day}} = 0.40 \text{ day} * 24 \frac{hour}{day} = 9.48 \text{ hour}$$

In practice, the $t_{sed-sel}$ should be less than 8 hours [1]. Higher than 8 might cause overflow of sludge which results to bad effluent quality. The calculated $t_{sed-sel}$ on the other hand is higher than 8 hours; one reason for this is that the water flow used in the calculation was an average. Since water flow is very variable depending on the season and time of the day, $t_{sed-sel}$ could be lower than 8 hours. There are 4 aeration tanks as well as 7 sedimentation tanks in the treatment plant. When the water flow would be too high due to rain or high peak hours, the remaining aeration and sedimentation tanks can be used to accommodate the high water flows.

34. Weir loading rate

$$weir \text{ loading rate} = Q \left(\frac{m^3}{d} \right) * \frac{d}{24h} * \frac{1}{(4 * \pi * D) m}$$

$$weir \text{ loading rate} = \frac{140363m^3}{day} * \frac{d}{24h} * \frac{1}{(4 * \pi * 5334) m} = 0.088 \frac{m^3}{mh}$$

It shows that the weir loading rate for the average flow rate is lower than 3 – 5 m³/mh, which is good.

IV. ECONOMIC ASPECTS

The investment and operational cost can be seen on Table 17.

Table 17. Investment, Maintenance and Operating Cost of the Waste Water Treatment Plant.

Investment		
Flemish govt. installation (25 years ago)	16.5	M Euro
installation	17.5	M Euro
Land	2.5	M Euro
Total	36.5	M Euro
Maintenance & Operating Cost		
Energy (Aeration)	2	M Euro/yr
Personnel	0.2	M Euro/yr
Sludge treatment	2	M Euro/yr
Maintenance and other operation costs	1.7	M Euro/yr
Total	6	M Euro/yr

Twenty-five years ago, the waste water treatment plant was operated and owned by the Flemish government. The installations were therefore built by the government. After 1999, the waste water treatment plant (WWTP) was given to installed some other treatment

installations and an additional capital cost was obtained. About 1/3 of the maintenance and operating cost goes to the aeration. The sludge treatment also consumes 1/3 of the maintenance and operating cost.

V. RECOMMENDATIONS

It can be shown from the calculations that the average amount of sludge produced a day is 12834.79 kg. This amount of sludge is a lot. With the production of this amount of sludge, the cost for the sludge treatment is also high. To solve this problem, it is recommended to treat the sludge in an anaerobic digestion and recover energy from it to minimize the cost for the treatment of the sludge. The amount of phosphate treated is also high which on average is 468.80/day. This can be a good opportunity to do some research on how to recover phosphorus and that it might be used as a fertilizer, but only in accordance with the environmental legislation.

VI. CONCLUSION

According to the calculation, this treatment plant has already maximized the use of a conventional activated sludge system. This system has several advantages:

- Relatively low capital and operational cost [2]
- Efficient removal of BOD, COD and nutrients (e.g. nitrogen, phosphate) [2]
- Good quality of effluent [2]
- Small head loss [2]
- Flexible adaptation of the processes inside the whole system, e.g. application of Anammox bacteria to remove nitrogen, application of UCT lay out to remove phosphate [2]

There are also several disadvantages of this system:

- Not a flexible method to treat sudden increase of sewage water with permanent setting size of the plant, and thus some amount of untreated water might be discharged to environment [2]
- High operational cost due to constant energy supply requirement [2]
- Large scale sludge disposal [2]
- Sensitive to certain industrial wastes [2]
- Require skilled supervision to make sure the returned sludge remains active [2]

However, when compared with other systems, this activated sludge system is still an effective and low cost system. The conventional activated system is also more sustainable compared to other treatment system. So it is still a good option to continue using this system for this treatment plant at the moment.

VII. REFERENCES

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